Errata and New Materials

The last entirely new edition of the Rainwater and Land Development manual was issued in November of 2006, but new materials have been added and changes and corrections made since that time to the manual text or figures. All of these have been included in subsequent printings and in files posted to the ODNR Rainwater and Land Development manual web page.

The following corrections or additions have been made since the first printing of the 2006 Rainwater and Land Development manual in November of 2006.

On **October 1, 2007**, the following pages were changed or corrected:
Table of Contents (both pages);
Chapter 1 pages 1, 2, 3, 4, 7, 8, 9, and 10;
Chapter 2 pages 1, 30, 33, 34, 36, 39, 40, 47, 48, 51, 60, 61, 72, Practice Note regarding Bioretention soils added;
Chapter 4 page 1;
Chapter 5 pages 1, 11, and 19;
Chapter 6 pages 6, 7, 8, 24, 25, 27, and 41;
Chapter 7 pages 1, 4 and 5.

On **November 29, 2007**, the following pages were changed or corrected:
Chapter 2 pages 44, and 76.

On **June 23, 2009**, the following pages were changed or corrected:
Chapter 2 pages 44 (equation rewritten but effectively unchanged);
Chapter 5 page 29 (practice numbering corrected);
Chapter 6 page 21 (practice numbering);
Chapter 7 pages 3, 7, 10, and 17 (practice numbering);
Appendix 6 pages 1-18 (new text-pg. 1, removed extra tables & a typo).

On **May 7, 2012**, the following pages were changed, corrected or added:
Table of Contents (both pages re: Perm Pavement and Appendixes 1, 4, 9, 10 and 11);
Chapter 2 (Page 1 table of contents changed, and pages 85-112 added);
Appendix 1 added;
Appendix 4 pages 2, 3, 4 and 5 updated;
Appendix 9 added;
Appendix 10 added;
Appendix 11 added.

On **August 30, 2012**, the following pages were changed, corrected or added:
Chapter 2 page 90 an overlapping figure and caption were corrected;
Chapter 2 pages 95, 96, and 98 properly formatted text (i.e. Italic and subscript);

If the printing date is known, then a set of new and updated pages can be obtained from the ODNR Rainwater and Land Development web page. See the errata tab, at this address: http://www.dnr.state.oh.us/soilandwater/water/rainwater/default/tabid/9186/Default.aspx. All new materials regarding the manual can be found at this site or by emailing or calling the Ohio Department of Resources, Division of Soil and Water Resources (614-265-6685 or dswc@dnr.state.oh.us).

This manual was updated on the following date ____________. Initials_________

Title: Rainwater and Land Development: Ohio's Standards for Stormwater Management, Land Development and Urban Stream Protection

Date: December 2006

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Abstract: Stream systems, including their corridors, and wetland resources are vital environmental features and are extremely sensitive to urbanization. The intent of this book is to allow development to occur while minimizing the impact on water resources, especially streams.

This book defines Ohio's standards and specifications for stormwater practices implemented during land development. It is an update of the previous Rainwater and Land Development book completed in January 1996. The target audience is that group of professionals involved in the design and implementation of development projects.

This book aims to integrate water resource protection into development site planning in order to maintain or improve stream integrity. Early chapters discuss practices and strategies for protecting streams and wetlands, treating stormwater pollutants, rehabilitating streams and establishing permanent runoff controls. The latter portion of the book includes chapters regarding construction-phase practices, including standards and specifications for sediment control, temporary runoff control, soil stabilization and control of pollutants other than sediment. Appendixes offer further information regarding stormwater design examples, permits, helpful contacts, and soils.

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CONTENTS

We want to acknowledge all of the people who deserve credit for helping prepare this significant improvement to Ohio's *Rainwater and Land Development* manual – they were many.

Initially there were the ODNR Division of Soil and Water Conservation's traditional conservation partners: Ohio's soil and water conservation districts, the Ohio Environmental Protection Agency, the USDA Natural Resources Conservation Service and The Ohio State University Extension. Ultimately, many other individuals, representing the development and consulting industry, and local government became involved. They contributed suggestions, photos and content for this manual. Some helped by participating on a Rainwater and Land Development committee or subcommittee, by writing material or perhaps by reviewing drafts as they were developed. All who have contributed their time and efforts have our sincere thanks for their contributions. And we hope all will remain involved in our work to make further improvements in the future. Finally, from within the ODNR, we thank John Mathews for his leadership.

Samuel W. Speck, Director
Ohio Department of Natural Resources

David Hanselmann, Chief
Division of Soil and Water Conservation

CHAPTER 1

Selecting Stormwater Management Practices for Development Projects

This chapter describes common impacts to prevent as well as the major objectives to apply in order to protect water resources during the development process. Understanding the nature of the impacts prepares site designers to better manage these through alternative site layout and the implementation of practices. Principles utilized during design are provided through stormwater management objectives that also direct designers to the appropriate portions of the manual for applications.

1.1 Adverse Effects of New Development on Water Resources

In order to protect water resource integrity, several impacts must be addressed during development:

- Hydrologic changes of the landscape
- Changes to the drainage network – streams, ditches, swales, waterways
- Increased delivery of warmer, more polluted runoff

Hydrologic Changes

How water is intercepted, stored, used, lost or gained changes substantially after development. Less rainfall is intercepted and utilized by vegetation after development. Less rainfall is infiltrated and percolated into the soils and groundwater following development. And less rainfall is stored in or on top of the ground following storms. All of these hydrologic changes result in more stormwater runoff reaching creeks or rivers faster than before development.

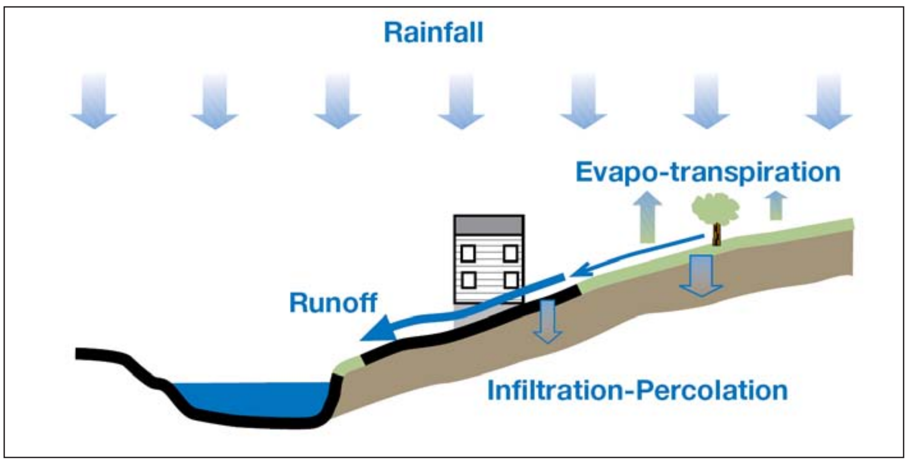
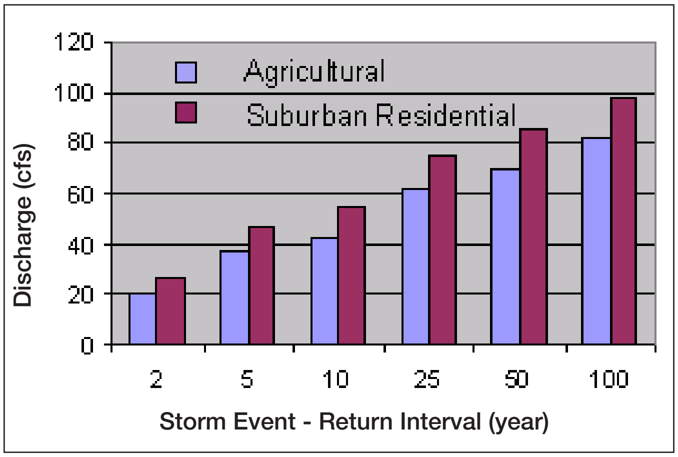


Figure 1.1.1 The hydrologic cycle is altered during urbanization.

The total volume of runoff increases significantly after development as rooftops, roads and hard surfaces replace soil and vegetation. There are other corresponding changes in the hydrology of a developed site. The hardening of a watershed, the compaction of soils, and the direct impacts to streams contribute to them becoming flashier, that is, flows quickly rising and quickly diminishing after each rainfall. Groundwater, normally replenished by percolating rainfall, receives lower levels of recharge in urban areas, affecting both the human and natural communities dependent on groundwater. Wetlands and small streams that require groundwater recharge to sustain them are impacted hydrologically. In its extreme, reduced groundwater recharge, with the subsequent reduction in base flow, may cause former perennially flowing streams to cease flowing during dry periods.

As watersheds urbanize and contribute more runoff, downstream areas experience greater flooding and longer duration flows. It's important to note that even as communities enact flood control strategies, there is still more flow in streams after development that increases flooding and stream erosion.



Storm Event Return Interval (years)	Pre-development Discharge (cfs)	Post-development Discharge (cfs)	Percent Increase
2	21	27	29%
5	37	47	27%
10	43	55	28%
25	61	75	23%
50	70	85	21%
100	82	98	20%

Figure 1.1.2 Stream discharge increases as land use changes from cropped agricultural land to residential using USGS empirical equations for estimating discharge for small urban streams.

To illustrate changes in peak runoff from urbanization, stream discharges were calculated for a typical development site in Eastern Franklin County, Ohio. The peak discharges were estimated for the pre-development condition (agriculture) and the resulting residential development using empirical equations developed by the U.S. Geological Survey (Sherwood, 1986[1]). The table and graph above show the resulting increases in stream discharges. The result was an average 25% increase for the 2, 5, 10, 25, 50 and 100-year return intervals in the estimated peak stream discharges. In water cycle terms, as land urbanizes, significantly less rainfall infiltrates and transpires from vegetation, therefore substantially increasing runoff to streams.

Storm water runoff to this ravine has caused over 2 feet of vertical stream erosion (incision).

Stream Instability and Consequences

As faster and higher stream flows occur on a regular basis, stream channels typically respond by adjusting their shape and size through erosion. Unfortunately, the typical pattern in urban areas is that a healthy stream with a naturally stable form (where bank erosion is balanced by floodplain deposition) becomes physically degraded. The stream cuts downward, losing access to its floodplain and the many functions provided by the floodplain and stream corridor. These deeply entrenched urban streams provide less storage and treatment of stormwater runoff along their corridor than healthy channels. These streams are plagued with bank erosion, contribute more sediment to downstream areas and rarely maintain high quality habitat features such as clean gravel substrates, deep pools and stable riffles. While rehabilitation of degraded streams is possible, the high cost and difficulties associated with working near developed properties makes it critical to prevent these problems in the first place. It makes sound economic and ecological sense.

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[1] Sherwood, J.M. 1986. *Estimating Peak Discharge, Flood Volumes, and Hydrograph Shapes of Small Ungaged Urban Streams in Ohio*. USGS Water Resources Investigation Report 86-4197, 52 pp.

Urbanization and Stream Decline

Studies indicate that even at low levels of urbanization (5-10% imperviousness) stream ecosystems begin to rapidly decline (Schueler, 1994[1]). These urban streams lose much of their biological diversity, leaving only populations of pollution tolerant species. A study[1] by the Ohio Environmental Protection Agency of 110 urban sites found poor or very poor scores at the majority of the urban impacted sites (85%). More than 40% of suburban sites were impaired with many reflecting the negative impacts of new developments for housing and commercial uses. As this study stated, "The results demonstrate the degree of degradation which exists in most small urban Ohio watersheds and the difficulties involved in dealing with these multiple and diffuse sources of stress."

[1] Schueler, T. 1994. The Importance of Imperviousness. *Watershed Protection Techniques.* 1(3): 100-111.

[1]Yoder, C.O. and E.T. Rankin. 1996. Assessing the condition and status of aquatic life designated uses in urban and suburban watersheds, pp. 201-227. in Roesner, L.A. (ed.). Effects of Watershed Development and Management on Aquatic Ecosystems, American Society of Civil Engineers, New York, NY.

1.2 Stormwater Management Objectives for Development

This section presents stormwater objectives a site developer must address in planning and designing a development project which protects water resource integrity, along with the specific management practices available for addressing these objectives, and the appropriate chapter in this manual where guidance about these practices may be found.

1. Preserve the natural drainage system and important water resources

The natural drainage system that exists prior to development provides many benefits ranging from stormwater management and drainage services, to natural pollutant removal and wildlife habitat. For this reason, designers must preserve as much of the existing stream system as possible, by preserving streams, their corridors (streamways) and wetlands and by minimizing the extent that storm drains and constructed ditches replace natural drainage ways. Even open swales and ephemeral drainage without well-defined channels provide valuable stormwater benefits and should be preserved where possible.

Developments that build too close to watercourses may cause significant problems after the property is occupied since:

- structures on active floodplain areas may be damaged by flooding
- loss of natural floodplains increases flooding and pollutant loads, and decreases natural stream stability elsewhere along streams
- property or infrastructure may be damaged by natural stream migration or movement (meandering), or the elevated groundwater associated with saturated floodplains or wetlands
- stream integrity may be degraded due to the loss of the natural riparian corridor

For these reasons, this manual strongly encourages site designers to begin site layout by defining the existing drainage system, an adequate stream corridor, and floodplains based on the projected built out conditions in the watershed. The following management practices in Chapter 2 and additional resources will help achieve this objective:

• Wetland Setbacks	Chapter 2, Page 15
• Stream Setbacks	Chapter 2, Page 21
• Stream and wetland permitting	Appendix 2

2. Minimize imperviousness of the proposed development

Minimizing imperviousness must be a major objective during site layout. Numerous studies show that increases in pollutant loads, runoff volumes, and peak discharge rates are directly related to increases in the impervious areas within a watershed or project area. The greatest opportunity to reduce imperviousness lays in the sizing and layout of streets and parking areas. Parking standards traditionally have promoted having excess parking even during peak use. Some communities have begun to modify parking requirements to reflect stormwater concerns. Where building regulations and zoning allow, options such as reduced parking ratios or shared parking, clustered development reduce total site imperviousness and often reduce development costs.

Streets should be designed for the minimum pavement area to support the uses and the traffic based on the expected volume of traffic and the access needed. Alternative residential street layouts that maximize the number of homes per length of street also help reduce overall imperviousness. In commercial development, separating frequently utilized parking areas from rarely used areas provides an opportunity to use alternative parking materials such as modular pavers that reduce runoff and allow some infiltration in low use areas. Even without changing ordinances and development regulations, site designers have potential to reduce excess imperviousness by not exceeding minimum standards and utilizing shared parking between compatible uses, and variances may be an option. Of course, the area not used for parking must be replaced with open space or landscaping, not with other impervious surfaces.

Where hard surfaces cannot be reduced for development goals to be achieved, it may be possible to "disconnect" impervious surfaces from the drainage system and provide opportunities for runoff from small storms to percolate into the soil.

The following management practices in Chapter 2 may be used to achieve this objective:

- Low Impact Development — Chapter 2, Page 3
- Impervious Area Reductions — Chapter 2, Page 5
- Conservation Subdivision Design Principles — Chapter 2, Page 11

3. Improve degraded streams

In many cases, a watercourse through or adjacent to a development project has been degraded by past land uses and/or upstream development. Occasionally a developer may be relocating portions of a degraded watercourse during development. Developers may also be required to restore watercourses to mitigate for on-site impacts. In other cases, eroding channels may need special measures to prevent further degradation that can be more easily addressed during development or can prevent substantial future property loss.

In any of these cases, guidance in Chapter 3 can be used to address issues of degraded stream resources. Ultimately, by promoting or maintaining the naturally sustainable functions of these streams, many valuable stormwater management and water quality services will be provided via functional streams in addition to those provided through the still critical individual best management practices. Protection and restoration of floodplain and stream resources provide benefits of sediment reduction, nutrient removal and higher quality stream habitats. Restoration and rehabilitation of streams is best accomplished before or during development, since there are fewer impediments regarding stream access, and movement of materials and equipment. Costs increase and managing restoration/rehabilitation projects become more difficult as the area around the stream becomes more developed. Other issues such as the use of soil from previously filled floodplains can be more easily solved if coordinated with the site development plans.

The practices and reference materials in Chapter 3 and Appendix 7 may be used to address unstable and degraded streams.

4. Plan additions to the site drainage system that are stable and sustainable

Often the changes in runoff that occur during development will subject some areas, such as existing swales or watercourses to increased erosion. Areas of particular concern on developed sites include outlets from storm sewers and detention facilities, open drainage ways, areas receiving concentrated flow, and slopes.

Chapter 4 of this manual provides guidance on permanent runoff controls that typically must be installed during development to convey runoff and prevent accelerated erosion.

5. Manage post-construction runoff

Nearly every development project will require measures to control the impacts of increased runoff from the project. Those impacts include:

- Higher peak discharges
- Increased runoff volume
- Accelerated flow velocity
- Elevated pollutant loading

An effective system of stormwater runoff controls will address the increased energy and frequency of peak flows, as well as the increased pollutant load in the runoff. These controls must be in place throughout the watershed to address the cumulative impacts of urban land uses on stream stability, downstream flooding and water quality.

Stormwater management practices typically perform multiple functions including flood control, pollutant removal, and reducing downstream erosion potential. Stormwater practices can be integrated into the landscaping, drainage network, and other open spaces of development projects. Properly designed they can become amenities rather than impediments to development projects.

Chapter 2 and Appendix 1 provide guidance on the design, construction, and maintenance of the most common stormwater management practices that incorporate water quality and stream protection applications. The control practices described have: a proven performance track record, wide applicability within Ohio, and extensive resources available about their design, construction, and maintenance. Alternative measures are evolving but should be considered only if extensive data and justification can be presented to support their proper design and long-term performance.

The following management practices in Chapter 2 may be used to achieve this objective:

- Water Quality Ponds — Page 2-27
- Infiltration Trench — Page 2-41
- Sand Filter — Page 2-49
- Grass Filter — Page 2-63
- Bioretention Area — Page 2-69

6. Control erosion and sediment impacts during construction

Construction and associated earthmoving activities cause high sediment loads in construction site runoff. Planning for these controls begins during site layout, with overall sediment and erosion control strategies developed during the final phases of project design. While implementation of construction-phase controls is left to the contractor, they must be guided by the strategies specified by the site designer in construction plans.

CHAPTER 2

Post Construction Stormwater Management Practices

Post-construction stormwater management practices treat runoff from a development site *after* construction is complete. Their objectives range from capturing and treating pollutants in runoff to managing the increased frequency, volume and energy of stormwater runoff so that water resources are not degraded.

Historically, stormwater ponds were used to reduce downstream flooding. Today post-construction stormwater ponds add pollution control and stream protection as important design elements. Apply the structural practices found in this chapter to reduce pollutants, meet state and local permits and reduce downstream erosive effects of runoff. While all structural practices require maintenance, those provided here emphasize lower maintenance and generally self-sustaining processes. Other structural practices are available for use; yet all should be examined for their effectiveness, maintenance requirements and ability to function if maintenance is delayed.

Treatment occurs primarily through the processes of settling, adsorption, and biological uptake, while detention is utilized to curb the impact of increased runoff. Where soils are appropriate, infiltration provides substantial hydrologic benefits.

Structural practices treat runoff, but more is needed to effectively prevent and minimize impacts. Therefore additional management practices are strongly encouraged. Practices such as stream setbacks or reduction of impervious areas influence the layout and design of a development site so that important hydrologic areas are maintained and runoff is limited. Many of the management practices provided have more exhaustive reference sources given that should be consulted as they are applied. Note that while each of the management practices is beneficial, some community zoning or building standards may limit your ability to use a particular practice.

MANAGEMENT PRACTICES

STRUCTURAL PRACTICES

Page updated 5-4-12

Table 2.6.1 Pond types and appropriate characteristics and treatment goals

Pond Type	Minimum Drainage Area (acres) *	Drainage Area: Surface Area:	Suspended Solids Estimated Effectiveness	Dissolved Pollutants Estimated Effectiveness	Stream Warming Potential	Target Depth (apply % to surface area)
Extended Detention	≥10		Low to Moderate	Low	Moderate	3’
Extended Detention with Forebays and Micropool	≥10		Low to Moderate (improve over ED)	Low	Moderate	3’
Wet Extended Detention	≥20 or sufficient baseflow to support permanent pool	<6:1	Moderate-high	Moderate-High	High	Generally not deeper than 6-8’
Wet Extended Detention with wetland fringe	≥20 or sufficient baseflow to support permanent pool	<6:1	Moderate-high	Moderate-High	High	Generally not deeper than 6-8’ – 20% at 6-8”
Wetland	≥20 or sufficient baseflow to support permanent pool	>50:1	Moderate-high	Moderate-High	Moderate	5-20% 1.5-6’ wetland areas range from 0 – 18’ with avg of 6 – 12”
Wetland (pocket)	Dependent upon baseflow	>100:1	Variable	Variable	Moderate	5-20% 1.5 – 6’ Wetland areas range from 0 – 18” with avg of 6 –12”

*Note: Extended detention basins are appropriate for areas less than 10 acres, if the outlet is designed to prevent clogging.

Advantages of Wetland Features – Wetland vegetation, in addition to promoting settling, stabilizes deposited sediment. Wetlands can further treat stormwater in ways most other treatment practices cannot, by plant uptake, adsorption, physical filtration, microbial decomposition and shading. Wetland plants readily absorb heavy metals, and other toxic wastes.

Microorganisms that thrive in wetland plant root systems consume and decompose pollutants. These microorganisms that live among the plants are very good at breaking down poisonous organic compounds such as benzene, toluene and PCBs into harmless elements that the microorganisms and plants can digest.

Mosquito Concerns – Water quality ponds have extended detention times less than the time needed for common vector mosquitoes to hatch (generally 72 hours). But it is still important to design and maintain stormwater ponds in order to prevent conditions most favorable to mosquitoes. When designing and maintaining stormwater ponds apply the following considerations:

- Avoid stagnant water by assuring there is sufficient flow to support a wet or wetland ponds.
- Maintain the outlets so that detention does not occur beyond the extended detention period.
- Design wet ponds with wetland benches and wetlands with varying depths (mix of deeper water and wetland areas) in order to have improved habitats for natural mosquito predators like small fish, birds, dragonflies and aquatic insects.
- For areas that will have standing water without wave action or deeper water, consider aeration to prevent stagnation.

Design Criteria (Applicable to Each Pond Type)

Water Quality Volume (Applicable to all pond configurations)

The water quality volume (WQv) is the volume of runoff that is treated in a water quality pond. Depending upon the type of pond (dry extended detention, wet or wetland) all or a portion of this volume is stored above wetland or permanent pool features and drained over a 24-48 hour period. Detaining this volume has two stream protection objectives: reducing the pollutants suspended in the runoff and reducing the energy of common storm events responsible for most channel erosion. The water quality volume is calculated using equation 1 below, adapted from Urban Runoff Quality Management (ASCE/WEF, 1998). This is required by the Ohio EPA NPDES general permit for construction activities.

$$WQv \text{ (ac-ft)} = C * 0.75 * A / 12 \qquad \text{(Equation 1)}$$

Where:

C = runoff coefficient

A = area draining into the BMP in acres

The runoff coefficient, C, is calculated using the following equation or alternatively values provided in the current Ohio EPA NPDES general permit for construction activities.

$$C = 0.858i^3 - 0.78i^2 + 0.774i + 0.04 \qquad \text{(Equation 2)}$$

Where:

i = watershed imperviousness ratio, the percent imperviousness divided by 100

Note: The Ohio EPA NPDES stormwater general permit for construction activities requires that the water quality volume be increased by 20% for capacity lost over time due to sediment accumulation.

Pond Configuration

Configure the pond so that water quality treatment is optimized through pond shape and flow length. Improved settling of pollutants occurs as the flow length is maximized. Optimally, designs will avoid the problems of dead storage or incoming water short-circuiting through the pond and the resuspension of deposited sediments.

Forebays and micropools, pool water at the inlets and outlet of a pond in order to improve the effectiveness and ease of maintenance of water quality ponds. The shape and grade of pond side slopes also strongly influence pond effectiveness and potential safety.

7. Additional Specifications for Pond Construction

Embankment ponds must be well constructed and built according to NRCS Conservation Practice Standards 378 (Pond) addressing issues such as:

- Ponds must incorporate emergency spillways designed to safely convey flows exceeding design storm flows.
- Outlet structures should be built to withstand floatation and incorporate anti-vortex and debris or trash rack devices.
- Embankments and principal spillway shall utilize adequate soils and compaction, core trenches and anti-seep collars.

Dry Extended Detention Basin – Design Criteria

Detention Volumes

The extended detention volume is equal to the water quality volume (WQv) found in equation 1. An additional capacity of 20% must be provided within the water quality volume for sediment accumulation. This additional volume may be utilized in forebays at inlets and in a micropool at the outlet, which will improve the maintenance and efficiency of the pond.

Local government may require additional detention volumes for peak discharge control (flood control). Appropriate design procedures, including routing design storms through the basin, shall be used to insure the pond and outlet geometry meet local and state requirements. See the figure below.

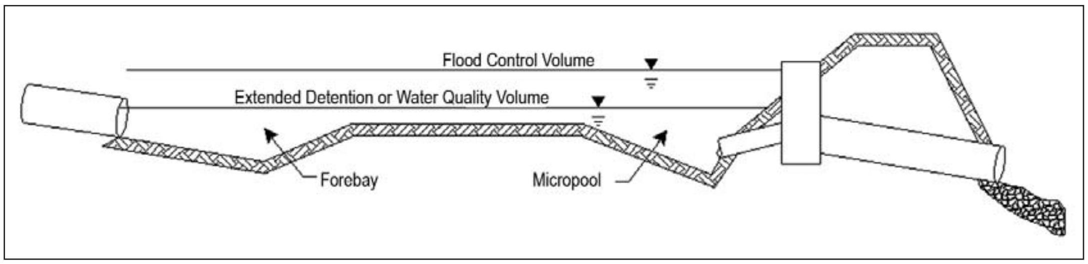

Fig 2.6.4 **Storm Water Pond with Extended Detention and Flood Control Volumes**

Outlet Design

Design the outlet structure (principal spillway) to draw down the extended detention volume over a 48-hour period. The outlet should empty less than 50% of this volume in the first 16 hours.

Peak discharge control (flood control) required by local government can be incorporated into the spillway with additional control devices (e.g. orifices or weirs) above the extended detention outlet. This type of multiple outlet spillway incorporates outlet controls for each attenuation goal.

Permanent Pool

Dry extended detention basins do not have a permanent pool except for the establishment of forebays at inlets and a micropool at the outlet. While these are not required, they increase the effectiveness of the pond and the ease of maintenance. More information is provided on these in the design criteria applicable to all ponds above.

Wet Extended Detention Basin – Design Criteria

Detention Volumes

Wet extended detention ponds detain a volume equal to 75% of the WQv found with equation 1 (0.75 WQv) above a permanent pool.   See figure 2.6.5.

Local government may require additional detention volumes for peak discharge control (flood control). Appropriate design procedures, including routing design storms through the basin, shall be used to insure the pond and outlet geometry meet local and state requirements. See the figure below.

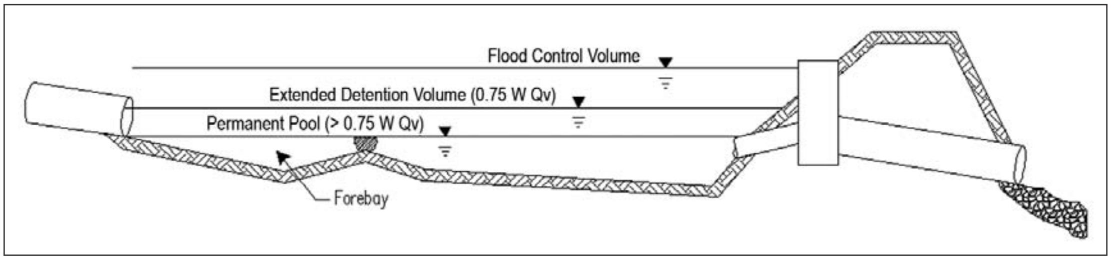

Figure 2.6.5   **Wet Storm Water Pond with Extended Detention and Flood Control Volumes**

Outlet Design

Design the outlet structure (principal spillway) to draw down the extended detention volume over a 24-hour period. The outlet should empty less than 50% of this volume in the first 8 hours.

Peak discharge control (flood control) required by local government can be incorporated into the spillway with additional control devices (e.g. orifices or weirs) above the extended detention outlet. This type of multiple outlet spillway incorporates outlet controls for each attenuation goal.

Permanent Pool Volume

The permanent pool of a wet extended detention pond is equal to three fourths of the WQv (0.75 WQv) found with equation 1 plus an additional volume equal to 20% of the WQv (0.2 WQv) added for sediment accumulation. Thus the original capacity of the permanent pool shall be equal to 0.95 of the water quality volume. This volume may include forebays, cells created within the permanent pool for increasing efficiency.

Permanent Pool Depth

The mean depth of the permanent pool should be between 3 and 6 feet in order to optimize settling of suspended particles. This is calculated by dividing the permanent pool's storage volume by the pool's surface area. A pool that varies in depth will allow diverse conditions for wetland vegetation and portions, which are deep enough for fish. If fish are to be maintained in the pool, approximately 25% of the pool should be at least 6 to 8 feet deep.

Overly shallow pools will have increased problems with algae and the re-suspension of deposited sediments by wind or as runoff enters the pond. Overly deep pools may encourage thermal stratification and anaerobic conditions at the bottom, which allow pollutants (e.g. metals and phosphorus) to be released from sediments. Deep pools are often associated with short flow paths from inlet to outlet, allowing runoff to short-circuit treatment provided by flow through the main volume of the pool.

Wetland Benches

Wet extended detention ponds may include wetland environments that greatly enhance water quality treatment by establishing a shallow aquatic bench around the main pool. These areas also improve safety by creating a vegetative barrier to discourage children from venturing into deeper water and reducing the hazard of steep grades at the pond edge.

When used as one water quality design feature within a wet extended detention pond pond, wetland vegetation should occupy at least 20% of the wet pool's water surface.   It is also recommended that benches be at least 6 feet wide and have depths of 6 to 12 inches on average and not exceed 18 inches. See the Design criteria for wetland extended detention ponds for guidance on establishing wetland plants.

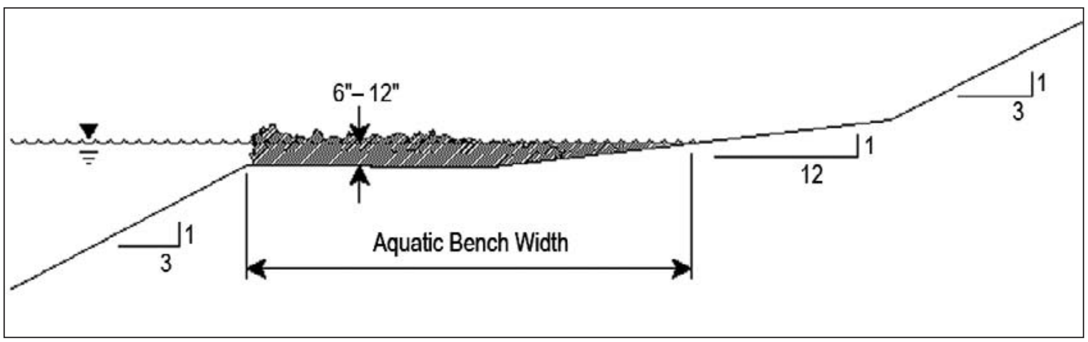

Figure 2.6.6 **Grading of Side Slope to form a Wetland Bench**

Reducing Thermal Impacts Through Shading

Warm water released from a permanent pool may adversely impact the thermal regime of receiving streams, particularly if the receiving water is a cold-water fishery. The pool acts as a heat sink between storm events during the summer months. Water released downstream from the pond can be as much as 10 F warmer than naturally occurring base flow. Large impervious surfaces also warm surface runoff significantly which can be critical to stream systems where fish and other aquatic life are threatened by high summertime water temperatures.

Add Shading – Shading a pond can significantly reduce thermal impacts. Trees planted around the pond, particularly on the south and west sides offer the most protection from the summer sun. Trees planted on islands or peninsulas should also be considered. Because tree roots can damage dams, trees must not be planted on the embankment itself. Wetland vegetation also contributes to shading and reduces thermal impacts.

Leaf litter introduces nutrients to the pond and adds to the accumulation of sediment. While nutrients and sediment are pollutants, nutrients in plant material or detritus are more readily utilized by aquatic insects and incorporated into the food chain. Fallen leaves are a vital part of aquatic environments, whereas soluble nutrients and nutrients attached to fine sediments easily wash through a pond system or promote algal growth.

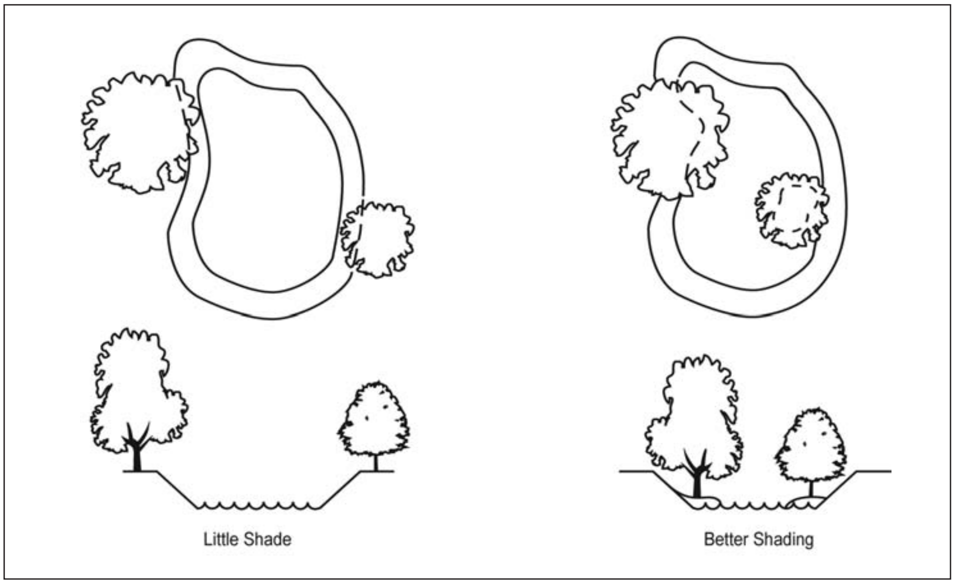

Figure 2.6.7 Tree Placement to Shade Ponds and Reduce Thermal Impacts

Wetland Extended Detention Basin – Design Criteria

Detention Volumes

Wetland extended detention ponds detain a volume equal to the water quality volume (1.0 WQv) found in equation 1 above the permanent wetland pool.

Local government may require additional detention volumes for peak discharge control (flood control). Appropriate design procedures, including routing design storms through the basin, shall be used to insure the pond and outlet geometry meet local and state requirements. See the figure below.

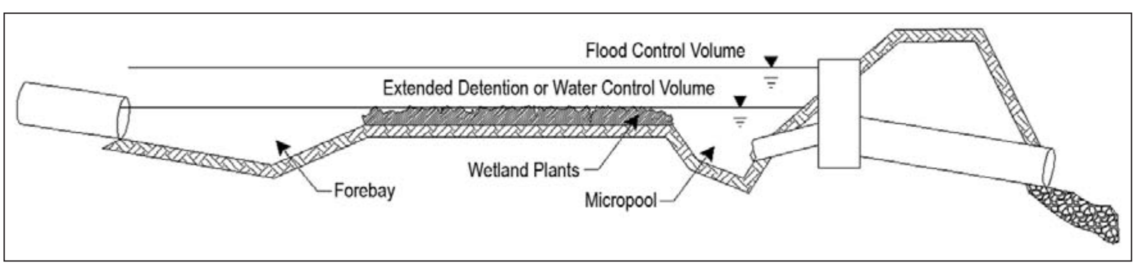

Figure 2.6.8 Wetland Storm Water Pond with Extended Detention and Flood Control Volumes

Maintenance of Water Quality Ponds

While maintenance is inevitable, the amount of maintenance required and its cost can vary considerably depending on the initial design of a pond. A number of design features are helpful in this regard:

Sediment Storage – Reduce the frequency of sediment cleanout easily by increasing the volume available for sediment storage. Increasing the permanent pool volume by 20% or according to the predicted sediment loads is recommended. Ponds used for sediment control during construction should be cleaned out when the site is stabilized, as the cost of cleanout will be considerably less expensive during construction than in the future.

On-Site Disposal – Transporting dredged sediment is often the largest cost associated with pond cleanout. This can be avoided by providing an area on-site for future sediment disposal. A disposal site should be designated during site design.

Forebay – Trapping most sediment in a confined, easily accessible forebay can reduce maintenance costs.

Maintenance Easements – Maintenance easements must be established to allow access to the pond, particularly to the forebays, embankment, outlet structure and sediment disposal areas.

Disposal of Pollutants – Water quality treatment practices are intended to trap pollutants. The fate of these pollutants must be considered. Trapped sediment is usually clean enough for on-site use. The large volume of sediment poses the most common disposal problem. Sediments may also have high concentrations of hydrocarbons, nutrients and heavy metals. Soil tests should be done if the pond has received spills, is in a highly industrial area, or if the watershed has intensive traffic.

Sediment should be spoiled in areas, which will keep pollutants bound in the sediment (e.g., metals and phosphorous). To avoid these pollutants from becoming soluble, acid and anaerobic conditions, such as wetlands, should be avoided.

Table 2.6.3 Typical Maintenance Activities For Water Quality Ponds (USEPA) Adapted from WMI, 1997 and SMRC

Schedule	Activity
Monthly	Mow embankment and clean trash and debris from outlet structure. Address any accumulation of hydrocarbons.
Annually	Inspect embankment and outlet structure for damage and proper flow. Remove woody vegetation and fix any eroding areas. Monitor sediment accumulations in forebay and main pool.
Semi-Annually	Inspect wetland areas for invasive plants.
3-7 years	Remove Sediment from forebays.
15-20 years	Monitor sediment accumulations in the main pool and clean as pond becomes eutrophic or pool volume is reduced significantly.

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Design Criteria

Diversion – Storm water runoff should be directed to the infiltration trench via dispersed sheet flow wherever possible. A grass filter strip of at least 25 feet must precede the infiltration trench in these situations. Where runoff is directed to the infiltration trench as concentrated flow (via a swale, storm sewer or other discrete conveyance), the infiltration trench must be designed "off-line" such that flows in excess of the Water Quality Volume (WQv) are diverted around the infiltration trench.

In addition, a diversion that allows the trench to be bypassed when the pretreatment system becomes clogged or otherwise fails should be included in the design. This can be accomplished by providing a drain valve.

Soil Hydraulic Conductivity – Soil infiltration rates within the trench must be between 0.52 and 2.4 inches per hour. The soil should have no greater than 20 percent clay content and less than 40 percent silt/clay content.

The list of soils in Ohio that meet the required infiltration rates and are potentially suitable for the installation of infiltration trenches can be found in Appendix E. However, do not use this or county soil surveys to determine final suitability. Site-specific soil tests should be performed to confirm that the hydraulic conductivity falls within the required range. A certified Soil Scientist or other trained professional shall perform one test hole per 5000 feet, with a minimum of two borings within the planned facility location. This evaluation shall include an evaluation of the normal and seasonal high groundwater levels.

Pretreatment – The potential for failure of infiltration practices due to clogging by sediments is high. Failure will result if sediment is not trapped before runoff enters the trench. Thus, it is imperative that the facility design includes a durable, maintainable pretreatment system for removing sediment from stormwater before the trench. This can be accomplished by installing a plunge pool. Where infiltration trenches are used to treat rooftop runoff with drainage areas of 1 acre or less, pretreatment can be accomplished by providing an underground trap with a permanent pool between the downspout and the infiltration trench (Fig 2.7.1). The trap must be accessible, but sealed tightly so that it does not become a breeding ground for mosquitoes.

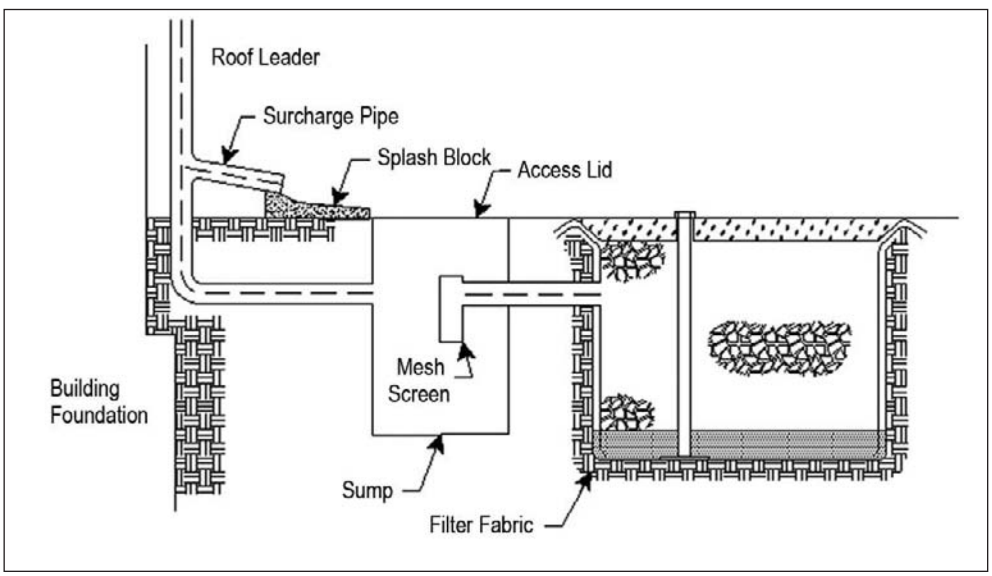

Figure 2.7.1 **Underground pretreatment facility and infiltration trench for treating rooftop runoff.**

Sizing the Pretreatment Facility – The size of the pretreatment facility is based on the infiltration rate of the soil in which the infiltration trench is built. For soils with infiltration rates of 2.0 inches per hour or less, the pretreatment facility shall be sized to contain 25% of the WQv. For infiltration rates greater than 2.0 inches per hour, the pretreatment facility shall be sized to contain 50% of the WQv.

Exit Velocity from Pretreatment Facility – The velocity of runoff as it exits from the pretreatment device must be non-erosive.

Drain Time Requirements – The practice is to be designed to infiltrate the Water Quality Volume (WQv, see page 30 of this chapt.) through the bottom floor of the structure in 24 to 48 hours. Drain times in excess of 72 hours should be avoided to prevent mosquito-breeding habitat from forming. Flows in excess of the WQv are to be diverted around the trench.

Dimensions – The dimensions of the storage reservoir (infiltration trench) are made by fitting the length, width and depth into a configuration, which satisfies drain time and storage volume requirements. The trench dimensions shall be sized by accepted engineering methods such as those outlined below:

1. Determine Initial Storage Depth – The bottom of the infiltration trench must be deeper than 2 feet to avoid freezing and shallow enough to leave at least 3 feet between the seasonal high-water table or bedrock and the trench bottom. Soil morphology also must be considered in determining the dimensions of the storage reservoir to utilize the optimum horizons or strata. The presence of a thin, slowly permeable soil horizon may require a trench depth which completely penetrates it to more permeable underlying material. Long trenches may need to be curved parallel to the topographic contour in order to keep the trench bottom elevation within the optimum depth in the soil profile.

2. Determine Area of Trench Bottom – The bottom of the trench is to be completely flat so as to allow runoff to infiltrate through the entire surface.

$$A_{min} = \frac{WQ_v}{Porosity * (E * T)}$$

Where:
- A_{min} = Minimum area of the bottom of the trench (ft^2);
- WQ_v = Water Quality Volume (ft^3); (Trench volume less stone volume).
- E = Exfiltration Rate (ft/hr); (Soil infiltration rate at trench bottom)
- T = Drain Time (hr) (Must be 24 to 48 hrs per Ohio EPA requirements)

The excavated volume of the trench is the WQ_v divided by porosity or the void space of the stone.

Determine Length and Width – A long, narrow trench is less affected by water table mounding. If depth to seasonal high-water table or bedrock is within 5 feet of the trench bottom, it is advisable to design the trench as long and narrow as possible. Otherwise, the configuration of the trench is not restricted and is only limited by site design constraints.

Stone – The infiltration trench is filled with clean, washed aggregate. Stone with a diameter of between 1 and 3 inches should be used.

Geotextile – The sides and top of the trench must be lined with a non-woven geotextile to restrict the amount of sediment entering the structure. The top layer of the geotextile should be covered by 6-to-12 inches of smaller sized gravel (0.75-inch diameter). This top layer

Infiltration trenches have a high rate of failure. In one study in Prince George's County, Maryland (Galli, 1992), less than half of the trenches investigated were still functioning properly and less than one third still functioned properly after 5 years. However, many of these structures did not incorporate pretreatment of runoff. Thus, it is critical to ensure that proper pretreatment of runoff has been provided.

Maintenance

The following regular maintenance and inspection protocol is recommended:

Table 2.7.2 Typical Maintenance for Infiltration Practice

Schedule	Activity
Twice per year	Check observation wells following 3 days of dry weather. Failure to percolate within this timeCheck observation wells following 3 days of dry weather. Failure to percolate within this time period indicates clogging. Inspect pretreatment devices and diversion structures for sediment build-up and structural damage.
Standard maintenance	Remove sediment and oil/grease from pretreatment devices as well as overflow structure.
Upon failure	Total rehabilitation of the trench should be conducted to maintain storage capacity within 67% of the design treatment volume and 72-hour exfiltration rate limit. Trench walls should be excavated to expose clean soil.
Annually	Trim adjacent trees to assure that drip-line does not extend over the surface of the infiltration trench.

Adapted from WMI, 1997 and SMRC

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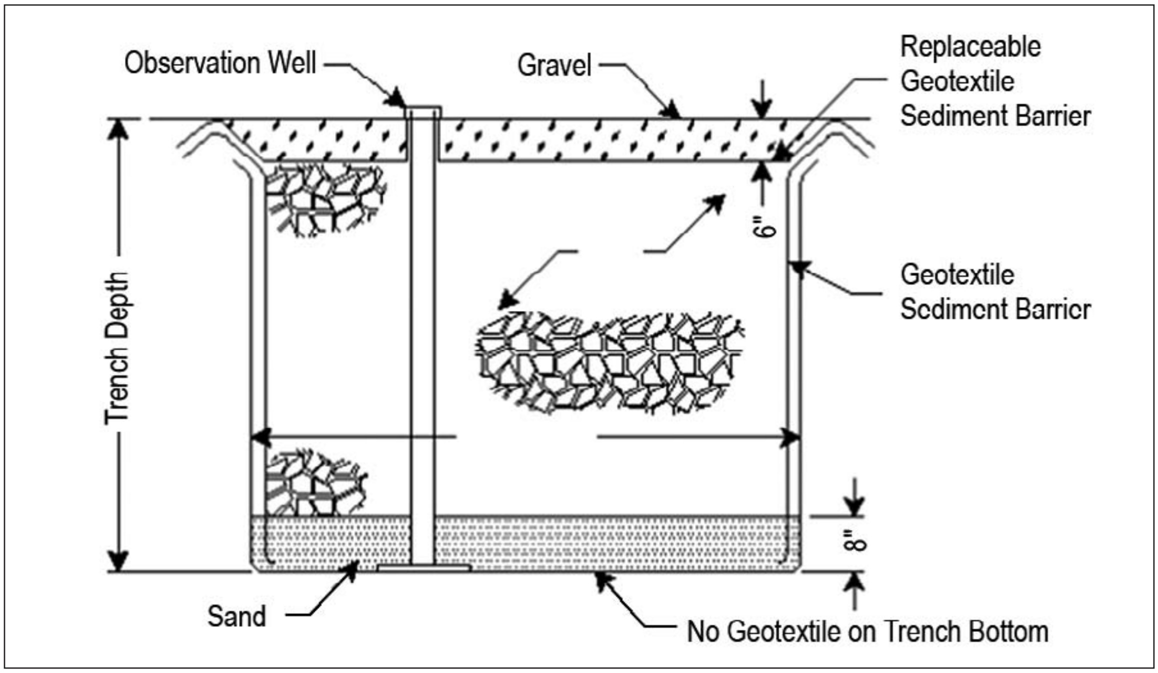
Specifications
for

Infiltration Trench



1. SEDIMENT SHALL BE PREVENTED FROM ENTERING THE INFILTRATION TRENCH. Sediment clogging and sealing off the permeable soil is the most common cause of infiltration trench failure. Runoff from the construction site shall NOT be allowed to flow to the trench until construction is complete and upslope areas have been stabilized. If storm drains enter the infiltration trench directly and cannot be rerouted, they shall be sealed with a masonry plug until all contributing drainage areas are stabilized.
2. The infiltration trench design shall include a system for removing sediment from stormwater before it enters the infiltration structure. However, this system shall NOT be used to control sediment during construction.
3. Trench excavation and backfilling of sand and rock shall be done when the soil moisture is low enough to allow the soil to crack or fracture. No trench excavation or fill shall occur on wet soil to prevent compaction and maintain soil permeability.
4. Bottom Sand Filter - The bottom of the trench shall be covered with an 8-inch layer of clean sand. The sand layer shall be placed the same day excavation is completed.
5. Observation Well - A 4-inch diameter, rigid perforated vertical pipe shall be installed in the trench. The vertical pipe shall be securely and permanently attached to a base to prevent upward movement. The top of the vertical pipe shall have a secure removable cap. The original depth shall be permanently marked on the top of the observation well.
6. Geotextile - The sides and top of the trench shall be lined with geotextile. The bottom of the trench shall NOT be covered with geotextile.
7. Rock - Rock fill shall be clean, poorly-graded, uniform size crushed washed rock. Well-graded rock has less void space available for runoff storage and shall not be accepted.
8. Gravel Top Layer - The top layer of the geotextile shall be covered by 6 inches of gravel (0.75-inch diameter).

Sand filters are usually constructed inside a concrete shell or built directly into the terrain over an impermeable liner. Where possible, the filter bed should be constructed below the frost line to prevent freezing. Although most Austin Sand Filters are open, they have been installed underground in parking areas, along the perimeter of parking lots, and in medians or landscaped areas.

1. Determine the Treatment Volume (Water Quality Volume)

The water quality volume (WQv) is the volume of runoff that is treated by a sand filter system. The sand filter should be designed to capture and store the entire WQv within the sedimentation chamber with a weir, perforated riser, or other outlet structure used to gradually release the captured runoff into the filtration basin over a 24-hour period. The filtration basin is designed to provide a filtration time of no less than 24 hours (when the filter media is new) and no more than 40 hours (when the filter media is clogged and requires maintenance). A total drawdown time of 40 hours is used for facility design. The water quality volume is calculated using equation 1 below, adapted from Urban Runoff Quality Management (ASCE/WEF, 1998). This is required by the Ohio EPA NPDES general permit for construction activities.

$$WQv \text{ (ac-ft)} = C * 0.75 * A / 12 \qquad \text{(Equation 1)}$$

Where:

C = runoff coefficient

A = area draining into the BMP in acres

The runoff coefficient, C, is calculated using the following equation or alternatively values provided in the current Ohio EPA NPDES general permit for construction activities.

$$C = 0.858i^3 - 0.78i^2 + 0.774i + 0.04 \qquad \text{(Equation 2)}$$

Where:

i = watershed imperviousness ratio, the percent imperviousness divided by 100

Note: The Ohio EPA NPDES stormwater general permit for construction activities requires that the water quality volume be increased by 20% for capacity lost over time due to sediment accumulation.

2. Divert Flows Exceeding Treatment Volume

In most cases flows into the sand filter are limited to the water quality volume (WQv). Therefore other measures may be necessary to meet flood control detention requirements either (1) by diverting all runoff exceeding the water quality volume to separate facilities or (2) by increasing the size of the sedimentation basin and placing a second outlet sized to meet flood control requirements above the stage of the water quality volume. Figure 2.8.2 shows a device that utilizes a weir to divert the water quality volume to a sand filter.

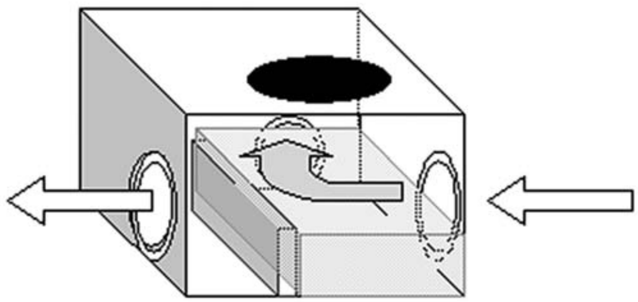


Figure 2.8.2 A weir inside the junction box of the storm sewer system diverts initial flows to sand filter.

3. Designing the Sedimentation Chamber (Basin)

The sedimentation chamber is the first stage of treatment within a sand filter. The chamber provides pretreatment of runoff by settling out coarser particles from runoff in order to prevent clogging and to reduce regular maintenance of the sand filter.

a) The Austin Sand Filter - Sedimentation chamber

The sedimentation chamber within an Austin Sand Filter is designed to completely empty between storms. This requires a somewhat larger size in order to minimize re-suspension of settled material, but also minimizes potential mosquito breeding conditions that exist within Delaware Perimeter Sand Filters and other designs that retain water between storms.

Basin Dimensions – The volume of the sedimentation basin equals the WQv plus an additional 20% of the WQv for sediment storage. The water depth in the sedimentation basin when full should be at least 2 feet and no greater than 10 feet. The minimum surface area of the sedimentation basin is determined by using the equation:

$$As = (1.2 * WQv)/(ds + freeboard) \quad \text{(Equation 3)}$$

Where:

As = Minimum surface area of sedimentation chamber (cubic feet)
WQv = Water Quality Volume (cubic feet)
ds = Basin depth (feet)
freeboard = 0.5 feet

The sedimentation chamber should be configured so that it has a minimum length-to-width ratio of 2:1 between inlet(s) and the outlet, otherwise baffles may be necessary within the sedimentation chamber. A fixed vertical sediment depth marker should be installed in the sedimentation basin to indicate when 20% of the basin volume has been lost because of sediment accumulation.

Delaware Sand Filter

(Not to Scale)

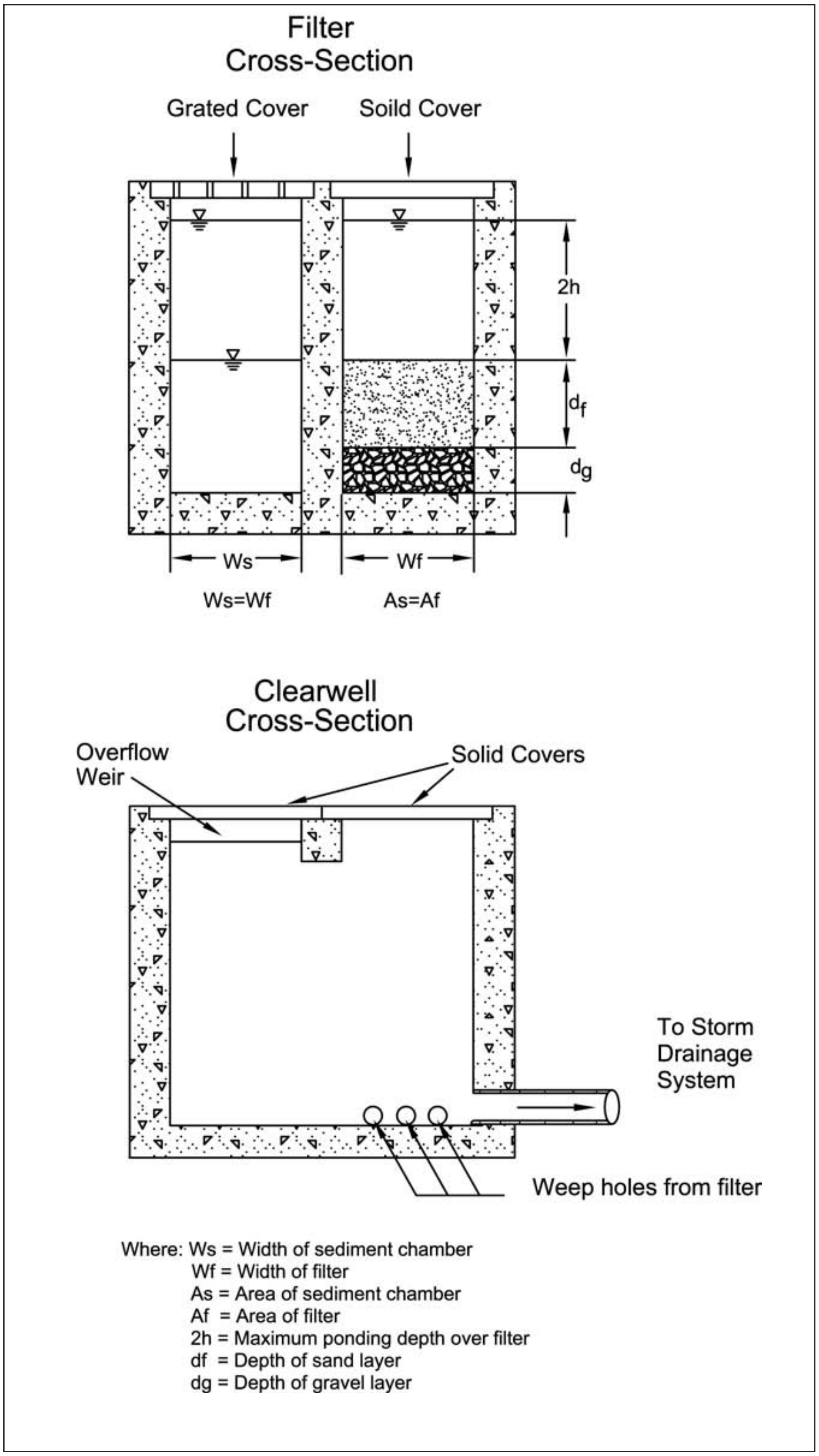


Figure 2.8.5 Delaware filter cross sections.

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Suitable soils for infiltration or with a designed underdrain system – The Bioretention practice must be designed so that the runoff storage capacity will be drained between 40 and 72 hours either through infiltration into the existing soils under the facility, through an underdrain system, or through a combination of in-situ soils and an underdrain system. Facilities designed without an underdrain system shall have a qualified professional certify that in-situ soils are appropriate for infiltration. This certification shall include a description of the soil depth and horizons that correspond to the design elevations of the bioretention practice.

Setbacks to Prevent Water Damage - Appropriate setbacks from property lines, wells, septic systems, basements and building foundations shall be maintained to prevent damage to these systems or offsite areas. The following table provides recommended minimum setbacks.

Table 2.10.1 Minimum setbacks from important infrastructure.

	Setback Distance	
Feature protected by setback	Bioretention Utilizing Underdrain System	Bioretention Utilizing Suitable Soils For Infiltration
Property Line	2' (check with local requirements)	
Well	100'	
Septic System (including perimeter drain)	50'	
Building Foundations or Basements	10'	25'
Asphalt (parking,drives or roadways)	2' (check with local requirements)	

Long Term Maintenance and Easements - Since bioretention combines plant materials with the temporary storage and filtering of stormwater, more frequent regular maintenance is required than other more traditional stormwater facilities. Designs shall include easy access for maintenance as well as an appropriate plan of operations and maintenance that considers the spectrum of activities described later in this standard. A legal and enforceable maintenance agreement shall be in place and executed. Although many bioretention facilities are located on private property and will be privately maintained, the area shall be placed in a drainage easement to permit public access if maintenance should be necessary. For residential developments, additional measures such as education materials and deed restrictions may be necessary to prevent alterations that would affect the use or diminish the effectiveness of the bioretention practice.

Surface Area - Bioretention facilities may require more land then some other water treatment practices. The surface area of the cell will generally be between 5 and 10 percent of the contributing drainage area.

Construct Bioretention after Site Stabilization - Bioretention facilities shall be constructed after all other areas of the drainage area have been constructed and finally stabilized. That is, sediment-laden runoff from construction shall not be allowed to pass through the practice. Runoff from actively eroding sites will cause the premature failure of bioretention facilities.

Design Criteria

Detention - All bioretention practices shall be designed to treat the water quality volume (WQv, see page 30 of this chapter) by initially ponding that volume and allowing it to infiltrate through a soil medium within the practice. Ohio EPA requires that runoff treated with a bioretention practice have a minimum drawdown time of 40 hours. While detention practices begin discharging soon after water begins to pond, each practice shall regulate the release of water such that no more than one-half of the water quality volume is released in less than one-third of the drawdown period (40 hours).

In the case of bioretention practices, drawdown time may be controlled by the planting soil medium, the rate of infiltration into in-situ soils under the practice (no underdrain provided) or the capacity of the underdrain system.

Design of bioretention practices may vary depending on the water quality objectives, the potential for groundwater recharge, and the potential for groundwater pollution. While all bioretention practices provide filtration through the soil medium, the following conditions and design variations may enhance or limit the infiltration of water into in-situ soils or enhance denitrification at the bottom of the practice.

Low infiltration in-situ soils – For sites having in-situ soils with low permeability, a standard underdrain bedded in a gravel layer provides the primary drainage for the practice (Figure 2.10.2a).

Limited infiltration/enhanced nitrogen treatment - This design provides a deeper gravel layer below the underdrain pipe to act as a fluctuating anaerobic/aerobic zone (Figure 2.10.2b). This encourages denitrification, the conversion of nitrate to nitrogen gas, thus aiding in preventing eutrophication of receiving waters. This design is expected to provide better treatment of runoff from areas with high nutrient loadings, particularly nitrogen.

Impermeable liner – For areas with a high water table, shallow bedrock or high pollution loads, an impervious liner separates the entire practice from in-situ soils (Figure 2.10.2c). This design also relies on the underdrain system as the primary drainage. It is appropriate where heavy pollution is expected and/or where groundwater must be protected.

In-situ soils suitable for infiltration – Where in-situ soils can infiltrate a majority of the water quality volume within 72 hours and where groundwater pollution potential is low, soil drainage may be used as the primary drainage for the bioretention practice. Although this situation may be designed without an underdrain, a larger gravel layer provided with an underdrain near the top of this layer serves as a backup to natural infiltration (Figure 2.10.2d). Systems designed without an underdrain may not be used where extensive ponding of water above the practice will cause damages. Infiltration capacity of the soils shall be tested by a qualified professional.

Underdrains shall be a perforated pipe capable of withstanding the expected load above it and exceeding the drainage capacity of the planting soil layer. The following requirements apply to underdrains:

- The underdrain system shall be placed at a minimum 0.5 % slope.
- Underdrain pipes shall be a minimum 4-in. diameter perforated pipe.
- Underdrains are placed within a layer of # 57 washed gravel, having a minimum of 3-in. of gravel above and 3-in. below the pipe.
- Underdrains shall be placed depending upon the purpose of the gravel layer:
    - o The underdrain is typically placed in the middle of the gravel layer in order to provide bedding material.
    - o To promote infiltration into in-situ soils or to create an anaerobic zone for denitrification, the underdrain is placed near the top of a gravel bed. Gravel depth is determined by water storage needed to infiltrate the entire water quality volume into the soil or the volume of water targeted for anaerobic treatment.
- Underdrain pipes shall end with a cap, or an elbow with a vertical pipe providing observation and/or cleanout at the elevated end of the pipe. Observation/cleanout pipes shall consist of a minimum 4 inch diameter vertical non-perforated PVC pipe extending to the surface of the practice and sealed with a removable watertight cap.
- Underdrains shall drain to an existing drainage system or other suitable stable outlet having positive drainage.

7. *Overflow and Routing* - Bioretention facilities shall have a non-erodible means of discharging flow exceeding the capacity of the practice. Commonly this will be an overflow pipe or drop inlet set at the maximum ponding elevation. Off-line facilities collect runoff and then are bypassed by major storm plows. Consideration for tailwater from the receiving system shall be made.

8. *Planting Materials* – Species planted in bioretention practices should be adapted to the region, pollution tolerant, and able to survive the variable moisture conditions. Most plants should be facultative (found equally in wetland or upland conditions) though some species found in either environment may be acceptable. Native and non-invasive plants shall be used.

   Select plants, which in a mature condition will be appropriate to the depth of soil and the underdrain system. For examples, trees may be selected if the planting soil can accommodate the root ball of the selected trees. Trees and large shrubs will require staking to prevent being dislodged by wind. A qualified landscape architect, botanist, or native plant dealer will be helpful to design a planting plan.

Design Checklist

1. Compute water quality volume (WQV). _____ WQV = C * P * A / 12, where:

WQv= water quality volume in acre-feet

C = runoff coefficient (Use formula below or coefficient from Ohio EPA NPDES permit)

P = 0.75 inch precipitation depth

A = area draining into the BMP in acres _____

Planned Site Imperviousness (i) _____ (Eg. For 80% imperviousness use 0.8)

$C = 0.858i^3 - 0.78i^2 + 0.774i + 0.04$

C = _____

2. Compute critical storm detention requirements. Substitute local requirement if they differ from the critical storm method.

Design Storm	Peak Discharge Rate (cfs)	24-hour Runoff Volume (show units)		Percent Increase	Design Discharge (cfs)
		Pre-Development	Post- Development		
1-year					
2-year					
5-year					
10-year					
25-year					
50-year					
100-year					

3. Determine whether bioretention is an appropriate stormwater practice for the area.

- Limited drainage area (<2 acres perhaps even less than 1 acre)
- Outlet for an underdrain and or soils of sufficient hydraulic conductivity to fully drain the practice or a suitable outlet for an underdrain system in a period of 40 to 72 hours.
- Sites with sufficient fall between inflow point to outflow (generally 5 feet). Shallower facilities are expected to reduce the effectiveness of treatment.
- Additional stormwater detention needs do not make bioretention unfeasible
- No continuous groundwater flow or seasonal high groundwater table above the practice bottom. Or perimeter drains are sufficient to lower seasonal high groundwater table.
- Low potential for groundwater pollution (high pollution loads, high groundwater table or extremely permeable soils)
- Can meet setback requirements found in Table 2.10.1

Practice Notes

Notes regarding current practices and potential changes for the Rainwater and Land Development Manual

Practice Notes is a way to inform users of the Rainwater and Land Development manual of potential changes under consideration for the manual, additional references or factors important to design.

New information will be added occasionally and dated so that readers may know when it was posted.

You are welcome to provide feedback to us by sending a note or email to John Mathews(john.mathews@dnr.state.oh.us).

ISSUES

Practice Note Regarding 2.10 Bioretention

The Issue & Recommendations

Anticipating the rate at which water moves through the soil media is critical to sizing a bioretention area. The Rainwater and Land Development manual (p. 66) advises users to use either actual values of hydraulic conductivity (k, also called the coefficient of permeability) derived from testing the soil media or the predicted hydraulic conductivity for the soil media for designing the facility. This may lead to undersized facilities with greater potential for excessive ponding as the practice ages or some of the water quality volume bypassing the practice and discharging untreated. We recommend using a k that represents the projected value for the soil media after settling and use. This value is used to calculate the area of the bioretention area and subsequently the draintime for the practice.

Using the hydraulic conductivity (k) of the soil media, as initially placed, will give a higher flow-through rate and thus a smaller practice, which may not be sustainable as the practice is used and ages. The soil media is expected to become less permeable as settling occurs, and as fine particles come into the practice through normal functioning.

The question is, what k value should be used to size the practice? The Rainwater and Land Development manual specifies a soil media expected to have a high k value. Design guides from other areas of the country suggest utilizing a k value 0.5 ft/day or 0.25 inches/hour. This value is probably nearer the point at which the soil media would need to be rejuvenated. Utilizing a k value at or near this figure will make the bioretention practice larger, and will also reduce the likelihood of failure due to excessive ponding or discharging portions of the water quality volume untreated.

To summarize:

1. We recommend that soils continue to be specified to meet the current description of soil media in Rainwater and Land Development.

2. We recommend that the k value, used for developing the bioretention area calculation and subsequent drain time, reflect the settled future condition after years of use. Utilizing a figure at or near 0.5 feet/day (0.25 inches/hour) is thought to be more representative of this condition.

2.11 Permeable Pavement

Figure 2.11.1 Pervious Concrete at Indian Run Park in Dublin, Ohio.

Description

Permeable pavement systems consist of a permeable pavement surface layer and one or more underlying aggregate layers designed to temporarily store stormwater. Most permeable pavement systems are designed to infiltrate stormwater into the underlying soil, reducing the volume of runoff leaving the site. Where the underlying soil will not permit full infiltration of runoff, outlets and/or underdrains are used to remove excess runoff and discharge it to an appropriate outlet[1].

Research has shown that permeable pavement can be a very effective component of a stormwater management system, mitigating many of the water quality and quantity impacts associated with runoff from impervious pavements. Permeable pavements reduce suspended solids, metals and petroleum hydrocarbons in runoff, and significantly reduce runoff volumes and peak flow rates.

Permeable pavements perform water quality functions by filtering suspended solids and hosting microbial organisms known to biodegrade pollutants. Depending upon the construction of the pavement, soil infiltration, transpiration (vegetated open celled grids), and increased soil adsorption may all contribute to reducing offsite runoff and associated adverse impacts. Additionally permeable pavements provide some moderating of water temperatures compared to conventional pavements.

1 Note: Permeable pavements and their drainage strucures must be considered as part of the larger site and stormwater system when meeting local peak discharge requirements.

There are a variety of permeable pavement surfaces available in the commercial marketplace, including pervious concrete, porous asphalt, permeable interlocking concrete pavers, clay pavers, concrete grid pavers, and plastic grid pavers. While the design specifics vary for each product, permeable pavements have the same general structural components detailed in this practice.

There are several examples of permeable pavement installations that are still functioning well after 15 or 20 years (see e.g., Adams, 2003). If designed, constructed, and maintained according to the following guidelines, permeable pavements should have life spans comparable to traditional impervious pavements.

Condition where practice applies and settings to avoid

Permeable pavement can be used in most settings where traditional pavements are used. It is especially well suited to parking lots, sidewalks, playgrounds and plazas. Permeable pavement can be used in driveways if the homeowner is aware of the stormwater management function and subsequent maintenance requirements of the pavement.

Areas of Heavy Traffic - Permeable pavement typically is not specified for areas that experience high traffic loads or high vehicle weight traffic such as busy roadways or travel lanes in heavily used parking lots. However, permeable pavement is suited for parking lanes on roadways and in parking lots. When it is necessary to use traditional pavement for traffic lanes, runoff can be directed as sheet flow to permeable pavement areas.

Areas of Potential Groundwater Contamination – Permeable pavements should not be used in heavy industrial developments, areas with chemical storage, fueling stations or areas with significant risk of spills that might contaminate groundwater. Permeable pavements should not be used for sites located over contaminated soils without placing an impermeable liner between the pavement structure and soils.

Other Sites to Avoid

Unstable slope areas – permeable pavement should not be used in slip prone areas where concentrated infiltration may exacerbate slope instability

Steep slopes - areas with slopes steeper than 10 percent present design challenges that are difficult to overcome

Sediment sources - sites with sources of sediment (from vehicles, bare soils, spoil piles, sand storage, etc.) should be separated from permeable pavements with filter strips or other sediment removal practices.

Anticipated Performance

Permeable pavements are projected to perform well in reducing the annual load of suspended solids, metals and hydrocarbons in runoff, and significantly reduce runoff volumes and peak flow rates. Permeable pavements filter solids in the pavement layer and may completely remove them in the matrix of the sub-pavement layers depending upon the nature of the subgrade and designed drainage of the system. Though this varies with design; filtering, detention, adsorption processes all contribute to some degree in reducing pollutants in contributed flows and offsite runoff. Permeable pavements also buffer water temperatures. Increased infiltration into the subgrade soils contributes to the highest removal of pollutants from site runoff, although some pollutants such as soluble nutrients, chlorides or sodium raise concern for groundwater pollution.

Table 2.11.1 Anticipated performance of permeable pavements.

Category	Subcategory	Full WQv Infiltration	Partial Infiltration	No Infiltration
Runoff Water Quality	Suspended Solids*	>90%	80-90%	80%
	Phosphorus*	Medium	Medium	Medium
	Nitrogen/Nitrates*	Low	Low	Low
	Heavy Metals	High	High	High
	Bacteria	Not clear at this time. Other practices using media filtration do treat bacteria. Using a sand layer may enhance this.		
	Thermal	Permeable pavements with a reservoir storing the WQv or most of that volume are expected to provide good thermal attenuation, but this will vary based on the particular design (i.e. material, the storage volume, outlet configuration etc.)		
	Oil and Grease	High	High	High
	Poly Aromatic Hydrocarbon	Comparable to conventional pavements, reduced compared to asphalts sealed with coal-tar based coatings.		
	Chlorides & Sodium**	Not controlled.		
Runoff Volume Reduction		85-90%	%WQv-captured * 85%	
Recharge		High	Medium	Not at all.
Runoff Time of Concentration		Improved lag time, but varies with design.		
Peak Flow Attenuation		Significant peak flow attenuation, but varies with design.		

* There would be an expected improvement with the addition of sand layers and/or vegetative systems.
** May be a significant groundwater concern depending upon winter application practices.

Planning Considerations

Preliminary Site Evaluation - The overall site should be evaluated for potential permeable pavement/infiltration areas early in the design process, as effective permeable pavement design requires consideration of soils, grading, outlets, groundwater, and other site infrastructure.

Size of Project – Small projects such as walkways, or driveways with limited traffic may not have associated requirements for treating or storing stormwater. Therefore small scale projects may not need the depth of stone reservoir described in this practice. There are still numerous benefits to applying permeable pavements even with less stone subbase than this practice describes. For small scale practices where local or state regulations do not require treating the water quality volume, manufacturer recommendations should be consulted.

Soils - Permeable pavements may be used on any soil type, although soil conditions determine whether an underdrain is needed. Less permeable soils (most Hydrologic Soil Group C or D soils, some HSG B soils) usually require an underdrain, whereas soils with higher permeability (HSG A, and some HSG B soils) often do not. Estimates of soil permeability are available based on soil type, but designers should verify underlying soil permeability rates before proceeding with site and stormwater system design (see discussion below). Special measures may be needed when permeable pavement will overlay high shrink-swell soils in order to limit moisture or to stabilize these soils.

Subgrade Compaction - One of the major benefits of permeable pavement is runoff volume reduction from infiltration into underlying soils. Subgrade compaction severely limits the infiltration capacity of the underlying soil. For permeable pavement systems with an infiltration component, the subgrade should not be compacted according to traditional pavements. Structural integrity of permeable pavements is ensured through several mechanisms other than subgrade compaction (see discussion below). If the structural design of the pavement section requires subgrade compaction to achieve the required design strength or to minimize the possibility of pavement failure, then soil permeability should be measured based on the required subgrade design.

Separation Distances - Permeable pavements should not be located or used where their installation would: create a significant risk for basement seepage or flooding; interfere with public or private wells, septic or sewage disposal systems; or cause problematic groundwater issues. These issues should be evaluated and potential problems avoided by the designer.

Horizontal Separation Distances

- separation from buildings - permeable pavement systems should be installed at least 10' away from up-gradient building foundations and 100' from down-gradient foundations, unless an acceptable barrier is provided or the building foundation can adequately handle additional water;
- sanitary sewers - care should be taken to minimize infiltration of runoff into sanitary sewers and building laterals;
- septic systems - permeable pavement should be installed no closer than 100' from a septic system or leach bed; when this or any infiltration BMP is located up-gradient, appropriate perimeter drainage should be used to prevent flows from reaching the septic system;
- drinking water wells - permeable pavement should not be located within 25' of a private drinking water well or within the sanitary isolation radius of a public drinking water supply well. (The isolation radius ranges from 50 to 300 feet, and is based on the well's average daily pumpage; see the chart below.) If it is necessary to pave within the sanitary isolation radius, use of an impermeable bottom liner and an underdrain discharging beyond the isolation radius is recommended, especially if the pavement will support motorized vehicles.

Feature protected by setback	Setback Distance (feet)	
Building Foundations or basements	At least 10’ downgradient or 100’ upgradient of foundations	
Septic Systems	At least 100’ separation	
Private Well	At least 25’ (See OAC 3701-28-10)	
Public Well	50 – 300 ft minimum depending upon Average Daily Water Demand (based upon sanitary isolation distance found in OAC 3745-9-04)	
	Average Daily Pumpage (Q) (gal/day)	Sanitary Isolation Radius (feet)
	0-2500	50
	2501-10,000	Square root of Q
	10,001 – 50,000	50 + Q/200
	Over 50,000	300
Source Water Protection Area	See Ohio EPA Source Water Protection Area. Each area may have its own specific requirements.	

Table 2.11.2 Horizontal separation distances.

Vertical Separation Distances - Give special consideration to the following situations:

- Infiltrating permeable pavement systems with recharge layers located over soils with ground water tables that reach within 2 feet of the subgrade infiltration bed.
- Infiltrating permeable pavement systems with recharge layers located over impermeable bedrock within 2 feet of the subgrade infiltration bed.

These situations are likely to result in mounding of stormwater to the level of the infiltration bed for extended periods, especially during the spring. These systems may still help meet watershed management goals - for example, baseflow maintenance and temperature moderation during summer low-flow periods. However, a more thorough mapping and modeling of surface and subsurface hydrology is necessary to prevent unintended consequences. The pavement system configuration and drainage system should be modified to achieve stormwater management goals while minimizing unintended consequences.

Soil surveys can be used as rough guidance during initial planning and site layout to identify areas where shallow water tables or shallow bedrock may be a concern. However, in areas where these concerns are known, a professional geotechnical engineer and/or professional soil scientist should be contracted to take core samples to a depth of 6 ft below the proposed subgrade depth and report: depth to bedrock, any layering of the subgrade representing significant changes in texture or structure, the particle size distribution of the subgrade soil, the particle size distribution of any deeper layers, and depth to water table (ideally the water table will be checked between late March to early May when the water table is highest).

Groundwater Concerns – Permeable pavement, as with any infiltrating practice, requires the designer to consider the potential for adversely impacting groundwater. Elevated pollution sources or areas with high risk of toxic spills should not be directed to permeable pavement without appropriate pretreatment.. Examples include maintenance yards where salt storage or distribution takes place, airport areas where deicing occurs, fueling stations and composting facilities.

Development sites that include both relatively clean runoff (e.g., rooftop runoff) and dirtier runoff (e.g., from a maintenance yard or material storage area) should consider separate stormwater management systems appropriate to the specific runoff source. In such a scenario, rooftop runoff or runoff from office parking could be safely directed to an infiltrating BMP without pretreatment, whereas runoff from a maintenance yard should be treated in a separate facility designed to minimize potential negative impacts to groundwater. Such areas should be separated with physical barriers (fence, curb, etc.) to minimize tracking of pollutants into "clean" runoff areas.

Karst Terrain - Active karst regions are found in parts of Ohio (Hull, 1999; ODNR, 1999), and complicate development and stormwater system design. The use of permeable pavement or other infiltration BMPs in karst regions may promote the formation of sinkholes. In karst regions, a detailed geotechnical survey should be conducted to the satisfaction of the local approval authority. Permeable pavement designs in karst should exceed the minimum vertical separations recommended above and consider the use of an impermeable bottom liner and an underdrain. Additionally they should not receive runoff from other (external) impervious areas.

Freeze-Thaw - Water entrapped in the pavement during freezing and thawing cycles will result in cracking, scaling and/or deterioration of the pavement (NRMCA, 2004). Therefore, the pavement structure and drainage system should be designed to ensure free drainage of the pavement surface and to prevent ponding into the pavement structure. Permeable pavements may be more resistant to freezing and may thaw faster than conventional impermeable pavement due to the water content and ground temperature in the underlying soil and the ability to infiltrate meltwater (Backstrom, 2000).

Frost Heave - Frost heave occurs when underground water accumulates in ice formations or ice "lenses", expanding and pushing the pavement structure upward resulting in uneven pavement (Leming et al., 2007) . Unlike their traditional counterparts, permeable pavements are specifically designed to introduce water below the pavement surface. Therefore, the pavement structure and drainage system should be appropriate for the subgrade soils (Leming et al., 2007; UNH, 2009).

One recommendation is to increase pavement or aggregate base thickness to accommodate the extra load carried by the surface course during spring thaw (Leming et al., 2007) and is reflected in some guidance for portland cement pavement surfaces (see ORMCA, 2009).

Frost heave is a serious concern for finer textured soils. Sands and coarser aggregates are much less susceptible to frost heave. One straightforward approach to minimize frost heave is to provide a base aggregate course thickness to minimize the formation of ice in the underlying subgrade. The University of New Hampshire Stormwater Center (UNH, 2009) recommends that the thickness of the permeable pavement structure (i.e., pavement plus sub-base thickness) be a minimum of 0.65 x design frost depth for the location. Local maximum frost penetration depth oftentimes can be provided by the local building authority. In the absence of locally available information, the following table can be used.

Located North of Latitude	Max. Frost Depth (inches)	Min. Recommended Thickness (0.65 x Max Frost Depth in inches)
38.3	24	16
38.7	26	17
39.0	28	18
39.3	30	20
39.7	32	21
40.0	34	22
40.3	36	24
40.7	38	25
41.0	40	26
41.3	42	27
41.7	44	29
42.0	46	30

Table 2.11.3 Frost depth and minimum recommended pavement system (pavement + sub-base) thickness by latitude (interpolated from Fig. 13 in Floyd, 1978; http://www.ngs.noaa.gov/PUBS_LIB/GeodeticBMs/#figure13)

Grading – The bottom of the infiltration bed should be level or nearly level. Sloping bed bottoms will lead to poor distribution and reduced infiltration. It is recommended permeable pavement surface slopes be less than 5% to optimize the ponding depth under the pavement surface. Where topography doesn't permit a single level infiltration bed, multiple infiltration beds may be benched or terraced to obtain the necessary infiltration area and to promote more uniform infiltration.

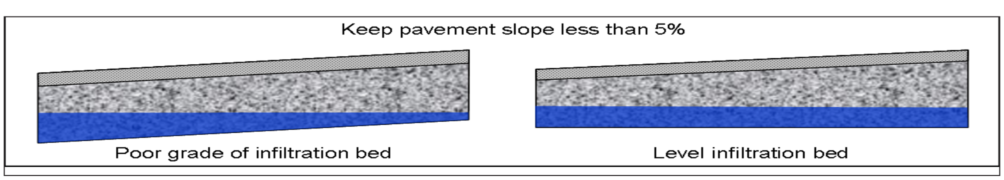


Figure 2.1.2 Level infiltration beds and limited pavement slope maximizes stormwater treatment and storage.

Figure 2.11.3 Terrace sloping areas to limit the pavement slope (photo credit: Brandon Andreson).

Runoff from External Areas - Drainage from traffic lanes or other impervious surfaces (e.g., sidewalks) can be directed to permeable pavement surface as sheet flow. The impervious area contributing runoff should be less than twice the area of permeable pavement receiving the runoff. Roof drains and leaders may connect directly to the subbase reservoir, but should be provided a means of trapping sediment prior to the subbase reservoir. Runoff from permeable areas (lawns or landscaping) or other sediment sources should not be directed onto permeable pavement.

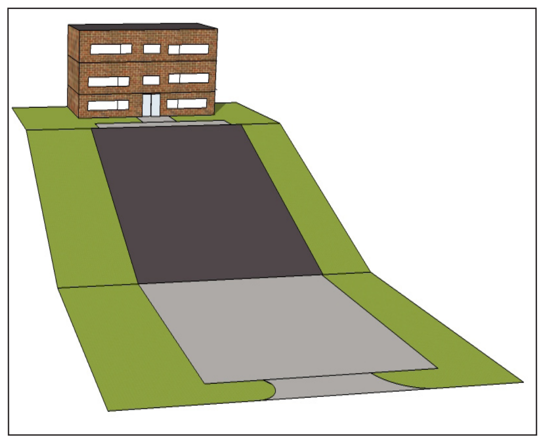


Figure 2.11.4 Calculate "run-on" from impervious areas, making sure it does not exceed twice the pervious pavement (infiltration bed) area.

Sites to Use or Consider Use of an Impermeable Liner

A impermeable liner should be used for permeable pavement systems for sites:

- all sites over contaminated (or potentially contaminated) soils
- sites with high pollution potential source areas
- sites with slip prone soils
- sites in source water protection areas

A impermeable liner may be considered for permeable pavement systems for sites with:

- subgrade soil infiltration rates less than 0.02 in/hr
- depth to bedrock or seasonal high water table less than 2 ft below subgrade infiltration bed
- karst geology

If the site requires a liner, the designer should consider whether a different BMP (e.g., bioretention, constructed wetland, wet swale) may be more appropriate.

Stormwater Detention - Sub-pavement infiltration beds are typically sized to manage the water quality volume and to convey stormwater without allowing ponding into the pavement itself. These sub-pavement aggregate "reservoirs" also may be designed to mitigate the peak discharge of less-frequent, more intense storms (such as the critical storm or 100-yr event). Discharge control typically is provided by an outlet control structure. The specific design of these structures may vary, depending on factors such as rate and storage requirements.

Construction Sequencing - The permeable pavement system is most susceptible to failure during construction, and therefore it is important that the construction be undertaken in such a way as to prevent:

• Compaction of underlying soil

• Clogging the subgrade soil or geotextile with sediment and fines

• Tracking of sediment onto pavement

• Drainage of sediment laden waters onto permeable surface or into aggregate base

Permeable pavement will be prone to failure if it is not protected from sources of sediment. Sediment on the subgrade infiltration bed will greatly reduce the infiltration capacity of the final practice. For this reason, it is ideal that nearby areas or areas contributing runoff are completely stabilized prior to construction of the permeable pavement system. Additional means of controlling sediment may be necessary if all disturbed areas can not be stabilized completely before permeable pavement construction. Leaving nearby disturbed area below grade can be helpful to prevent sediment from running onto the permeable pavement area. More effective sediment barriers and controls may be needed.

Quick succession from excavation to placement of materials during dry weather is ideal for protecting the practice's long term functioning. Planned pavement areas that will be exposed for a period of time while other site construction occurs may be excavated within twelve (12) inches, but no closer than six (6) inches, of the final subgrade elevation. Following construction and site stabilization, sediment should be removed and final grades established only when materials can be placed in a timely manner.

Maintenance - Permeable pavements have different maintenance requirements than traditional pavements, discussed in some detail below. The use of permeable pavement must be carefully considered in all areas where the pavement potentially could be seal coated or paved over due to lack of awareness by a new owner, such as individual home driveways. In those situations, a system that is not easily altered by the property owner may be more appropriate. Educational signage at permeable pavement installations may promote its prolonged use. Maintenance is critical to the long-term performance of permeable pavement, especially those activities that prevent clogging of the surface pavement and subsequent clogging of the subsurface layers by accumulated sediments and organic matter. The most important activities to protect the long term function of permeable pavement include periodic vacuum sweeping to remove accumulated sediments and organic materials, monitoring of the drainage functions of the pavement and maintenance/cleanup of landscaped areas contiguous to the parking area (CSN, 2010).

Cost Considerations - The primary added cost of a permeable pavement/infiltration system lies in the underlying aggregate bed, which is generally deeper than a conventional pavement subbase. However, this additional cost may be offset by a significant reduction in the number of inlets and pipes. Permeable pavement systems may eliminate or reduce the need (and associated costs, space, etc.) for surface detention basins. When all these factors are considered, permeable pavement with infiltration is increasingly competitive with traditional pavement for the pavement and associated stormwater management costs.

Types of Permeable Pavement

Porous Asphalt - Porous asphalt is very similar to conventional bituminous asphalt except the fines have been removed to maintain interconnected void space. Research has led to improvements in porous asphalt through the use of additives and higher-grade binders. Porous asphalt is similar in appearance to standard asphalt and is suitable for use in any climate where standard asphalt is appropriate. Guidance specific to the design, installation and maintenance of porous asphalt is available from the National Asphalt Pavement Association (NAPA, 2008), the University of New Hampshire Stormwater Center (UNHSC, 2009) and Flexible Pavements of Ohio.

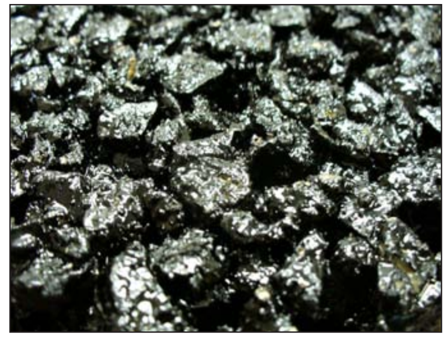


Figure 2.11.5 Porous Asphalt

Pervious Concrete - Pervious concrete is produced by reducing the fines in the mix to maintain interconnected void space for drainage. Pervious concrete has a coarser appearance than its conventional counterpart but may be colored similar to traditional decorative concrete. In northern climates such as Ohio, pervious concrete should always be underlain by a stone subbase designed for proper drainage and stormwater management, and should generally not be placed directly on a soil subbase. Special care must be taken during the placement of the pervious concrete to avoid overworking the surface and creating an impervious pavement. Guidance on the design, installation and maintenance of pervious concrete is available from the Ohio Ready Mix Concrete Association (ORMCA, 2009). ORMCA also offers installer training and certification for pervious concrete.

Figure 2.11.6 Pervious Concrete

Block or Brick Pavement - A number of concrete or clay paver products are available, providing either a traditional brick pavement look or more complex designs and configurations. Block or brick pavements maintain drainage through gaps between the pavers filled with small, uniformly-graded gravel. The pavers are bedded on a stone or sand layer that provides uniform support and drainage. Pavers are especially well suited for plazas, patios, small parking areas, parking stalls in larger lots, and streets.

Permeable interlocking concrete pavement (PICP) are one commonly used product that consist of 3 1/8” thick concrete units or pavers with various shapes, patterns, and colors. The size and complexity of the project determines whether PICP may be placed by machine or by hand. Guidance for design, installation and maintenance of concrete pavers is available from the manufacturer and the Interlocking Concrete Pavement Institute (ICPI, 1995).

Figure 2.11.7 Permeable Interlocking Concrete Pavement

Reinforced Turf and Gravel Filled Grids - Grid-type permeable pavements consist of open-celled concrete or plastic structural units filled with small, uniformly-graded gravel or turf that allows infiltration through the pavement surface. The structural units are underlain by a stone and/or sand drainage system for stormwater management. Reinforced turf applications are excellent for fire access roads, overflow parking, occasional use parking (such as at religious facilities and athletic facilities). Reinforced turf is also an excellent application to reduce the required standard pavement width of paths and driveways that must occasionally provide for emergency vehicle access.

Figure 2.11.8 Vegetatated Grid System utilitized for fire access.

Figure 2.11.9 Vegetated Grid System with established turf grass.

Design Criteria - General/Introduction

Permeable pavements typically will be designed to address two types of design criteria:

- Minimum specifications should be met to ensure the long-term structural performance appropriate to the specific use of the pavement (pavement type, location, type of traffic, traffic load, etc.). The pavement should meet all design, construction and maintenance requirements of the local approval authority.
- Secondly, permeable pavement typically will be part of the stormwater management infrastructure of the development site. Therefore, meeting specific design criteria should allow the permeable pavement system to receive credit toward meeting water quality treatment performance requirements of the NPDES Construction General Permit (OEPA, 2008) and/or receive appropriate credit toward meeting local peak discharge requirements.

Design Criteria - Stormwater Requirements

The Ohio DNR and Ohio EPA mandate is to ensure post-construction stormwater performance over the long-term. This means the permeable pavement system must show equivalent WQ performance to the structural BMPs listed in Table 2 of the NPDES Construction General Permit (Ohio EPA, 2008), or be part of a larger stormwater system that collectively meets those requirements. Permeable pavement can be used to meet the WQv requirement for either new development or re-development.

Full infiltration of WQv - Permeable pavement, without prior OEPA approval, may be used to meet the WQv requirements of the Construction General Permit (CGP) as long as the practices designed to fully infiltrate the WQv and follows the design, construction and maintenance protocols outlined in this section.

No infiltration - If the site is not suitable for deep infiltration (e.g., lined system or compacted subgrade), permeable pavement may be considered for WQv on a case-by-case basis with prior approval from OEPA and the local MS4. This scenario will require an appropriately designed outlet control to release runoff over a 24 hour period; however, no additional sediment storage volume (=0.2*WQv) is required. The volume of runoff detained shall drain over 24 hours, releasing no more than one half the volume in the first eight hours. Monitoring of system function/performance may be required.

Partial infiltration of WQv - If the site is capable of partially infiltrating the WQv, the volume infiltrated may be subtracted from the WQv when determining detention requirements. As for the no infiltration scenario, an appropriately designed outlet will be needed to release runoff over 24 hours, releasing no more than one half the volume in the first eight hours.. This scenario requires prior approval from OEPA and the local MS4.

Redevelopment Projects - For redevelopment projects, the area of permeable pavement receives a 1:1 credit toward the 20% reduction in impervious area requirement of the CGP. All areas draining to the permeable pavement receive credit toward the impervious area reduction as long as the storage layer is designed to hold and either infiltrate (within 48 hours) or release (with a drain time of 24 hours, releasing no more than half the WQv in the first 8 hours) the water quality volume AND the permeable pavement system meets all other requirements outlined in this guidance.

Inspection and Maintenance - Permeable pavement must be inspected and cleaned regularly to maintain the hydrologic performance of the pavement system. Therefore, Ohio EPA will consider permeable pavement as meeting the requirements of the CGP only if the property owner has a maintenance agreement approved by the local MS4 that includes the minimum practices outlined under the section titled “Maintenance” below.

Water Quality Calculations -

Calculate the **total water quality volume** (WQv) using the following equation:

$$WQv\ (ac\text{-}ft) = C*P*A \qquad \text{(Equation 1)}$$

Where: C = volumetric runoff coefficient
P = 0.75” rainfall
A = drainage area (acres)
For the permeable pavement surface, C = 0.89.

For other contributing drainage area, determine C according to guidance in the NPDES Construction General Permit (Ohio EPA, 2008). Either look up the C value in Table 1 of the CGP, or use the following equation:

$$C = 0.858i^3 - 0.78i^2 + 0.774i + 0.04 \qquad \text{(Equation 2)}$$

Where: i = watershed imperviousness ratio, the percent imperviousness divided by 100

If the additional contributing drainage area is entirely impervious surfaces (traditional pavements and/or roofs), i = 1 and C = 0.89.

No additional storage is required for sediment accumulation.

Converting Storage Volume to Storage Depth - The sub-pavement volume available for temporary storage of stormwater will typically be filled with aggregate (washed, uniformly-graded stone or gravel). The volume occupied by the aggregate itself is unavailable for water storage. The remaining volume of voids is available for storage of water:

$$V_T = V_S + V_V \qquad \text{(Equation 3)}$$

Where: V_T = Total Volume
V_S = Solids Volume
V_V = Voids Volume

A more common way to communicate about the volume available for water storage is the aggregate porosity, φ, the ratio of void-space volume to the total volume:

$$\varphi_{aggregate} = V_V / V_T$$

Aggregate porosity can range from 0.30 to 0.40 (Ferguson, 2005). However, some percentage of the voids will be unavailable for additional stormwater storage because of previous wetting and entrapped air resulting in a lower usable or effective porosity. We recommend using an aggregate porosity of $\varphi_{aggregate} = 0.35$ in the following calculations[2,3].

The aggregate thickness required to meet the WQv objective can be calculated:

$$D_{agg\text{-}WQv} = WQv/(A_{reservoir} * \varphi_{aggregate})$$

Where: $D_{agg\text{-}WQv}$ = required aggregate thickness (L)
WQv = water quality volume (L^3)
$A_{reservoir}$ = basal area of aggregate reservoir (L^2)
$\varphi_{aggregate}$ = aggregate porosity

2 Note that the porosity of the pavement itself typically is substantially lower than the aggregate base; when needed for calculations, porosities for the pavement should be taken from guidance provided by the specific industry association.

3 A number of underground storage chambers have been developed and designed to provide both sturctural support for pavements and temporary stormwater storage. Because the void space within the chambers approaches 100%, these chambers may provide a cost-effective alternative to a sub-pavement reservoir consisting entirely of aggregate. Guidance for both the chambers and the industry association for the desired pavement should be consulted to ensure sturctural performance.

Drawdown Calculation - Ideally, the water quality volume will be drained within 48 hours in preparation for the next runoff event. The approach to determine drawdown characteristics is different depending on whether the permeable pavement is an infiltrating or non-infiltrating system.

The entire area under both permeable (e.g., parking lanes or pull-in parking) and conventional pavement (e.g., traffic lanes) may be used as infiltration or storage area as long as the WQv/sub-base gravel layer is fully interconnected and the soil infiltration capacity is adequate throughout the area. A minimum of 33% of the infiltration bed should be covered with permeable pavement.

For non-infiltrating systems, the drawdown calculation should follow the procedure used for surface detention basins with the depth and head adjusted for the porosity of the aggregate. For WQv detention under permeable pavement, a 24 hour drawdown time is recommended, with no more than ½ of the water quality volume draining from the facility in the first 8 hours. The drawdown control device should have a minimum orifice diameter of 1".

For infiltrating systems, the WQv should be infiltrated into the subgrade soil within 48 hours. The design infiltration rate of the subgrade soil will be based on field measurements at the appropriate depth, and be verified during construction (see section on measurement and verification of subgrade infiltration rate). The infiltration rate shall be based on the final, after-compaction subgrade properties, if compaction is required[4].

There are a number of factors - including soil compaction, surface smearing, aggregate "masking", sedimentation, and air entrapment - that typically mean the actual infiltration rate under real-world, post-construction conditions will be substantially lower than the measured infiltration rate. To increase the likelihood of achieving design performance over the long-term, it is recommended that an infiltration rate equal to one-half the measured infiltration rate of the subgrade be used for the design:

$$f_{design} = 0.5 * f_{measured}$$

Where: f_{design} = design subgrade infiltraion rate (L/T)

$f_{measured}$ = field measured subgrade infiltration rate (L/T)

The following table presents estimates of design infiltration rate that can be used for initial planning considerations until field measurements can be collected[5].

Soil Texture of Subgrade Soil	Clay Content (%)	Clay + Silt Content (%)	Preliminary f_{design} (in/hr)	Soil Texture of Subgrade Soil	Clay Content (%)	Clay + Silt Content (%)	Preliminary f_{design} (in/hr)
Sand	< 8	< 15	3.0	Sandy Clay Loam	20 - 35	<55	0.05
Loamy Sand	< 15	< 30	2.0	Clay Loam	27 - 40	54 - 80	0.02
Sandy Loam	< 20	< 60	0.9	Silty Clay Loam	27 - 40	>80	0.02
Loam[5]	7 - 27	48 - 80	0.2	Silty Clay	40 - 60	>80	0.02
Silt Loam[5]	< 27	48 - 100	0.1	Sandy Clay	35 -55	<55	<0.01
Silt[5]	<12	80 - 92	0.1	Clay	> 40	>55	<0.01

Table 2.11.4 Estimated infiltration rate based on soil texture.

4 If the subgrade will be compacted to meet structural design requirements of the pavement section, the design infiltration rate of the subgrade soil shall be based on measurement of the infiltration rate of the subgrade soil subjected to the compaction requirements.

5 For silt, silt loam and loam subgrade textures, check for the presence of a fragipan, which can severely limit permeability.

For infiltrating systems, the drawdown calculation shall be determined using the following equation. The infiltration area A_{inf} shall be the bottom area of the infiltration bed.

$$T_d = WQv / (f)(A_{inf})(\varphi_{aggregate})$$

Where

T_d = drawdown time (T)

WQv = water quality volume (L^3)

f = infiltration rate of subgrade soil (L/T)

A_{inf} = area of infiltration bed (L^2)

$\varphi_{aggregate}$ = porosity of aggregate base

WQv Sample Problem

A site in Columbus proposes to install 1 acre of permeable pavement that will also receive sheet flow from 2 acres of traditional asphalt. The subgrade infiltration area is equal to the area of the permeable pavement. The measured subsurface infiltration rate (fmeasured) of the native soil is 0.5 in/hr. The aggregate base is composed of No. 57 aggregate. Calculate the WQv, the depth of the WQv, the porosity adjusted WQv depth, and the time necessary for the WQv to drain into the native soil.

Calculate the WQv:

WQv = C*P*A

i = 100% impervious = 1.0

C = 0.89

P = 0.75 inches

A = 3 acres

WQv = (0.89)(0.75 in)(3 ac) = 2.0 ac-in = 0.17 ac-ft = 7300 ft^3

Calculate the WQv "depth":

$D_{WQv} = WQv/A_{inf}$ = 2.0 ac-in/1.0 ac = 2.0 inches

Calculate the porosity adjusted WQv depth:

$\varphi_{aggregate} = 0.35$

$D_{agg\text{-}WQv} = WQv /(A_{inf})(\varphi_{aggregate}) = D_{WQv} /(\varphi_{aggregate})$ = 2.0 in/0.35 = 5.7 inches

Calculate the WQv drain time:

$f_{design} = 0.5\ f_{measured}$ = 0.5 (0.5 in/hr) = 0.25 in/hr

$T_d = WQv / (A_{inf})(f_{design})$ = 2.0 ac-in/(1.0 ac * 0.25 in/hr) = 8 hr

T_d = 8 hr < 48 hr

Water Quantity (incl. Peak Discharge) Credits - The peak rate of runoff from a site is radically altered by development. The hardening of pervious areas, and the improved hydraulic efficiency of the drainage network contribute to increased flow peaks, as well as extended periods of higher discharge. Permeable pavements considerably reduce flow peaks, when compared with traditional pavements, through several mechanisms including subgrade infiltration (also called exfiltration), temporary storage and increased flow path resistance.

Permeable pavement can be encouraged by appropriately crediting the storm-water management benefits provided. The ways that permeable pavement potentially can receive credit include:

- infiltration or extended detention of the WQv (described above)
- stormwater utility credit or fee reduction
- critical storm adjustment
- peak discharge attenuation

The ways that permeable pavement may be used to fulfill the WQv requirement are discussed in the previous section. The other three quantity "credits" are discussed here.

Stormwater Utility Credit - [Note: All credits are at the discretion of the local stormwater management authority.] All contributing drainage area for which the permeable pavement system fully infiltrates the WQv should receive full credit for runoff volume reduction and water quality purposes, and partial to full credit for peak flow reduction. Permeable pavement systems with partial or no infiltration should be considered for a partial credit because of the combination of water quality benefits, runoff volume reduction, and flow peak reduction.

Critical Storm Adjustment - The State of Ohio does not regulate stormwater discharges for large, infrequent rainfall events (e.g., 1-year to 100-year events). However, controlling discharge for these events is an important consideration toward protecting public safety and minimizing damage to property and infrastructure. Many Ohio communities have peak discharge or "flood control" regulations aimed at reducing the impacts of large events. Many of those communities have adopted the Critical Storm criteria for peak discharge control (ODNR, 1980). The following recommendations are designed to encourage consideration of permeable pavement while still protecting the public interest.

For permeable pavement systems, the CN for Critical Storm determination should be based on the abstraction potential, which is a function of infiltration capacity of the underlying soil and the elevation at which underdrains are placed above subgrade. Until more definitive research is developed by NRCS or another research entity, it is recommended that the Critical Storm CN for the permeable pavement system be based on TR-55 guidance (USDA, 1986) for "newly graded areas" or "open space in poor condition" based on the hydrologic soil group (HSG) of the in-situ soil and the measured subgrade infiltration rate upon completion of excavation of the underground reservoir.

Soil HSG (in/hr)	Measured Infiltration Rate	CN
A	> 1.0	68
B	> 0.2	79
C	> 0.05	86
D	> 0.02	89

Table 2.11.5 Recommended Critical Storm CN for A_{inf} for No Underdrains or Underdrains Placed D_{agg}-WQv or Higher above Subgrade.

Soil HSG (in/hr)	CN
A	77
B	86
C	91
D	94

Table 2.11.6 Recommended Critical Storm CN for A_{inf} for Underdrains Placed Directly on Subgrade.

Modeling Stormwater Detention and Peak Discharge Attenuation - The aggregate subbase "reservoir" can be used as a detention basin to temporarily store stormwater. Outfitted with an appropriate outlet, the aggregate reservoir may be able to meet local peak discharge requirements for the area that drains to the permeable pavement system. Otherwise, the aggregate reservoir and outlet become part of the overall drainage network that needs to be properly "routed" to determine inflow to an end-of-pipe facility.

The following guidelines will help ensure the permeable pavement system achieves long-term structural and stormwater management goals:

- Peak discharge requirements are set by local regulations. All stormwater systems that incorporate permeable pavement require review and approval from the local stormwater authority. Preliminary approach, plans and calculations should be discussed as early as possible with the plan reviewer to facilitate communication and avoid delays in review and approval.
- The available storage volume is equal to area*depth*effective porosity of the aggregate layer(s).
- though porosities for washed, uniformly-graded aggregate may approach 0.4, some percentage of the voids will be unavailable for storage because of previous wetting and entrapped air resulting in a lower usable or effective porosity; for consideration of intense design events such as a NRCS type II distribution, use of a conservative effective porosity of 0.35 for clean, uniformly-graded aggregate is merited.
- the porosity of the pavement course typically will be substantially lower than the aggregate base; when needed for calculations/routing, porosities for the pavement should be taken from guidance provided by the specific industry association.
- For infiltrating systems, the modeler should assign a steady discharge (often termed exfiltration rate) equal to the final (or minimum) infiltration rate.
- The aggregate reservoir should be designed to prevent the (routed) 10-yr, 24-hr design event from rising to the elevation of the bottom of the pavement course.
- The site design should include a secondary, surface drainage network that will pass the 100-yr, 24 hr event without damage to property assuming failure of the permeable pavement system. The model should show flow paths and elevations for the 100-yr, 24-hr design event with the permeable pavement treated as impervious.

Subgrade Infiltration Capacity - The hydrologic performance of infiltrating permeable pavement systems requires special attention to the subgrade soil (i.e., soil at the bottom of the aggregate reservoir) and the infiltration bed surface throughout planning, design and construction. The following guidelines will help ensure the permeable pavement system achieves long-term stormwater management goals:

- The bottom surface area of the infiltration bed should not be less than the surface area of the permeable pavement. The designer should consider increasing the infiltration bed surface area by extending the infiltration bed under adjacent traditional pavement. Such an expansion of the infiltration bed may be necessary to achieve the required drawdown time for the WQv.
- The bottom surface area of the infiltration bed should be at least 33% of the sum of the area of the permeable pavement surface plus all contributing impervious surfaces (parking lot, roads, driveways, sidewalks, roofs, etc.), that is $A_{inf} > 0.33*(A_{perm\text{-}pave} + A_{impervious})$.

- The bottom of the infiltration bed should be level or nearly level. Sloping bed bottoms will lead to poor distribution and reduced infiltration.
- For infiltrating systems, the subgrade should not be compacted as it would be for traditional pavements. If the structural design of the pavement section requires subgrade compaction to achieve a required design strength, then subgrade infiltration should be measured based on the required subgrade design.
- The design infiltration rate of the subgrade soil should be based on field measurements at the appropriate depth and verified during construction (see section on measurement and verification of subgrade infiltration rate).

Design Criteria - Pavement Structure Design

Structural Design – The designer shall refer to the appropriate industry association or manufacturer's specifications for structural design of the permeable pavement system.

Table 2.11.7 Reference appropriate specifications for structural design.

Pavement Type	Guidance	Website
Porous Asphalt	Porous Asphalt Pavements for Stormwater Management: Design, Construction and Maintenance Guide. Info Series 131, Revised November, 2008. National Asphalt Pavement Association, Lanham, MD.	www.asphaltpavement.org www.flexiblepavements.org
Pervious Concrete	Specifier's guide for Pervious Concrete Pavement with Detention. Revised October, 2009. Ohio Ready Mixed Concrete Association, Columbus, OH.	http://www.ohioconcrete.org
Concrete Pavers	Structural Design of Interlocking Concrete Pavement for Roads and Parking Lots. ICPI Tech Spec Number 8. Interlocking Concrete Pavement Institute, Washington, DC.	http://www.icpi.org
Grid Pavements	Concrete Grid Pavements. ICPI Tech Spec Number 8. Interlocking Concrete Pavement Institute, Washington, DC.	http://www.icpi.org
Vegetated Grid Pavements	See various manufacturer specifications	

Infiltrating Systems:

Pavement & bedding material - see industry association guidance.

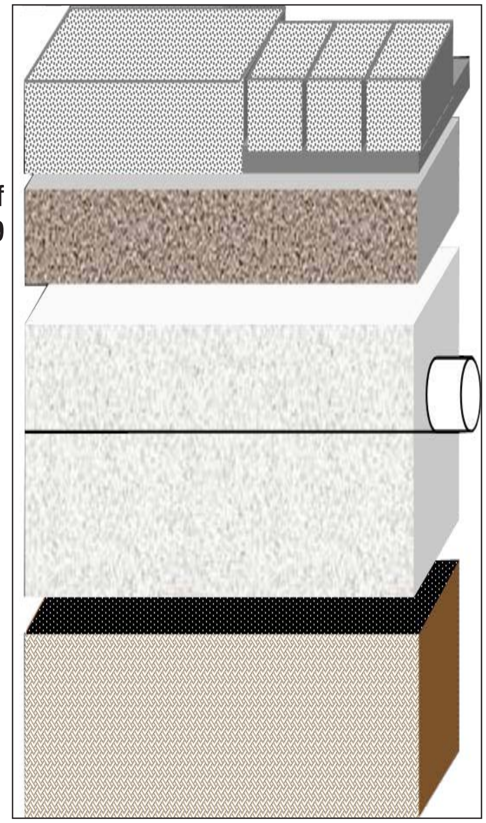
Filter or Stabilizing (choker) course - minimum 2” of AASHTO #57 if larger aggregate is used for reservoir course or AASHTO #7, #8 or #9 if the reservoir course uses #57 and the paving material requires it for stability. Use open graded crushed stone for maximum stability.

Underdrains - 4”-6” dia. PVC placed at top of recharge layer.

Recharge course - sized to infiltrate the WQv from the contributing drainage area (minimum 3” depth). Typically AASHTO #57 or larger clean, uniformly-graded coarse crushed aggregate.

Permeable geotextile fabric or sand layer equivalent

Subgrade - uncompacted subgrade



Closed Systems:

Pavement & bedding material - see industry association guidance.

Filter or Stabilizing (choker) course - minimum 2” of AASHTO #57 if larger aggregate is used for reservoir course or AASHTO #7, #8 or #9 if the reservoir course uses #57 and the paving material requires it for stability. Use open graded crushed stone for maximum stability.

Reservoir course - clean, uniformly-graded coarse crushed aggregate, typically #57, #4, #3 or #2.

Underdrains - 4”-6” dia. PVC placed placed on subgrade.

Impermeable liner (if necessary)

Compacted subgrade graded with positive slope toward the outlet

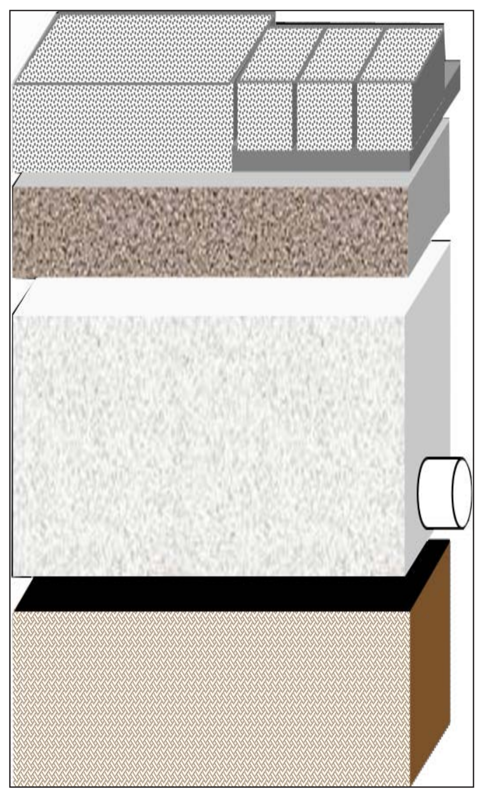


Figure 2.11.10 Types of materials used in infiltrating and closed permeable pavement systems.

Subgrade Preparation - The subgrade shall be designed to carry the desired traffic load. Check the appropriate industry association or manufacturer's specifications for compaction requirements. Design infiltration rates must be adjusted to account for intended and unintended subgrade compaction.

Subgrade Soil/Aggregate Base Interface - For open (infiltrating) systems on fine-textured soils a geotextile should be placed between the native soil and the aggregate base[6]. The geotextile limits the migration of fines, limits the settling of aggregate into the underlying soil, and helps to distribute surface loads.

For infiltrating systems, given the soil characteristics of the native soil, alternative materials such as a layer of clean sand may be placed in lieu of a geotextile on top of the native soil layer to provide adequate separation between the native soil and aggregate base in an open system (UNHSC. 2009).

For closed systems, an impermeable liner shall be placed between the native soil and the aggregate base using standard measures to prevent puncture of the geomembrane (e.g., smooth subgrade, sand bedding, geotextile). Prevent lateral flow by bringing the impermeable liner to the surface or by securing the liner to a cut-off or perimeter wall making sure that the outlet pipe and any other penetrations of the liner are adequately sealed. An impermeable liner should be used for permeable pavement systems for:

- all sites over contaminated (or potentially contaminated) soils
- sites with high pollution potential source areas
- sites with slip prone or shrink swell soils
- sites in source water protection areas

A closed system may also be used to prevent saturation of the underlying soil for structural reasons; consult a geotechnical engineering or pavement design engineer to determine whether a closed system is required based on soil conditions.

Perimeter Barrier: Some paving materials will be prone to lateral movement unless secured against a perimeter barrier. This may be a cut stone or concrete barrier or a manufactured edge restraint. Concrete barriers at the surface grade or as a raised curb can also serve as a way to secure the impermeable liner in non-infiltrating systems to prevent lateral flow between cells in a sloping situation. Where open graded subbase material will be placed against conventional road base material or soils, some type of barrier may be needed to prevent migration of fines into the permeable pavement subbase and movement of water into the conventional road base.

Aggregate Bed - The underlying aggregate bed is typically 8-36 inches deep and is a function of structural requirements, stormwater storage requirements, frost depth considerations, site grading, and anticipated loading. Several sizes of aggregate may be required for pavement bedding, stabilizing courses, or stormwater storage. It is critical the aggregate be uniformly graded, clean washed, and contain a significant void content. Pavements subject to movement will need fractured or crushed stone to mazimize stability of the base. A range of aggregate sizes has been used successfully in permeable pavement projects. Choice of aggregate(s) will depend on structural requirements, local availability, and cost. Check the appropriate industry association or manufacturer's specifications for specific aggregate requirements.

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6 UNHSC, 2009. UNHSC Design Specifications for Porous Asphalt Pavement and Infiltration Beds. Revised October, 2009. University of New Hampshire Stormwater Center, Durham, NH. http://www.unh.edu/erg/cstev/pubs_specs_info.htm.

Underdrains and observation well - Most permeable pavement systems should be designed with an underdrain system to efficiently drain the system during larger events. To avoid damage to the pavement layer, water within the subsurface stone storage bed should only rise to the level of the pavement surface in extremely rare events based on the risk tolerance of the engineer, owner or MS4 (we recommend a minimum of the 10-yr, 24-hr event). Underdrains should be installed with positive drainage and capped at dead ends of drains. For permeable pavement areas of at least 10,000 square feet, at least one observation/cleanout standpipe should be installed near the center of the pavement and shall consist of rigid 4 to 6 inch non-perforated PVC pipe. This should be capped flush with or just below the top of pavement elevation and fitted with a screw or flange type cover. Portions of the underdrain system within 1 foot of the outlet structure should be solid and not perforated.

Figure 2.11.11 Commonly used stone for stabilizing and reservoir layers (not to scale).

Additional Manufacturer or Industry Recommendations: There may be industry or manufacturer specific recommendations or requirements that will be unique to the particular paving material. These should be followed without undermining the water quality functioning of this practice. For instance, some porous asphalt guidance recommends the use of additional drainage such as catch basins with surface inlets or perimeter aggregate drains that can capture surface runoff and direct the storm water to the reservoir course. Designers must consider how sediments will be kept out of the aggregate base in this particular instance.

Construction

Any non-traditional stormwater practice presents challenges during the construction phase that require extra attention to plan detail (both for the design engineer and the contractor) and benefit from construction oversight by the design engineer or others with intimate knowledge of system design and function. Infiltrating permeable pavement systems increase complexity by striving to maintain infiltration capacity while ensuring structural integrity. For these systems, the design engineer should provide additional detail or requirements that protect or assure design infiltration capacity, and this capacity should be confirmed with field measurements during construction.

Acceptable Conditions for Initiating Construction - Construction of the permeable pavement shall begin only after all the contributing drainage area has been stabilized with vegetation and the planned cover or suitable sediment barriers placed in order to prevent contamination with sediments. Do not construct the permeable pavement practice in heavy rain or snow. Check industry guidance for suitable temperatures for construction. Construction of any infiltration BMP should be completed during a window of dry weather - excess compaction or smearing of the subgrade will ensure failure of the stormwater functions of the practice and threaten non-compliance with local or state requirements.

Erosion, Sediment and Runoff Controls - Keeping sediment out of this practice is critical. Rigorous installation and maintenance of erosion, sediment and runoff control measures should be provided to divert runoff and to prevent sediment deposition on the pavement surface, the subgrade or within the stone bed. A non-woven geotextile may be folded over the edge of the pavement to reduce the likelihood of sediment deposition. Any construction materials that are contaminated by sediments must be removed and replaced with clean materials (CSN, 2010). Surface sediment should be removed as soon as possible using a vacuum sweeper.

Clearing and Excavation - Clear and excavate the area for pavement and base courses in a manner that maintains the infiltrative capacity to the greatest extent possible (Brown, 2010). First insure plans detail staging of work in order to maintain the infiltrative capacity of the subgrade soils. Compaction of the subgrade soils will be increased by working in wet conditions, allowing construction equipment to work or travel across the area and by smearing the final soil surfaces during excavation. Final grade of the bed should be level for infiltrating systems, while closed or lined systems should have positive drainage to the outlet. To protect and maintain subgrade infiltrative capacity (adapted from Brown, 2010):

- Do not all allow excavation in wet conditions or if wet weather is forecasted for the construction period or before the area can be filled. Excavate in dry soil moisture conditions and avoiding excavating immediately after storms without a sufficient drying period.
- Do not allow equipment or haul routes to cross the planned pavement area, especially once excavation has begun.
- Station and operate excavating equipment from outside the planned pavement area or from unexcavated portions of the area using an excavation staging plan (see figure 2.11.12).
- Leaving 6 to 12 inches of undisturbed soil above the sub grade elevation if geotextile and base material placement will be delayed.

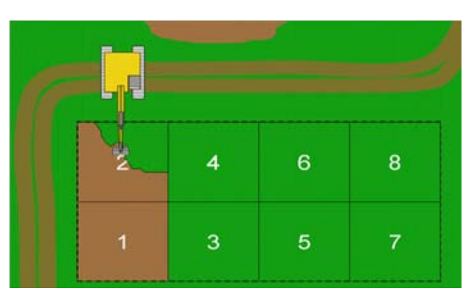

Figure 2.11.11 Stage excavation so that it can be done without compacting the subgrade.

- Dig the final 9-12 inches by using the teeth of the excavator bucket to loosen soil so as not to smear the sub grade soil surface. Avoid grading the bottom (subgrade) surface of the practice with construction equipment. Final grading or smoothing of the bottom should be done by hand or suitable tracked equipment with low ground pressure.
- Avoid allowing water to pond in bottom of cuts.
- Areas that have been allowed to trap sediment must have sediment removed and be relatively dry before final excavation down to the subgrade elevation. Any accumulation of sediments on the finished subgrade should be removed with light equipment and the subgrade surface lightly scarified with hand tools. *Very important note: limit breaking natural soil structure (especially for clayey-silty soils) or risk adversely impacting the infiltrative capacity of the subgrade.
- Finally, before placing geotextile and base aggregate, the final subgrade infiltration rate must be measured for infiltrating systems and reported to the local stormwater authority.

Place geotextile or planned filter material on the uncompacted subgrade and place geotextile up and over the sides of the excavated area. Place geotextiles so that there is a minimum of 16 inches of overlap between subsequent rolls of fabric (see manufacturers recommendation) and a minimum of four feet of material beyond the sides of the excavation. Secure geotextile so that it will not move or wrinkle as aggregate is placed. Some designers may use an alternative filter material such as sand and/or pea gravel between the base aggregate (reservoir layer) and the subgrade soils instead of geotextile (see e.g., UNHSC, 2009). Non-infiltrating designs may compact the subgrade and replace the geotextile with a suitable impermeable lining. Excess fabric (beyond the excavation) should not be trimmed until there is no possibility of sediment entering the pavement area.

Place reservoir course of aggregate and underdrain system. For infiltrating systems, plans will dictate the depth of aggregate to be placed beneath the underdrain system, although this generally exceeds 3 inches. Dead ends of pipe underdrains shall be closed with a suitable cap placed over the end and held firmly in place. For permeable pavement areas of at least 10,000 square feet, at least one observation/cleanout standpipe shall be installed near the center of the pavement and shall consist of rigid 4 to 6 inch non-perforated pvc pipe. This should be capped flush with or below the top of pavement elevation and fitted with a screw or flange type cover. Portions of the underdrain system within 1 foot of the outlet structure should be solid and not perforated.

Spread 4-12 inch lifts of the washed stone aggregate comprising the reservoir layer.   Place and spread lifts of stone without driving on the subgrade and being careful not to damage drainpipes, connections or observation wells. Place at least 4 inches of additional aggregate above the underdrain. The aggregate layer should be lightly compacted, although industry references vary on the degree and number of passes with a roller. The Interlocking Concrete Pavement Institute (ICPI, 2007; LID,) specifies making 2 passes with a roller in vibratory mode and at least 2 passes in static mode until there is no movement of the stone, while the National Asphalt Pavement Association recommends compacting each lift with a light roller or vibratory plate compactor. Do not crush the aggregate with the roller.

Install filter/stabilization layer (and bedding layer if used). This course transitions from a larger aggregate size of the subbase to a size that will fill large voids and provide a smooth surface for the pavement layer. Its use depends upon the size of the aggregate course below. For pervious concrete and porous asphalt, AASHTO No. 57 may be used for the reservoir layer and in the layer transitioning to pavement. For interlocking pavers, a smaller size aggregate will be used as a filter layer and also as a bedding layer. These layers should be spread, leveled and compacted to their designed thicknesses.

Install paving materials. Install the planned paving materials in accordance with manufacturer or industry specifications for the particular type of pavement, whether pervious concrete, porous asphalt (Hansen, 2008; Jackson, 2007), interlocking pavers or grid pavers.

Maintenance

Permeable pavements require maintenance to provide stormwater benefits over a long time period. Because permeable pavements convey water through the pavement and also effectively trap fine materials, the majority of maintenance efforts will be to keep the system permeable (unclogged) and to manage pollutants such as salts that might effect groundwater. Therefore regular inspection will evaluate whether the surface and the bed of the pavement are functioning as intended. In other words, water should continue to move through the pavement, not pond into the pavement layer, and drain from the reservoir layer in sufficient time. Maintenance of the pavement will remove fine materials as they collect in the surface and prevent winter deicing materials from being overused or clogging the system.

Effective management includes educating the property owner, landscapers, maintenance staff, snow removal personnel and general users. In this regard, an operation and maintenance plan, signage, maintenance agreements, and contracts will serve as important points of reference for these audiences. Each document should reflect the appropriate actions to take and those to avoid for the appropriate audience. For example, landscaping personnel that work adjacent to the pavement area should be required to keep landscaping materials, such as soil, mulch or plants off the pavement and to use adequate sediment control and/or stabilization for bare areas. Snow removal, pavement repair and similar contracts should include notes regarding appropriate and inappropriate actions regarding the permeable pavement area. Because permeable pavements will be maintained and managed differently than traditional pavements, signage at permeable pavement installations is recommended. This will promote its prolonged use and prevent conventional pavement management from damaging the system. An example of this includes preventing seal coating of porous asphalt or allowing snow to be stockpiled on a permeable pavement.

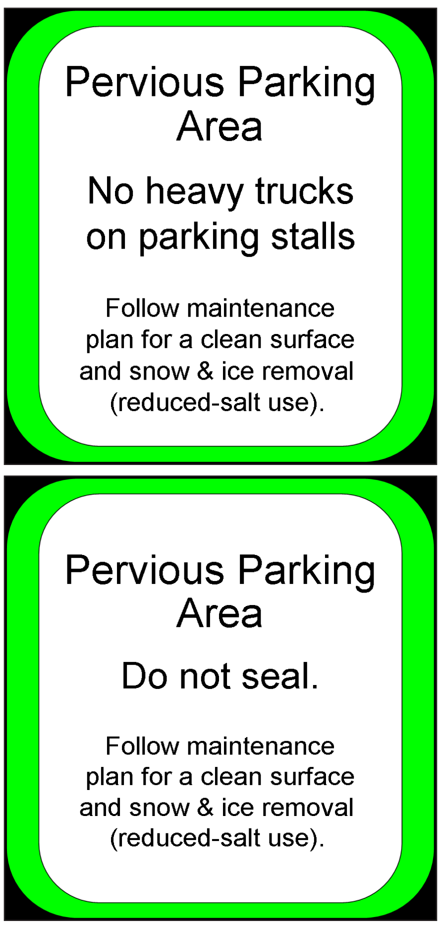

Figure 2.11.13 Examples of signage that might be used to protect permeable pavements.

An operation and maintenance plan should be prepared by the designer and provided to the owner and the stormwater authority as well as the property manager and maintenance personnel. An operation and maintenance plan for permeable pavement should detail specific actions that must be performed and their timing and/or frequency. It also describes potential damaging actions and measures to take to prevent damage to the permeable pavement. The operation and maintenance plan should also provide detailed information regarding the observation well and the depth or elevations of the underdrain system and outlet, so that the water levels under the pavement can be monitored and compared to the designed function of the system. The operation and maintenance plan should provide the normal drain time (hours) of the pavement as tested following construction (ASTM 1701).

Three main strategies dominate permeable pavement operation and maintenance:

Prevent clogging of the pavement and regularly remove accumulated fines. Vacuum sweeping is necessary to remove grit, leaves and other debris collecting at the pavement surface. This should be done two to four times a year. Times that especially will have an accumulation of material include after winter snow melt and after leaf drop in the fall. Vacuums used on paver systems with bedding material should be able to remove sediments and organic matter without removing the bedding aggregate. If bedding aggregate is removed, it should be replaced. Preventing clogging also involves managing adjacent vegetated and landscaped areas. These area should be maintained in healthy vegetation. Soil, mulch and other landscaping materials should never be stored or stockpiled directly on the pavement. Construction equipment should not be driven over or stored on the pavement.

Snow and Ice Removal. Sand or cinders is not recommended for use on permeable pavements with some exceptions[7]. Instead winter maintenance should focus on timely snow plowing and judicious use of deicing materials. Deicing materials present a problem in any pavement system due to their solubility and history of building up to levels that are toxic to plant and animal life. In permeable pavements, high salt use has an increased potential of reaching groundwater sources, but case studies of permeable pavements have shown a reduced need for deicing material to be applied to permeable pavements due to the effects of a warmer subbase. The operation and maintenance plan should provide guidelines for reduced salt use responsive to the actual ice on the pavement rather than typical rates applied on conventional pavements in the Midwest. Snow should not be stockpiled on the pavement. The operation and maintenance plan should show where snow will be pushed or stockpiled during plowing. The operation and maintenance plan should detail the blade depth that plow operators should use, because in some instances, such as grid pavements, snow plow operators may need to raise the blade slightly to avoid dislodging the surface. In every case, care should be taken with snow plowing to keep from gouging the pavement or dislodging aggregate or pavers.

Repair permeable pavements appropriately. Areas may be repaired using the same treatment as the original permeable pavement application or, in the case of porous asphalt or pervious concrete, small areas (not the lowest area on a sloping section) can be replaced with standard (impermeable) pavement. In that case the stone bed of the entire pavement will continue to provide storage and infiltration as designed. In no case should seal coats or new impermeable pavement layers be applied, as is typical in traditional asphalt pavements.

Inspection Items. The following are suggested items for inspection and are adapted from CSN, 2010:

- Using the observation well, observe the rate of drawdown in the practice. Measure the water level in the observation well following a storm event exceeding one half inch of rainfall. This should be done immediately after the storm, recording the precipitation amount, the time of the measurement and the water level in the well. Observe and record the water level after 24, 48 and 72 hours. Actual expected performance will depend on

7 No salt use is recommended during the first season on pervious concrete and some sand may be utilized being careful not to clog the pavement. Stockpiling snow on pervious concrete is not consider by industry representatives to be a problem.

the soils and the intended performance of the design. If the subgrade soils were hydrologic soil group D, there may still be water standing in the reservoir layer after 48 or 72 hours. There should not be standing water above the elevation of the underdrain, and this would indicate problems with the outlet or underdrain system being clogged. Assess potential clogging of the subgrade soils and geotextile by comparing the actual drawdown rate to the intended or design performance of the reservoir layer.

- Observe the pavement surface during and after rain for evidence of ponding, deposited sediments, leaves or debris. Address any signs of clogging or accumulated fine material by performing vacuum maintenance.
- Inspect the structural integrity of the pavement surface for damage such as missing infill material or broken pavers, spalling, rutting, or slumping of the surface. Any adversely affected areas should be repaired as soon as possible.
- Check contributing impervious areas and their associated pretreatment or runoff control structures for sediment buildup and structural damage. Remove sediment as needed.
- Inspect adjacent and contributing drainage area for sources of sediment or areas that may need better stabilization with erosion control.

Typical Maintenance Activities	Anticipated Schedule
Avoid sealing with construction sediments	During construction & long-term
Water vegetated grid pavement areas and adjacent vegetated areas to ensure good growth	As necessary during first growing season
Avoid sealing or repaving with non-porous materials	Long-term
Clean pavement to ensure pavement is free of debris and sediments	As needed (at least twice a year)
Check to see that pavement dewaters during large storms and does not pond into surface (check observation well for appropriate water levels)	After large storms
Inspect upland and adjacent vegetated areas. Seed & straw bare areas.	As needed
Inspect pavement surface for structural integrity and areas in need of repair. Repair as needed.	Annually

Table 2.11.8: Typical maintenance activities for permeable pavement (adapted from WMI, 1997)

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CHAPTER 4

Permanent Runoff Control

Permanent runoff controls convey water in a manner that is stable and doesn't contribute sediment and additional pollutants to runoff. As land development occurs and portions of the natural drainage patterns are changed to manage runoff, permanent runoff control practices such as diversions, level spreaders, grassed swales and outlet protection are used to protect against erosion, safely route flows, dissipate high energy flows and in some cases provide water quality benefits. Generally permanent runoff controls must safely convey the community's prescribed design storm or at least a 10-year frequency storm.

Lands previously used for farming often have subsurface drainage systems, which must continue to serve upland areas. Construction in these areas is bound to uncover drainage systems and disrupting these can cause severe drainage problems to adjoining land. So every effort should be made to locate drainage lines prior to construction and to reroute them during construction. These systems should not be used as stormwater runoff outlets, since they were not designed to function as storm drains. Adding surface water will likely exceed the capacity of the tile system, cause it to fail and subsequently cause adjoining land to suffer drainage problems.

4.1 Grassed Swale

Description

Grassed swales are constructed channels shaped and established with suitable vegetation in order to convey stormwater runoff without allowing channel erosion.

Condition Where Practice Applies

This practice is applicable where added capacity and protection by vegetation are needed to control erosion from concentrated runoff, to improve drainage, or to convey stormwater.

This practice applies generally to small channels having flow only during storm events.

This practice is not applicable in larger ephemeral streams where grass cannot be established and maintained. Chapter 3 Stream Channel Rehabilitation or further channel restoration resources should be referenced for larger channels having seasonal low perennial flow.

Use caution when design flow for the swale is greater than 100 cubic feet per second (cfs) from a 10-yr.-frequency storm. Generally, grassed swales are suitable for drainage areas less than 100 acres in flat to gently rolling terrain. In steeper terrain, it may be more difficult even on smaller drainage areas to design a stable waterway.

Planning Considerations

Constructed Channels vs. Natural Drainageways

Discretion must be used when replacing natural channels with constructed channels. Natural drainage systems, even small intermittent and ephemeral drainageways, provide many hydraulic and environmental benefits not duplicated by constructed channels. See the introduction to Stream Practices for more discussion of natural channel design.

CHAPTER 5

Temporary Runoff Control

Temporary runoff control is important on developing sites to minimize on-site erosion and to prevent off-site sediment discharge. Temporary runoff control primarily consists of two main strategies: keeping off site water clean and managing on site sediment laden water.

Off site water that passes through an active construction site should be kept as clean as possible. This is accomplished by routing this flow through the site without opportunity to mix with untreated site runoff or by diverting clean water can be diverted around construction areas. Sediment control practices generally will be more effective, less expensive and require less maintenance by not incorporating off site water. Diversions and temporary crossings are examples of runoff controls that are designed to keep off site water clean.

Once construction begins, erosion will occur. To minimize erosion, site runoff must be managed. This starts by minimizing the amount of disturbance and maintaining existing vegetation as much as possible to reduce the volume of runoff generated and subsequent erosion. It is important that the site designer and contractor recognize these opportunities to reduce site runoff.

Sediment laden water must be routed to an appropriate sediment control device before leaving the site. Conveying runoff through stabilized temporary diversions and slope drains not only directs muddy water to treatment devices but also reduces further erosion and prevents costly re-grading associated with gully development. Effective runoff control also makes re-vegetation easier and less costly.

The practices outlined in this chapter will significantly increase the effectiveness of a sediment and erosion control plan.

5.1 Rock Check Dams

Description

Check dams are small rock dams constructed in swales, grassed waterways or diversions. They reduce the velocity of concentrated flows, thereby reducing erosion within the swale or waterway. While this practice often traps some sediment, its trapping efficiency is extremely poor, thus, it should not be used as a primary sediment-trapping practice.

Condition Where Practice Applies

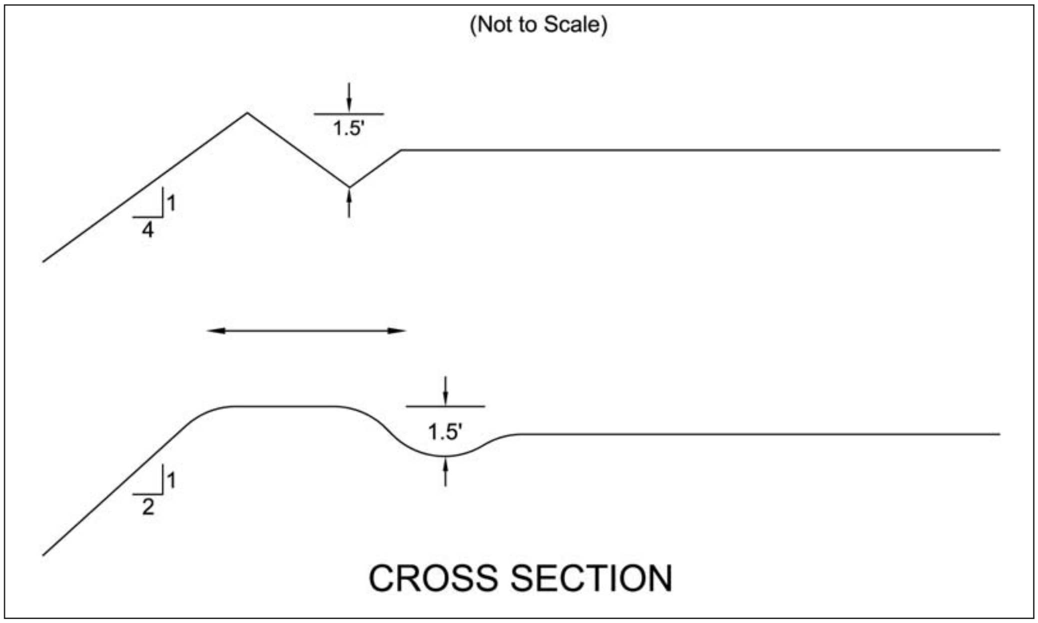
This practice is limited to use in small open channels where it is necessary to slow the velocity of flows in order to prevent erosion. Applications include temporary swales, which, because of their short length of service, are not practical to receive a nonerodible lining or swales, which need protection during the establishment of grass linings. See specifications for Rock Check and Gravel Riffle for larger channels and streams.

This practice is limited to small, open channels with a drainage area less than 10 acres with the object to protect live streams. Examples would be:

1. Ditches or swales that cannot receive a non-erodible lining and still need protection to reduce erosion.
2. The interim period while grassed lining is being established.
3. As an aid (not a substitute) to trap sediment from construction activity.

Specifications
for

Temporary Diversion



1. Drainage area should not exceed 10 acres. Larger areas require a more extensive design.
2. The channel cross section may be parabolic or trapezoidal. Disk the base of the dike before placing fill. Build the dike 10% higher than designed for settlement. The dike shall be compacted by traversing with tracked earth-moving equipment.
3. The minimum cross section of the levee or dike will be as follows: (Minimum design freeboard shall be 0.3 foot.) Where construction traffic will cross, the top width may be made wider and the side slopes flatter than specified above.

Table 5.3.2

Dike Top Width (ft.)	Height (ft.)	Side Slopes	Shape
0	1.5	4.1	Trapezoidal
4	1.5	2.1	Parabolic

4. The grade may be variable depending upon the topography, but must have a positive drainage to the outlet and be stabilized to be non-erosive.

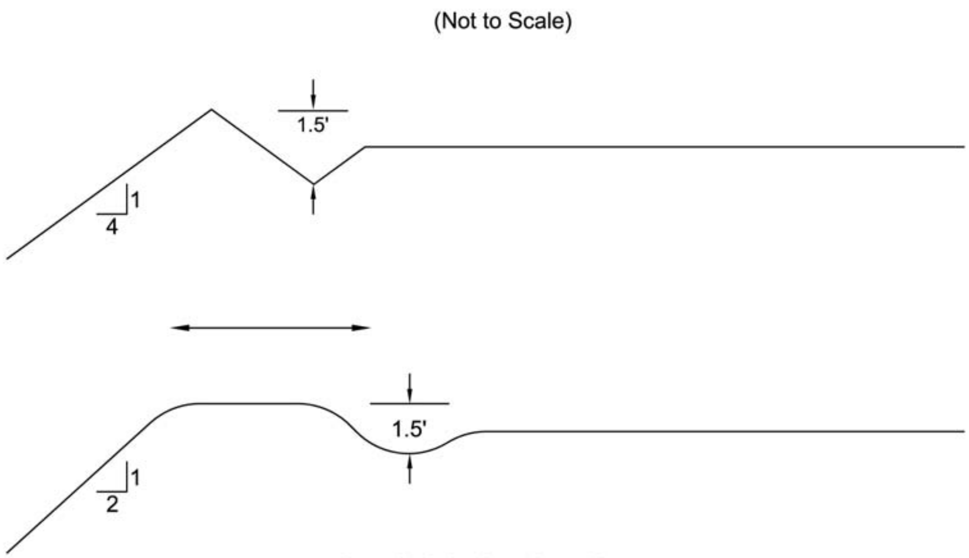
Table 5.3.3

Temporary Diversion Stabilization Treatment			
Diversion Slope	< 2 ac.	2 - 5 ac.	5 - 10 ac.
0 - 3%	Seed and Straw	Seed and Straw	Seed and Straw
3 - 5%	Seed and Straw	Seed and Straw	Matting
5 - 8%	Seed and Straw	Matting	Matting
8 - 20%	Seed and Straw	Matting	Engineered
Note: Diversions with steeper slopes or greater drainage areas are beyond the scope of this standard and must be designed for stability. Seed, straw and matting used shall meet the Specifications for Temporary Seeding, Mulching and Matting.			

5. Outlet runoff onto a stabilized area, into a properly designed waterway, grade stabilization structure, or sediment trapping facility.
6. Diversions shall be seeded and mulched in accordance with the requirements in practice standards TEMPORARY SEEDING (or PERMANENT SEEDING) and MULCHING as soon as they are constructed or other suitable stabilization in order to preserve dike height and reduce maintenance.

Specifications
for

Temporary Diversion Above Steep Slopes



CROSS SECTION

1. Drainage area should not exceed 5 acres. Larger areas require a more extensive design.
2. The channel cross section may be parabolic, v-shaped, or trapezoidal. Disk the base of the dike before placing fill. Build the dike 10% higher than designed for settlement. The dike shall be compacted by traversing with tracked earth-moving equipment.
3. The minimum cross section of the levee or dike will be as follows: (Minimum design freeboard shall be 0.3 foot.)

Table 5.3.2

Dike Top Width (ft.)	Height (ft.)	Side Slopes	Shape
0	1.5	4.1	Trapezoidal
4	1.5	2.1	Parabolic

4. The grade may be variable depending upon the topography, but must have a positive drainage to the outlet and be stabilized to be non-erosive.

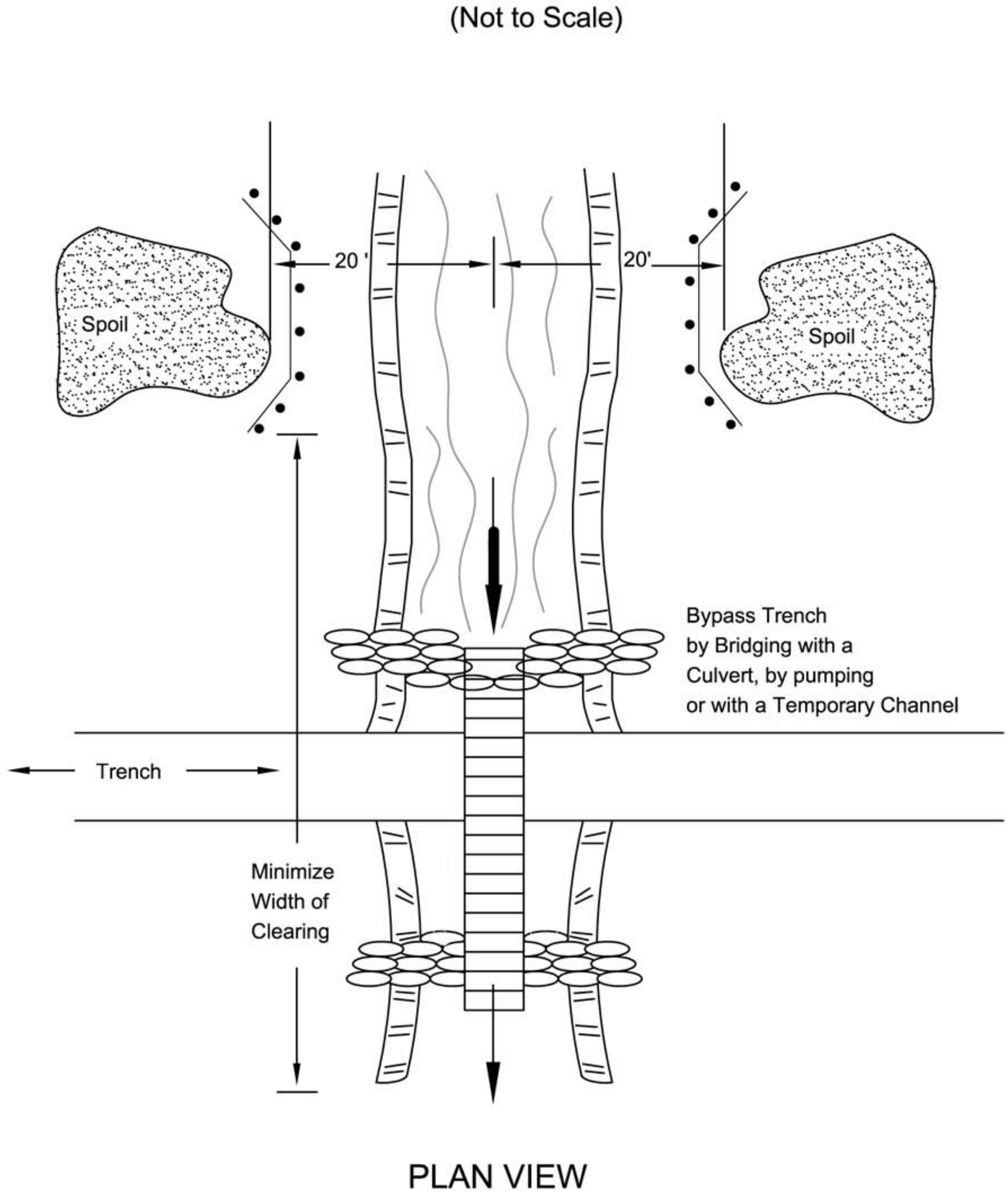
Table 5.3.3

Temporary Diversion Stabilization Treatment			
Diversion Slope	< 2 ac.	2 - 5 ac.	5 - 10 ac.
0 - 3%	Seed and Straw	Seed and Straw	Seed and Straw
3 - 5%	Seed and Straw	Seed and Straw	Matting
5 - 8%	Seed and Straw	Matting	Matting
8 - 20%	Seed and Straw	Matting	Engineered
Note: Diversions with steeper slopes or greater drainage areas are beyond the scope of this standard and must be designed for stability. Seed, straw and matting used shall meet the Specifications for Temporary Seeding, Mulching and Matting.			

5. Outlet runoff onto a stabilized area, settling pond, or into a drop structure.
6. Diversions shall be seeded and mulched in accordance with the requirements in practice standards TEMPORARY SEEDING (or PERMANENT SEEDING) and MULCHING as soon as they are constructed or other suitable stabilization in order to preserve dike height and reduce maintenance.

Specifications
for

Small Stream Utility Crossing



Note: Show appropriate practices for temporary equipment crossings and diversion of construction runoff. See Stream Crossing

Specifications
for

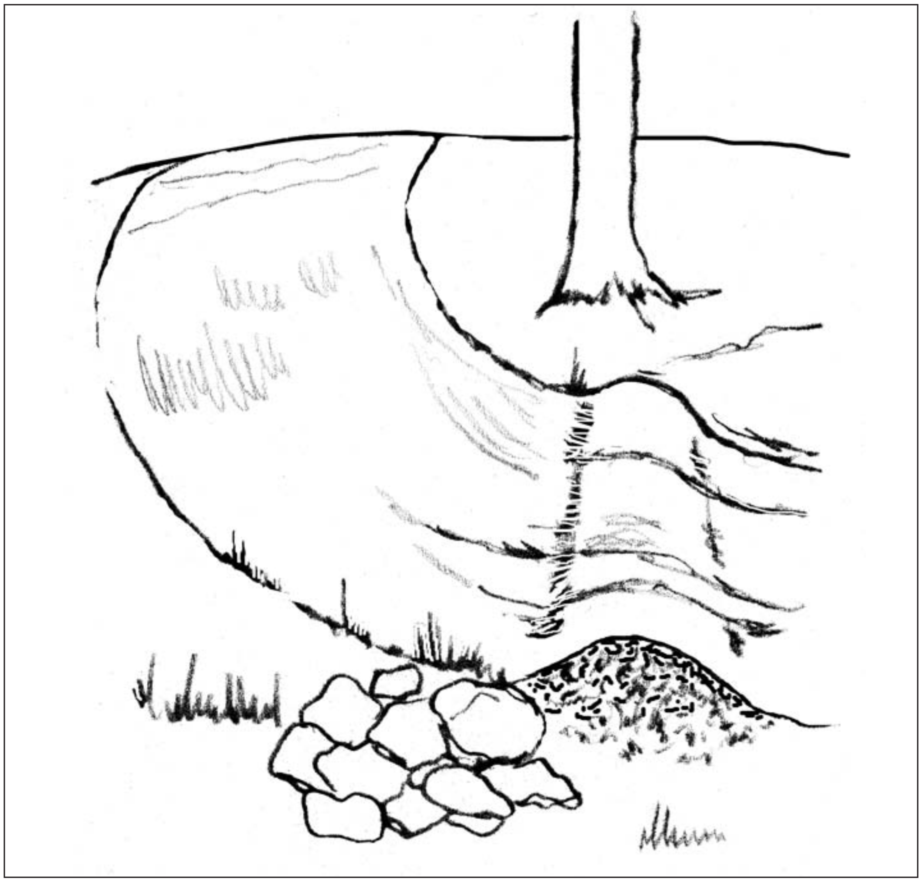
Stream Utility Crossing

1. When site conditions allow, one of the following shall be used to divert stream flow or keep the flow away from construction activity.

- Drill or bore the utility lines under the stream channel.
- Construct a cofferdam or barricade of sheet pilings, sandbags or a turbidity curtain to keep flow from moving through the disturbed area. Turbidity curtains shall be a pre-assembled system and used only parallel to flow.
- Stage construction by confining first one-half of the channel until work there is completed and stabilized, then move to the other side to complete the crossing.
- Route the stream flow around the work area by bridging the trench with a rigid culvert, pumping, or constructing a temporary channel. Temporary channels shall be stabilized by rock or a geotextile completely lining the channel bottom and side slopes.

2. Crossing Width -The width of clearing shall be minimized through the riparian area. The limits of disturbance shall be as narrow as possible including not only construction operations within the channel itself but also clearing done through the vegetation growing on the streambanks.
3. Clearing shall be done by cutting NOT grubbing. The roots and stumps shall be left in place to help stabilize the banks and accelerate revegetation.
4. Material excavated from the trench shall be placed at least 20 ft. from the streambanks.
5. To the extent other constraints allow, stream shall be crossed during periods of low flow.
6. Duration of Construction -The time between initial disturbance of the stream and final stabilization shall be kept to a minimum. Construction shall not begin on the crossing until the utility line is in place to within 10 ft. of the streambank.
7. Fill Placed Within the Channel -The only fill permitted in the channel should be clean aggregate, stone or rock. No soil or other fine erodible material shall be placed in the channel. This restriction includes all fill for temporary crossings, diversions, and trench backfill when placed in flowing water. If the stream flow is diverted away from construction activity the material originally excavated from the trench may be used to backfill the trench.
8. Streambank Restorations -Streambanks shall be restored to their original line and grade and stabilized with riprap or vegetative bank stabilization.
9. Runoff Control Along the Right-of-Way -To prevent sediment-laden runoff from flowing to the stream, runoff shall be diverted with water bar or swales to a sediment trapping practice a minimum of 50 ft. from the stream.
10. Sediment laden water from pumping or dewatering or pumping shall not be discharged directly to a stream. Flow shall be routed through a settling pond, dewatering sump or a flat, well-vegetated area adequate for removing sediment before the pumped water reaches the stream.
11. Dewatering operations shall not cause significant reductions in stream temperatures. If groundwater is to be discharged in high volumes during summer months, it shall first be routed through a settling pond or overland though a flat well-vegetated area.
12. Permits -In addition to these specifications, stream crossings shall conform to the rules and regulations of the U.S. Army Corps of Engineers for in-stream modifications (404 permits) and Ohio Environmental Protection Agency's State Water Quality Certification (401 permits).

5.6 Water Bar



Description

A water bar is a diversion constructed across the slope of an access road or utility right-of-way. Water bars are used to reduce concentrated runoff on unpaved road surfaces, thus reducing water accumulation and erosion gullies from occurring. Water bars divert runoff to road side swales, vegetated areas or settling ponds.

Conditions Where Practice Applies

Water bars are used at construction site ingress/egress points, on long sloping access roads, on temporary construction roads, or at utility right-of-ways which do not have a stable surface or where runoff would otherwise collect and cause erosion.

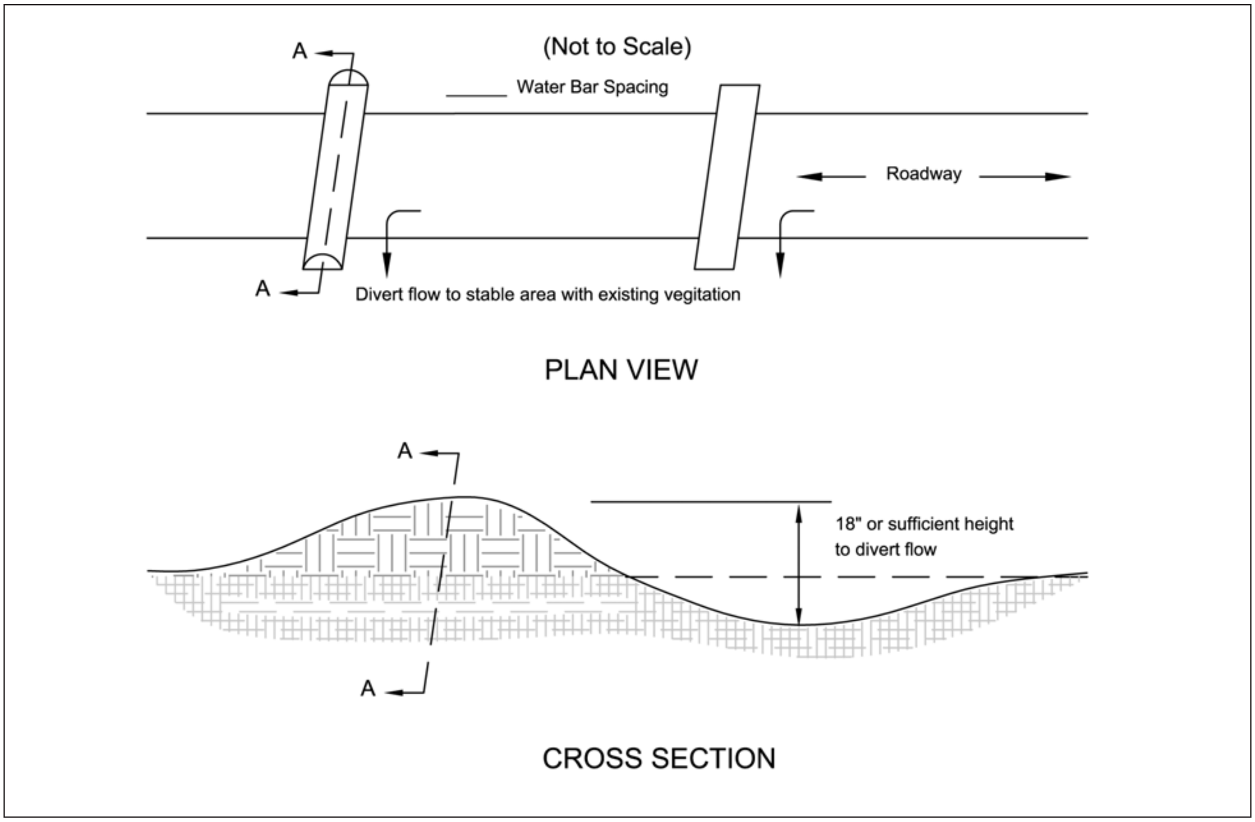
Planning Considerations

If the contributing area is disturbed, this practice should be associated with sediment traps that will receive and treat the runoff.

The outlet of each water bar must be resistant to erosion. For small contributing areas, spreading the flow into a undisturbed vegetated area may be sufficient. For larger areas or higher velocities flow may need rock outlet protection to prevent gully erosion.

Specifications
for

Water Bar



1. The minimum water bar dimensions shall be:

   Top width of berm/dike – 2 feet minimum.

   Height/depth – 18 inches unless otherwise noted on plans.

   Side Slopes – Sufficiently flat to accommodate the expected traffic.

2. The spacing between water bars shall be as noted:

   Table 5.6.1 Water Bar Spacing

Road Grade (%)	Distance (Ft.)
1	400
2	250
5	135
10	80
15	60
20	45

3. The field location shall be adjusted as needed to provide a stabilized safe outlet.

4. The diverted runoff shall be directed onto an undisturbed vegetative area, to a settling trap or basin or trap if contributing area is stable.

5. Diversions/dikes shall be compacted by traversing with equipment during construction.

6. The water bars shall be angled slightly downslope across the centerline of the travel lane.

1. POOL DESIGN:

Capacity – The minimum total design volume for the sediment basin shall consist of two components, the dewatering zone and the sediment storage zone. These zones are shown schematically in Figure 6.1.3. The volume of the dewatering zone shall be calculated for the entire drainage area by the method shown below. The drainage area includes the entire area contributing runoff to the sediment basin, offsite as well as on.

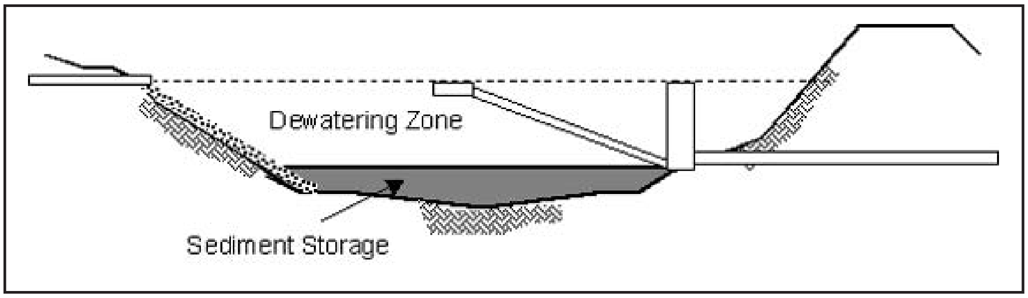


Figure 6.1.3 Pool showing dewatering area and additional sediment storage area

a) Dewatering Zone Volume -

The volume of the dewatering zone shall be a minimum of 1800 cubic feet per acre of drainage (67 yd^3/acre) or the minimum stated in the current NPDES construction general permit. Increasing this volume will increase the effectiveness of the basin, provided dewatering times are appropriately adjusted as well.

b) Sediment Storage Zone Volume -

The volume of the sediment storage zone shall be calculated by one of the following methods.

Method 1: The volume of the sediment storage zone shall be 1000 cubic feet (37 cubic yards) per disturbed acre within the watershed of the basin. OR

Method 2: The volume of the sediment storage zone shall be the volume necessary to store the sediment as calculated with RUSLE or a similar generally accepted erosion prediction model. While the sediment storage volume may extend to the expected time period of the construction project, the minimum estimated time between cleanouts shall be six months.

The total volume of the dewatering zone and the sediment storage zone shall be provided below the principal spillway elevation. The elevation at which the sediment storage zone reaches the design capacity should be designated by the top of stake located near the center of the basin. Accumulated sediment shall be removed from the basin whenever it reaches that elevation on the cleanout stake.

Depth – The pool shall be configured to maximize the optimum depth of 3 ft. Depths over 5 ft. should be avoided. The depth shall be measured to the invert of the principal spillway. These are optimum criteria and will not be feasible for all sediment basins.

Flow Length-to-Width Ratio – The length-to-width ratio shall be 4:1 or greater. If the flow length from the inlet of the basin to the principal spillway is not greater than or equal to the minimum length, either the inlet of the basin should be relocated farther away from the principal spillway, or one or more solid baffles should be used to increase the flow length within the basin. Flow length is to be measured at the elevation of the invert of the principal spillway. Where runoff from disturbed areas enters the basin from different directions, it is better to combine flows from the various areas into a single inlet into the basin rather than have multiple inlets into the basin. If multiple inlets to the basin exist, the flow length to width ratio from all inlets must be at least 4:1.

Use of Baffles in Sediment Basins – If individual situations require greater trapping efficiency or if optimum depth and length-to-width ratios are not feasible, baffles may be incorporated into the design. Baffles may be constructed of porous or solid materials depending upon their purpose. Solid baffles, as shown in Figures 6.1.4 and 6.1.5, may be used to increase the flow length within the basin.

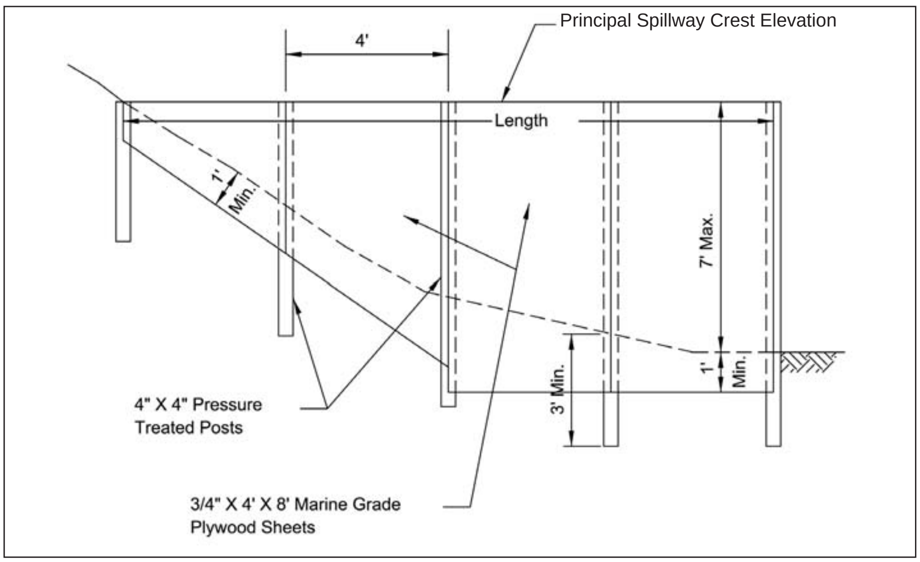


Figure 6.1.4 Typical construction of a solid baffle

Porous baffles, as shown in figure 6.1.5, are used to dampen turbulent currents and increase sedimentation. Porous baffles are typically constructed of jute matting, rock, plastic safety fence, or other material. Porous baffles typically partition the basin into two or three cells. Whether porous or solid baffles, the height shall extend to the crest elevation of the principal spillway.

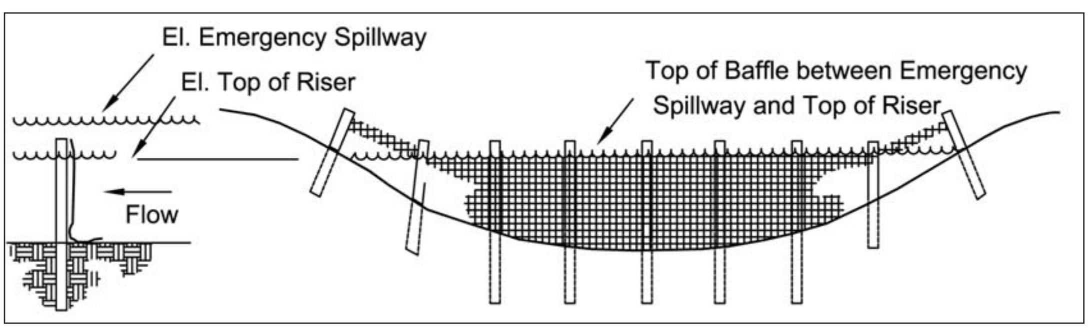


Figure 6.1.5 Porous baffle placed to increase pond efficiency (left shown in profile, right in cross-section)

Basin Inlet—A suitable protective lining for each collection channel or other device that discharges to the basin should be provided; the lining should extend to the bottom of the basin and at least 10' along the basin bottom for energy dissipation.

Safety—Sediment basins are attractive to children and can be dangerous, particularly where 2:1 or steeper side slopes lead directly into water 3 ft. or deeper. Danger is also increased where side slopes are not vegetated. Fencing and warning signs shall be installed to minimize the danger associated with sediment basins.

2. EMBANKMENT DESIGN:

Embankment Slope—Embankment slopes must be sufficiently flat to ensure stability; however, in all cases the combined upstream and downstream side slopes of the settled embankment shall not be less than 5 horizontal to 1 vertical (5:1) with neither slope being steeper than 2:1.

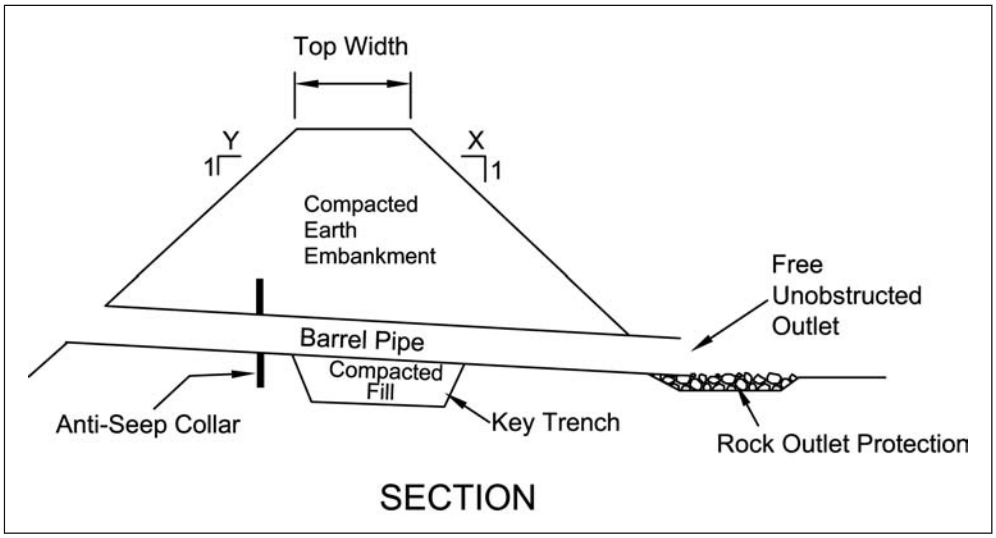


Figure 6.1.6 Embankment Design

Embankment Cutoff Trench—Use cutoff trenches to prevent seepage from flowing along the foundation of the embankment. Install cutoff trenches to a depth that extends into a relatively impervious layer. In all cases the minimum depth shall be 3 ft. and constructed of mechanically compacted material. A cutoff trench shall have a bottom width adequate to accommodate the equipment used for excavation, backfill, and compaction operations. Side slopes shall be no steeper than 1:1.

Embankment Settlement—The embankment design height shall be increased by the amount needed to ensure that after all settlement has taken place the height of the dam will equal or exceed the design height. This increase shall not be less than 5%.

Embankment Top Width—The minimum top width of the embankment shall be as shown below.

Table 6.1.1 Embankment Top Width

Embankment Height at Centerline (ft.)	Minimum Top Width (ft.)
< 15	8
15 – 20	10
> 20	12

3. DEWATERING DEVICE DESIGN:

Dewatering should be part of all sediment basins. The minimum dewatering time for sediment basins is 48 hours. The maximum dewatering time should not exceed 7 days. The lower limit of dewatering is the top of the sediment storage zone, or the top of the permanent pool if a permanent pool is used. The upper limit is the crest of the principal spillway. Sediment basins shall be dewatered using a device that discharges water from the top of the dewatering zone.

Typical methods or devices for accomplishing this may include the following: skimmers, floating pumps, siphons or other acceptable methods that provide dewatering between 48 hours and 7 days. Where ice or other reasons make dewatering from the top of the water surface impractical, multiple orifices or a single orifice may be used to dewater down to the top of the sediment storage zone. Any dewatering of the sediment zone must be accomplished using protected dewatering methods (e.g. perforated riser with gravel cone or wire mesh and filter fabric covering perforations). All of these methods are appropriate for meeting the requirements of this standard, but only sizing procedures for skimmers are included below. Concern regarding ice may justify changing outlet types during months of hardest freezing or provide frequent monitoring and mainteance as a means of preventing freezing of the skimmer.

A schematic of a skimmer is shown in figure 6.1.7 or 6.1.8. Typically a single orifice plate is placed in the discharge pipe to control water outflow or discharge. It is recommended that the orifice be placed near the water surface or floating device to allow a constant head and a more consistent discharge. Note the dewatering device is not the same as the principal or emergency spillway. However, the dewatering device outlet may be connected to the principal spillway outlet.

Sediment basins are often permanent stormwater detention facilities (wet pond, dry pond, ...) modified for sediment control use during construction. Permanent stormwater ponds and sediment basins often have different volume and drawdown requirements. Thus, if the same facility (basin) is to be used both for sediment control during construction and for permanent stormwater control, the facility will require two different outlet designs - one to be used during construction, and the other to be installed upon completion of the development. Plans should explicitly show design calculations and outlet design details for both uses and configurations.

It is recommended that calculation summaries and design details for both outlets be included on the same plan page during submittal for ease of evaluation by the reviewing agency. Table 6.1.1.a highlights summary information that should be included.

In addition, the point at which the temporary sediment basin outlet is to be replaced with the permanent stormwater basin outlet should be clearly specified on the page(s) with outlet design details.

Table 6.1.1.a Summary information for Sediment Basin versus Permanent Stormwater Facility

	Sediment Basin				Permanent Stormwater Facility			
Contributing Drainage Area (ac.)	Dewatering Volume (yd^3) or (ac-ft)	Sediment Storage Volume (yd^3) or (ac-ft)	Detention Time Min 48 hr (hours)	Sediment Control Orifice Size (in) or (in^2)	Water Quality Volume-WQv (yd^3) or (ac-ft)	Permanent Pool Volume (yd^3) or (ac-ft)	Detention Time (hours)	WQv Orifice Size (in) or (in^2)

6.2 Sediment Trap

Description

A sediment trap is a temporary settling pond formed by construction of an embankment and/or excavated basin and having a simple outlet structure that is typically stabilized with geotextile and rip-rap. Sediment traps are constructed to detain sediment-laden runoff from small, disturbed areas for a sufficient period of time to allow the majority of the sediment to settle out. They are established early in the construction process using natural drainage patterns and favorable topography where possible to minimize grading.

Conditions Where Practice Applies

Sediment traps are used:

1. At the outlets of diversions, channels, slope drains, or other runoff conveyances that discharge sediment–laden water.
2. Below disturbed areas where the total contributing drainage area is **5 acres or less**. If the contributing drainage area is greater than 5 acres, the use of a Sediment Basin is recommended.
3. Where access can be maintained for removal and proper disposal of sediment.
4. In drainage swales or areas, where sediment control is needed upstream of a drainage pattern leading to a storm drain inlet.
5. Where the required life of the structure will be 18 months or less.

6. Where failure of the structure will not result in loss of life; or cause damage to buildings, roads, utilities, or other properties.

Note: Sediment traps, that have the entire capacity achieved through excavation, may have larger drainage areas without compromising the stability of the sediment trap.

Planning Considerations

Timing – Sediment traps shall be constructed as a first step in any land-disturbing activity, and shall be made functional before upslope land disturbance takes place. Sediment traps are temporary measures with a typical design life of 6 months to 18 months. One or more traps are often built early in the construction process to capture sediment, prior to construction of a larger structure (e.g., sediment basin or modified detention basin) is constructed. Sediment traps are to be functional during the entire construction process, both before and after new drainage systems are constructed.

Location – Sediment traps usually are placed near the edges of construction sites so to be out of the way of major construction activities.

Diverting Runoff – Temporary diversions at the perimeter of sites are used to direct runoff to sediment traps (see Temporary Diversion Specifications).

Storm-Sewer Diversions – Storm drains may be temporarily redirected through sediment traps during construction. After construction, the temporary pipes are removed and runoff is allowed to flow through the permanent storm drain as originally intended.

Utilities – Give special consideration to sediment trap location and possible interference with construction of proposed drainage ways, utilities and storm drains.

Trapping Efficiency – Improved sediment trapping efficiencies can be achieved by including both a “wet” storage volume and a drawdown or “dry” storage volume that enhances settling and prevents excessive sediment losses during large storm events. In order to maintain effectiveness, sediment must be periodically removed from the trap to maintain the required design volume. Frequent inspection and appropriate maintenance should be provided until the construction site is permanently protected against erosion.

Design Criteria

Capacity - The minimum total design volume for the sediment trap shall consist of two components, the dewatering zone and the sediment storage zone. These zones are shown schematically in Figure 6.2.1. The volume of the dewatering zone shall be calculated for the entire drainage area by the method shown below. The drainage area includes the entire area contributing runoff to the sediment basin, offsite as well as on. The sediment storage volume may be in the form of a permanent pool or wet storage to provide a stable-settling medium, while the dewatered volume shall be in the form of a draw down or dry storage of at least 67 cubic yards per acre which will provide extended settling time during less frequent, larger storm events.

a) Dewatering Zone Volume –

The volume of the dewatering zone shall be a minimum of 1800 cubic feet per acre of drainage (67 yd3/acre) or the minimum stated in the current NPDES construction general permit. The total volume of the dewatering zone shall be measured from the base of the stone outlet structure to the crest of the stone outlet structure.

b) Sediment Storage Zone Volume –

The volume of the sediment storage zone shall be calculated by one of the following methods. The sediment storage zone shall be measured below the elevation of the base of the stone outlet structure.

Method 1: The volume of the sediment storage zone shall be 1000 cu. ft. per disturbed acre within the watershed of the basin; OR

Method 2: The volume of the sediment storage zone shall be the volume necessary to store the sediment yield as calculated with RUSLE or a similar generally accepted erosion prediction model. While the sediment storage volume may extend to the expected time period of the construction project, the minimum estimated time between cleanouts shall be six months.

Sediment shall be removed when it has accumulated to the top of the sediment storage or wet storage zone. This elevation shall be signified by the top of a stake near the center of the trap.

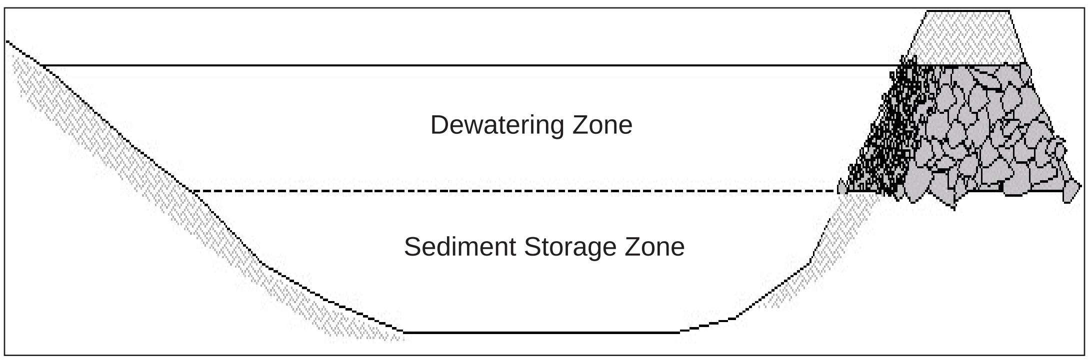


Figure 6.2.1 Capacity of a sediment trap is distributed between dewatering and sediment storage zones.

Embankment – Ensure that embankments for temporary sediment traps do not exceed 5 feet in height measured at the centerline from the original ground surface to the top of the embankment. Construct embankments with a minimum 4 foot top width and 2:1 (H:V) or flatter side slopes.

The design height of the embankment shall be increased by 5% to allow for settlement of the finished embankment. The original ground under the embankment shall be stripped of vegetation and scarified to a depth of 6 inches or more before placement of the fill material. Fill material should be made of clay, free of roots, large rocks, and organic material. Place fill in layers 6 inches thick and then compact using appropriate equipment. Fill material shall not be placed on frozen ground.

The completed embankment shall be seeded in accordance with temporary or permanent vegetation as found in this manual (Temporary Seeding or Permanent Seeding).

Excavation – Where sediment pools are formed or enlarged by excavation, keep side slopes at 2:1 (H: V) or flatter for safety. The maximum depth of excavation within the wet storage area (sediment storage zone) should be 4 feet to facilitate clean out and for site safety considerations.

Outlet Section – Construct the sediment trap outlet using a stone section of embankment located at the low point in the basin. The stone section serves two purposes: 1) the top section serves as a non-erosive spillway outlet for flood flow, and 2) the bottom section provides a means to de-watering the basin between runoff events. A combination of coarse aggregate and riprap shall be used to provide for filtering/detention as well as outlet stability.

Construct the outlet using well-graded stones with a d50 size larger than 6 inches (ODOT Type D). A 1 foot layer of AASHTO # 57 aggregate should be placed on the inside face to reduce drainage flow rate. Geotextile that meets the minimum requirements of ODOT Construction and Material Specification 712.09, Geotextile Fabric Type B, shall be placed at the stone-soil interface to act as a separation and to prevent piping. The geotextile shall be buried or keyed in at the upstream end a minimum of 6 inches. The crest of the stone outlet must be at least 1.5 feet below the top of the embankment to ensure that the flow will travel over the stone and not the embankment. The outlet shall be configured as noted in figure 2.

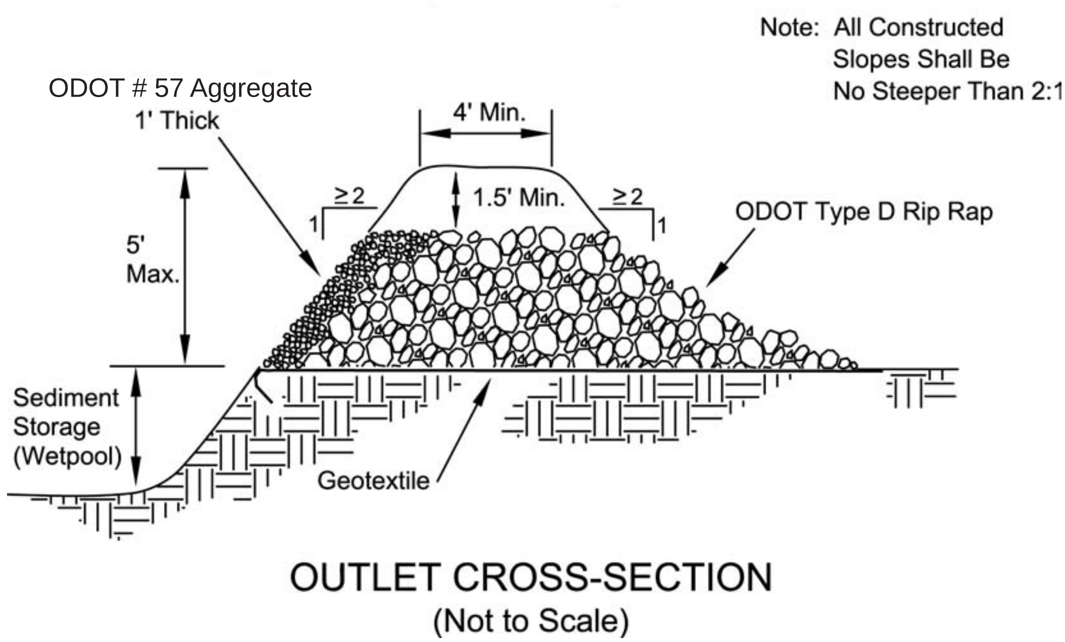


Figure 6.2.2 Outlet configuration

The spillway weir shall be at least 4 feet long and sized to pass the peak discharge of the 10-year, 24-hour storm without failure, overtopping of the basin or significant erosion. A maximum flow depth of 1 foot, a minimum freeboard of 0.5 foot, and maximum side slopes of 2:1 are required. See Table 6.2.1 for weir length associated with drainage area.

Table 6.2.1 Sediment Trap weir length.

Drainage Area (acres)	Weir Length (feet)
1	4.0
2	6.0
3	8.0
4	10.0
5	12.0

Note: alternatively use $Q_{weir} = CLH^{3/2}$
Where C = Weir coefficient
L = Weir Length (feet)
H = Head of 1 foot

Direct spillway discharges to natural, stable areas. Locate outlets so that flow will not damage the embankment. Discharges must be conveyed to a natural waterway via a channel of adequate capacity and stability. Where the channel enters a natural waterway, the discharge shall be less than 1 ½ feet per second or otherwise less than the velocity that will initiate erosion or scour within the receiving waterway. When traps discharge to storm water facilities, the facility must have adequate capacity to receive the discharge from the sediment trap.

Where an emergency spillway is utilized, the primary rock spillway crest should be at least 1.5 feet below the settled top of the embankment with the emergency spillway crest being 0.5 foot below the top of the embankment.

The plans and specifications should show the following requirements:

1. Location of the sediment traps.
2. Size of sediment trap including width, length and depth.
3. Minimum cross section of embankment.
4. Typical cross section through the spillway with geotextile fabric details and rock placement.
5. Location of emergency spillway, if used.
6. Gradation and quality of rock.
7. Plans shall detail how excavated sediment is to be disposed of, such as placement on areas where it will be stabilized or removal to an approved off-site location.

All plans should include the installation and maintenance schedules with the responsible party identified.

Install warning signs, barricades, perimeter fence and other measures around sediment traps as necessary to protect workers, children, equipment, etc.

Operation and Maintenance

1. The capacity and function of the sediment trap shall be maintained by inspecting on a weekly basis and after each runoff event, and by performing the necessary activities shown below.
2. Establish vegetative cover and fertilize as necessary to maintain a vigorous cover around the sediment trap.
3. Inspect the pool area, embankment and spillway area for burrowing rodents, slope failure, seepage, excess settlement, and displaced stone. The area should be inspected for structural soundness and repaired as needed.
4. Regularly inspect water discharged from trap for excess suspended sediments. Identify and perform necessary repairs to improve water quality. Excessive suspended sediments may require design modifications or treatment with flocculants.
5. Remove woody vegetated growth on the embankment and spillway areas.
6. Remove trash and debris that accumulate in the pond and have potential to block spillways.
7. Dewatering outlets shall be regularly checked to ensure that performance is maintained. Filter stone choked with sediment shall be removed and replaced to restore its flow capacity.
8. Remove sediment and restore the sediment trap to its original dimensions when sediment has accumulated to the top of the sediment storage or wet storage zone. This elevation shall be signified by the top of a stake near the center of the trap. Removing sediment by hand may be necessary adjacent to the outlet section of the embankment to prevent equipment damage. Place the removed sediment and stabilize with vegetation in a designated area where it will not easily erode again. Restore trap to its original dimensions and replace stone as needed on the outlet.
9. After the entire construction project is completed, temporary sediment traps should be dewatered and regraded so as to conform to the contours of the area. All temporary structures should be removed and the area seeded, mulched and stabilized as necessary.

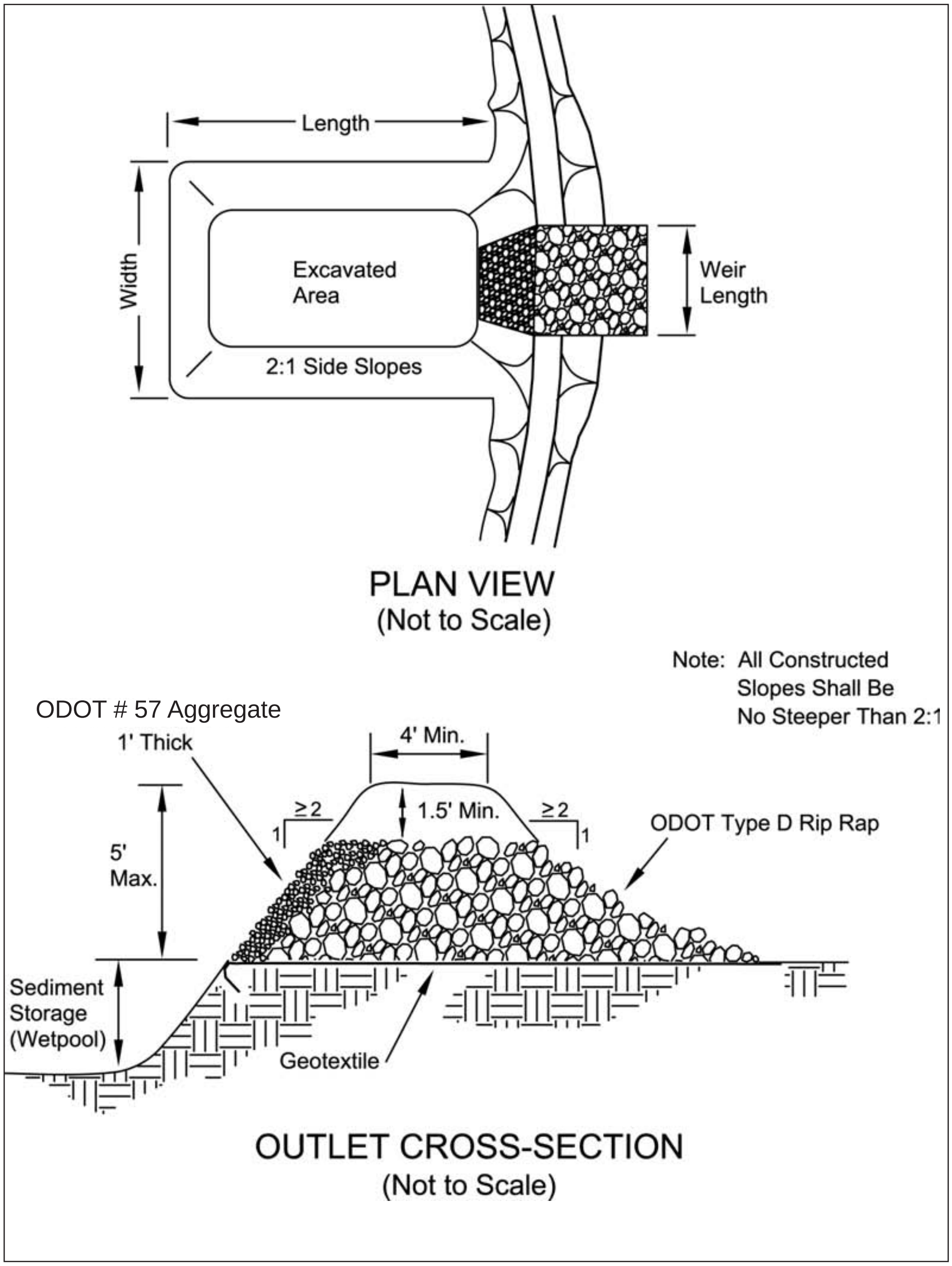
Common Problems/Concerns

Utilizing sediment traps on large drainage areas (greater than 5 acres) where Sediment Basins (see page 2 of this chapter) are appropriate will increase sediment discharged during construction.

Failure to removed trapped sediment will reduce the effectiveness of this practice in capturing sediment.

Specifications
for

Sediment Trap



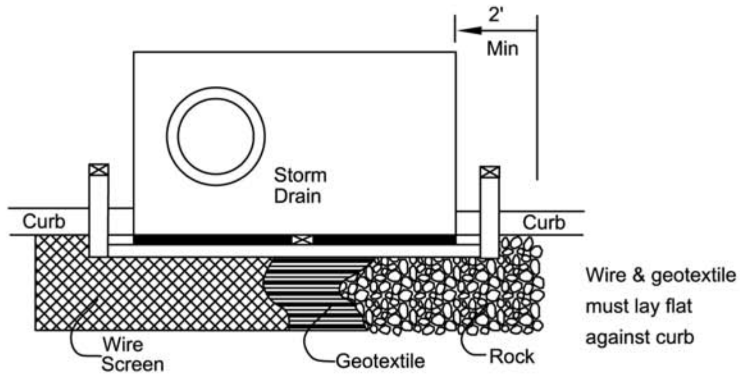
Specifications
for

Sediment Traps

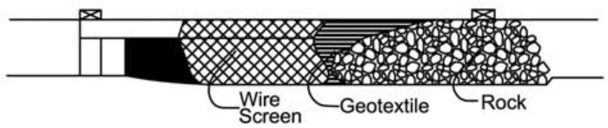
1. Work shall consist of the installation, maintenance and removal of all sediment traps at the locations designated on the drawings.
2. Sediment traps shall be constructed to the dimensions specified on the drawings and operational prior to upslope land disturbance.
3. The area beneath the embankment shall be cleared, grubbed and stripped of vegetation to a minimum depth of six (6) inches. The pool shall be cleared as needed to facilitate sediment cleanout.
4. Fill used for the embankment shall be evaluated to assure its suitability and it must be free of roots or other woody vegetation, large rocks, organics or other objectionable materials. Fill material shall be placed in six (6) inch lifts and shall be compacted by traversing with a sheepsfoot or other approved compaction equipment. Fill height shall be increased five (5) percent to allow for structure/foundation settlement. Construction shall not be permitted if either the earthfill or compaction surface is frozen.
5. The maximum height of embankment shall be five (5) feet. All cut and fill slopes shall be 2:1 (H:V) or flatter.
6. A minimum storage volume below the crest of the outlet of 67 yd^3. for every acre of contributing drainage area shall be achieved at each location noted on the drawings with additional sediment storage volume provided below this elevation.
7. Temporary seeding shall be established and maintained over the useful life of the practice.
8. The outlet for the sediment trap structure shall be constructed to the dimensions shown on the drawings.
9. The outlet shall be constructed using the materials specified on the drawings. Where geotextile is used, all overlaps shall be a minimum of two (2) feet or as specified by the manufacturer, whichever is greater. All overlaps shall be made with the upper most layer placed last. Geotextile shall be keyed in at least 6" on the upstream side of the outlet.
10. Warning signs and safety fence shall be placed around the traps and maintained over the life of the practice.
11. After all sediment-producing areas have been permanently stabilized, the structure and all associated sediment shall be removed. Stabile earth materials shall be placed in the sediment trap area and compacted. The area shall be graded to blend in with adjoining land surfaces and have positive drainage. The area shall be immediately seeded.

Specifications
for

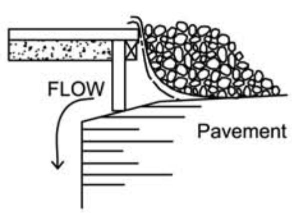
Geotextile - Stone Inlet Protection for Curb Inlets



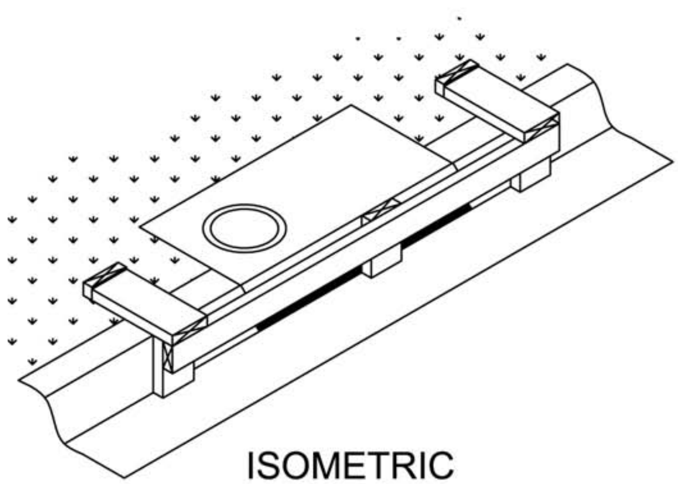
PLAN VIEW



ELEVATION



CROSS SECTION



ISOMETRIC

Specifications
for

Geotextile-Stone Inlet Protection for Curb Inlets

1. Inlet protection shall be constructed either before upslope land disturbance begins or before the inlet becomes functional.
2. Construct a wooden frame of 2-by-4-in. construction-grade lumber. The end spacers shall be a minimum of 1 ft. beyond both ends of the throat opening. The anchors shall be nailed to 2-by-4-in. stakes driven on the opposite side of the curb.
3. The wire mesh shall be of sufficient strength to support fabric and stone. It shall be a continuous piece with a minimum width of 30 in. and 4 ft. longer than the throat length of the inlet, 2 ft. on each side.
4. Geotextile cloth shall have an equivalent opening size (EOS) of 20-40 sieve and be resistant to sunlight. It shall be at least the same size as the wire mesh.
5. The wire mesh and geotextile cloth shall be formed to the concrete gutter and against the face of the curb on both sides of the inlet and securely fastened to the 2-by-4-in. frame.
6. Two-inch stone shall be placed over the wire mesh and geotextile in such a manner as to prevent water from entering the inlet under or around the geotextile cloth.
7. This type of protection must be inspected frequently and the stone and/or geotextile replaced when clogged with sediment.

CHAPTER 7

Soil Stabilization

Soil Stabilization is the most effective means to minimize erosion and off-site sediment from development sites. Stabilized soils have vegetative or other types of cover left during construction or replaced following disturbance in order to prevent wind or water. Maintaining stabilization involves taking key steps at planning and continuing until the end of construction.

During project planning all efforts should especially be made to retain existing vegetation. This can be accomplished by phasing construction activity, using 'open space' design concepts, and minimizing corridor widths for road and utility construction. Special emphasis on preserving natural vegetation should be made near sensitive areas such as wetlands, stream corridors, steep slopes, and woodlots. Ideally, natural areas should be set aside permanently; however even delaying disturbance of portions of a site through phasing will prevent significant erosion from occurring. Areas not to be disturbed must be shown on construction plans and clearly marked in the field.

Once clearing and grading begins, erosion will occur until the site is re-stabilized. This occurs as rough or finish grading operations become idle or finished and are seeded and mulched as soon as possible, during any season. The most common methods, seeding and mulching are relatively inexpensive, easy to implement, and requires minimum maintenance. No matter which method is used, all stabilization practices significantly reduce off-site pollution and reliance on more costly and less reliable sediment treatment practices.

7.1 Minimized Phased Disturbance

Description

Phased disturbance limits the total amount of grading at any one time and sequences operations so that at least half the site is either left as undisturbed vegetation or re-stabilized prior to additional grading operations. This approach actively monitors and manages exposed areas, so that erosion is minimized and sediment controls can be more effective in protecting aquatic resources and downstream landowners.

Condition Where Practice Applies

This practice can be applied anywhere development occurs and is well suited to protect critical areas on and off site, such as wetlands, streams, ponds and highly erodible areas subject to high erosion rates. The practice is applicable where natural vegetation can act as a soil stabilizer during development and perhaps as a water quality feature after construction.

Planning Considerations

Two planning principles should be applied for phased disturbance. First, developments should be fit around the natural site conditions (e.g. topography, drainage, vegetation and setting) and thus involve less grading and fewer offsite impacts than conventional development patterns. Practically this means retaining undisturbed green space around water resources and on critical areas like steep slopes.

The second planning principle is focused on managing active construction, so that at least 50% of the land area is maintained in vegetation. By anticipating the timing and extent each grading and construction operation, along with erosion and sediment controls, exposed ground does not sit idle. This management principle is applied by developing phases of a project that can be brought to completion quicker than the entire parcel; and by utilizing an effective construction sequence to assist project managers to anticipate the next step towards stabilization and completion.

Ideally with phasing and effective sequencing, a parcel is divided between vegetated inactive areas and active areas where work is continuous from clearing operations, through grading, drainage and construction until final re-stabilization with vegetation. A realistic construction sequence is an essential planning tool for this practice with the goal that only areas under active construction have exposed soils.

Construction Operation	J	F	M	A	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J
PHASE 1: Roadway, Storm & Utilities	◄						►												
Install construction site entrance	•																		
Fence natural & tree protection area	•																		
Install SW/sed basin, diversion and silt F.		•																	
Seed SW/Sed basin areas		•																	
Clear ROW		•																	
Grading, install storm, San. and utilties		•	•																
Place inlet protection on storm sewers			•																
Grade road swales and stabilize			•	•															
Road construction				•	•														
Seed/mulch graded areas					•	•													
PHASE 1: Home Construction					◄					►									
Clear home sites					•	•	•												
Install silt fence & filter berms					•														
Basement excavation & rough grading					•	•	•	•											
Temporary seeding on lots						•	•	•	•										
Final yard grading								•	•	•									
Permanent seed and mulch								•	•	•									
PHASE 2: Roadway, Storm & Utilities								◄			►								
Install sediment trap, silt F. and filter B.								•											
Seed sediment trap								•											
Grading, install storm, San. and utilties									•	•									
Place inlet protection on storm inlets										•									
Erosion control matting on swales										•									
Road construction											•								
Winterization- Seed/mulch graded areas										•	•								
PHASE 2: Home Construction												◄							►
Clear home sites												•	•						
Install filter berms													•						
Basement excavation														•	•				
Temporary seeding on lots															•	•	•		
Final yard grading																	•	•	
Permanent seeding and mulching																		•	•
Remove temp riser, clean out SW pond																			•
Adapt SW pond outlet for permanent configuration																			•

Figure 7.1.1 Sample Sequence of Construction Operations

Design Criteria

Specify all major construction operations including erosion and sediment controls with the estimated time for completion in a sequence of operations (see Figure 7.1.1). The sequence of operations shall be noted on construction drawings. Changes should be made to the construction sequence as work is completed or delayed.

Divide site work into major phases so that no more than 50% of the site is exposed at any one time. Within each phase, operations such as clearing can also be divided to keep from removing all the vegetation at once. For example, clearing for a roadway and infrastructure can be effectively separated from clearing operations required for homebuilding rather than removing all vegetation at once.

All areas that are disturbed shall be provided with appropriate controls such as sediment basins, traps or barriers to prevent sediment from impacting water resources or offsite areas. Disturbed areas that are expected to be inactive (idle) for 21 days or longer will be temporarily stabilized until the subsequent construction operations begin or permanent seeding and mulching can be completed.

Maintenance

Monitoring is essential to ensure that phasing and sequencing occur properly. This includes making sure only the areas that need to be exposed are exposed, and all other BMP practices are in good working order.

Routinely verify that work is progressing in accordance with the project's construction sequence. If progress deviates, take corrective actions.

When changes to the project schedule are unavoidable, amend the construction sequence schedule on drawings and plans well in advance to anticipate potential problems and maintain control.

Common Problems/Concerns

Proper planning not conducted – more than 50% of the site is bare at any one time. Areas may be too large and may need to be managed in smaller increments.

Active disturbance of the entire site does not allow portions to reach stages of completion so that temporary or permanent seeding and mulching can be employed. A failure to limit work areas to phases will result in erosion and sediment control being less effective.

Failure to anticipate completion dates for final or temporary grading stages can leave disturbed areas unprotected during winter months.

Failure to follow the construction sequence or maintain may result in erosion and sediment control items being delayed.

Temporary seeding and revegetation of graded areas is delayed as other work slows. Some areas such as slopes should proceed with seedings even though delays in other operations are occurring.

7.2 Clearing and Grubbing

Description

Clearing and grubbing is the removal of trees, brush and other unwanted material in order to develop land for other uses or provide access for site work. Clearing generally describes the cutting and removal of above ground material while grubbing is the removal of roots, stumps, and other unwanted material below existing grade.

Clearing and grubbing includes the proper disposal of materials and the implementation of best management practices in order to minimize exposure of soil to erosion and causing downstream sedimentation.

Condition Where Practice Applies

This practice may be applied anywhere existing trees and other material must be removed for development to occur. The potential for erosion and sedimentation increases as: the vegetation removed; area disturbed or watercourses encountered increases.

Planning Considerations

Site assessment, selection and marking

Sites should be assessed to determine areas to be left undisturbed as well as trees or vegetated areas to be saved (see tree preservation area). These areas need to be clearly marked on plans and in the field. Land clearing activities should not begin until the site assessment and the field marking is concluded.

Timing and Phasing

Large-scale sites should be cleared in phases, with initiation of each phase delayed until actual construction is scheduled for that area of the site.

Erosion, sediment and stream instability potential

Clearing in some areas should be avoided or delayed due to the potential for destabilization. Cleared sites on heavy soils and steep slopes are subject to excessive erosion and may require additional practices to keep the soil in place. Land clearing during dry or frozen times will decrease compaction and potential water quality problems from runoff.

Stream corridors should be left in tact unless and until plans have been made to immediately restore stable conditions. These areas are subject to rapid erosion once vegetation is removed and soon become a source of sediment downstream. Alternatively naturally vegetated stream corridors help protect water resources from pollution generated during grubbing and grading operations.

Design Criteria

Timber Salvage – Develop plans specifying the kind and location of timber to be salvaged, the location of haul roads and skid trails, location and width of natural buffer zones around water bodies, and the location and methods of stream crossings. The method of disposing of all material that will not be salvaged should also be specified. Plans should also include the best management practices that will be used to protect the cleared area from erosion.

Identify and protect healthy trees following specifications in the **Tree and Natural Area Preservation** practice. Where possible, preserve a natural buffer/filter strip adjacent to all water bodies. Avoid clearing to the water bodies' edge.

1. Where it is necessary to clear to the water's edge, appropriate sediment control should be used and seeding and other stabilization should be initiated within 2 days of work becoming idle.
2. Phase work so that only part of the site is being cleared at any given time. This will reduce the amount of time soil is exposed to erosive forces. Follow examples in the **Phased Disturbance** practice.
3. Install earth diversions to intercept and divert runoff to stable outlets and appropriate sediment ponds.
4. All debris should be kept out of surface water resources. If possible, leave mulch or vegetation on the ground to decrease runoff and potential runoff. See the "Disposal Options" section, below.
5. Exposed areas not planning for immediate earthwork should be temporarily seeded to prevent further erosion at the site. See the **Temporary Seeding** practice. Additional stabilization or sediment control practices may be necessary to keep soil on the site.

Grubbing – Grubbing removes roots and stumps by digging or pushing over with earth moving equipment. Grubbing should be carefully monitored near lakes and streams to protect the water's edge. Removing root systems near the banks of streams and lakes make cause the area to become unstable and erode. If possible, avoid grubbing at all near the water's edge.

Tree Removal –

1. Where trees and stumps are removed in separate operations, trees may be used for commercial purposes such as lumber, firewood, or mulch.

2. Trees and stumps may be removed in one operation. This method leaves materials that can be useful in stream restoration and stabilization (e.g rootwads, vanes). may be used as a rootwads for streambank restoration work. Be certain that sufficient trunk is left for effective anchoring in the bank. Tops of trees should be removed and chipped for mulch.
3. Operating heavy equipment too close to trees will result in damage or loss due to soil disruption, compaction and trunk damage. It is recommended that all heavy equipment operations be limited to outside the drip line of all trees to be preserved. The drip line is the area from the trunk of the tree outward to a point at which there is no longer any overhanging vegetation.
4. In forested wetlands, shallow–rooted species are protected by each other from potential wind damage. Whenever trees are removed from a forested wetland, the possibility of blow downs or windthrow increases. Shallow rooted species are also protected by edge trees, which shield the prevailing wind side of the woodlot. It is helpful to leave as many edge trees as possible on the prevailing wind side of the cleared area.

Disposal Options –

Where possible, all stumps, roots, logs, brush, limbs, tops and other debris resulting from the clearing or thinning operation should be disposed of by processing through a chipping machine. The chips can then be utilized as mulch (see Mulching practice), as part of a site stabilization or final landscaping plan. Organic material may also be disposed of at an approved composting facility.

Note that treetops, stumps and field stone which are cleared and piled/windrowed in suitable areas can improve habitat for wildlife such as rabbits, raccoons, snakes, salamanders, toads and frogs.

Maintenance

Land clearing itself requires no maintenance except maintenance of the equipment used in the land clearing operation. Tree protection that utilizes fencing and signage should be maintained throughout the clearing stages. It is also important to maintain all other temporary and permanent practices that are used in conjunction with the land clearing to prevent soil erosion and sedimentation.

Common Problems / Concerns

Clearing of areas planned for preservation may occur and desirable species may be damaged, therefore preservation areas should be well marked.

During construction, naturally vegetated banks of stream and lakes may become destabilized. Clearly mark areas where natural vegetation must be maintained, and immediately implement stabilization plans of denuded areas.

As large areas are disturbed, site erosion potential drastically increases until cover is re-established. Establish temporary seedings as soon as clearing/grubbing and grading activities stop or become idle.

7.3 Tree and Natural Area Reservation

Description

Tree and natural area preservation insures that important vegetated areas existing on-site prior to development will survive the construction process. Tree protection areas prevent the losses and damages to trees that are common as a result of construction. This practice is useful to protect individual trees, and areas of forest or natural vegetation in stream corridors, or open space.

Conditions Where Practice Applies

This practice is applicable to any tree, forested or naturally vegetated area planned for long-term survival and subject to construction impacts. Existing trees provide valuable benefits during and after construction including: reduced erosion, reduced runoff rates and volume, reduced cooling costs, sound and visual barriers and higher property values.

Planning Considerations

Preservation of important natural areas must begin before the location of buildings, roads and utilities is determined. Early site planning should include delineating forested areas and significant trees and creating an inventory of the existing trees on-site. These should influence the placement of roads, buildings, and parking areas in the same manner as topography, streams and wetlands.

Tree Stand Delineation – Useful information for the delineation may include:

- Stands of trees to be preserved
- Individual trees of significance due to age, size, history, or aesthetic value

7.4 Construction Entrance

Description

A construction entrance is a stabilized pad of stone underlain with a geotextile and is used to reduce the amount of mud tracked off-site with construction traffic. Located at points of ingress/egress, the practice is used to reduce the amount of mud tracked off-site with construction traffic.

Conditions Where Practice Applies

A construction entrance is applicable where:

- Construction traffic leaves active construction areas and enters public roadways or areas unchecked by effective sediment controls;
- Areas where frequent vehicle and equipment access is expected and likely to contribute sediment to runoff, such as at the entrance to individual building lots.

Planning Considerations

Construction entrances address areas that contribute significant amounts of mud to runoff by providing a stable area for traffic. Although they allow some mud to be removed from construction vehicle tires before they enter a public roads, they should not be the only practice relied upon to manage off-site tracking. Since most mud is flung from tires as they reach higher speeds, restricting traffic to stabilized construction roads, entrances and away from muddy areas is necessary.

If a construction entrance is not sufficient to remove the majority of mud from wheels or there is an especially sensitive traffic situation on adjacent roads, wheel wash areas may be necessary. This requires an extended width pad to avoid conflicts with traffic, a supply of wash water and sufficient drainage to assure runoff is captured in a sediment pond or trap.

Proper installation of a construction entrance requires a geotextile and proper drainage to insure construction site runoff does not leave the site. The use of geotextile under the stone helps to prevent potholes from developing and will save the amount of stone needed during the life of the practice. Proper drainage may include culverts to direct water under the roadway or water bars to direct muddy water off the roadway toward sediment traps or ponds.

Design Criteria

The area of the entrance must be cleared of all vegetation, roots, and other objectionable material. Geotextile will then be placed the full width and length of the entrance.

Stone shall be placed to a depth of at least 6 inches. Roads subject to heavy duty loads should be increased to a minimum of 10 inches. Surface water shall be conveyed under the entrance, through culverts, or diverted via a water bars or mountable berms (minimum 5:1 slopes) so as to convey sediment laden runoff to sediment control practices or to allow clean water to pass by the entrance.

The stabilized construction entrance shall meet the specifications that follow.

Maintenance

The entrance shall be maintained in a condition that will prevent tracking or flow of mud onto public rights-of-way. This may require periodic top dressing with additional stone or the washing and reworking of existing stone as conditions demand and repair and/or cleanout of any structures used to trap sediment. All materials spilled, dropped, washed, or tracked from vehicles onto roadways or into storm drains must be removed immediately. The use of water trucks to remove materials dropped, washed, or tracked onto roadways will not be permitted under any circumstances.

Common Problems / Concerns

Mud is allowed to accumulate and is tracked on to public right-of-ways. The entrance and associated construction roads may need dressing with additional stone.

Soft depression areas develop in entrance area. Stone may not have been underlain with geotextile or insufficient stone base has been provided.

Appendix 1: Post-Construction Stormwater Design Examples

This appendix uses three hypothetical development sites in order to demonstrate the design of post-construction stormwater practices presented earlier in specifications.

Each practice example utilizes the existing and the proposed developed site and hydrologic characteristics to determine the sizing and configuration of each practice. The base requirements are presumed to be Ohio EPA's Construction General Permit post-construction requirements (detention of the water quality volume) and the detention of the critical storm (see the Critical Storm Method) from the development in order to prevent increases in downstream flooding and streambank erosion.

Each practice use the following steps to proceed through the design:

- Step 1 - Calculate Water Quality Volume (WQv)
- Step 2 - Compute Peak Discharge Requirements
- Step 3 - Identify Other Local Development Criteria/Requirements
- Step 4 - Determine if the Site and Soils Are Appropriate for the Practice
- Step 5 - Determine Practice Location and Preliminary Geometry to Meet Requirements
- Step 6 - Check Design to Ensure All Requirements Are Met

CONTENTS

Section A: Dry Extended Detention Basin

This design example illustrates the design of a dry extended detention stormwater basin that provides water quality treatment and peak discharge control within a highly impervious commercial development.

The layout of the North Country Automotive development is shown in Figure 1.A.1. The development site totals 7.7 acres draining to a single point on the north property line with no off-site watershed area. The site impervious area at completion of construction is estimated to be 5.3 acres. The example assumes that the local community has adopted the Critical Storm Method criteria to control peak discharges[1]. The pre-developed and post-developed site flow paths are shown in Figure 1.A.2. (limited to those used for calculations).

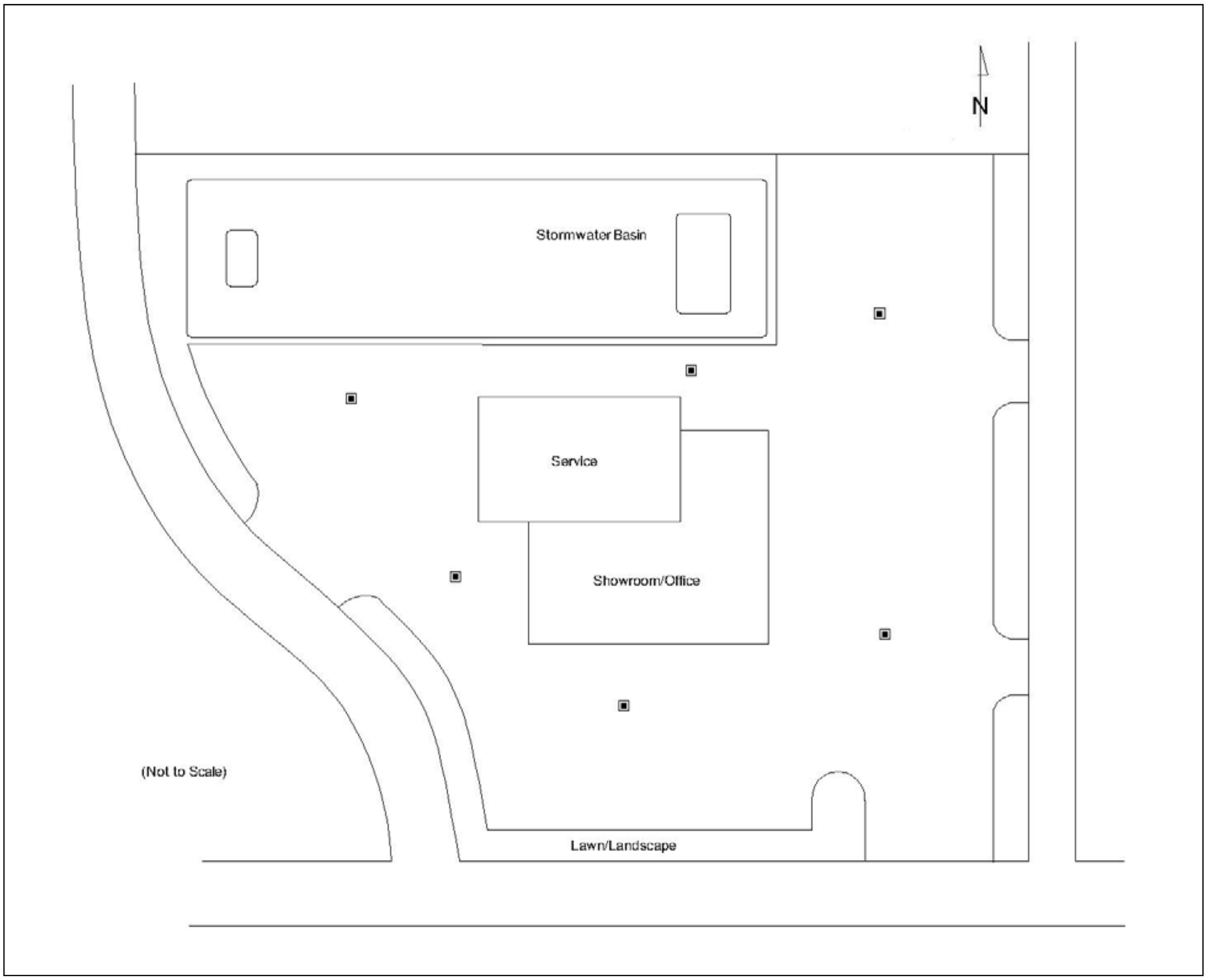


Figure 1.A.1. North Country Automotive Site Plan.

[1] The Critical Storm Method is a set of criteria for controlling the peak discharge of stormwater from large storm events (1 - 100 yr recurrence interval) recommended by ODNR-DSWC since 1980. See Goettemoeller, R.L., D.P. Hanselmann, and J.H. Bassett. 1980. Ohio Stormwater Control Guidebook. Ohio Department of Natural Resources, Division of Soil and Water Districts, p47.

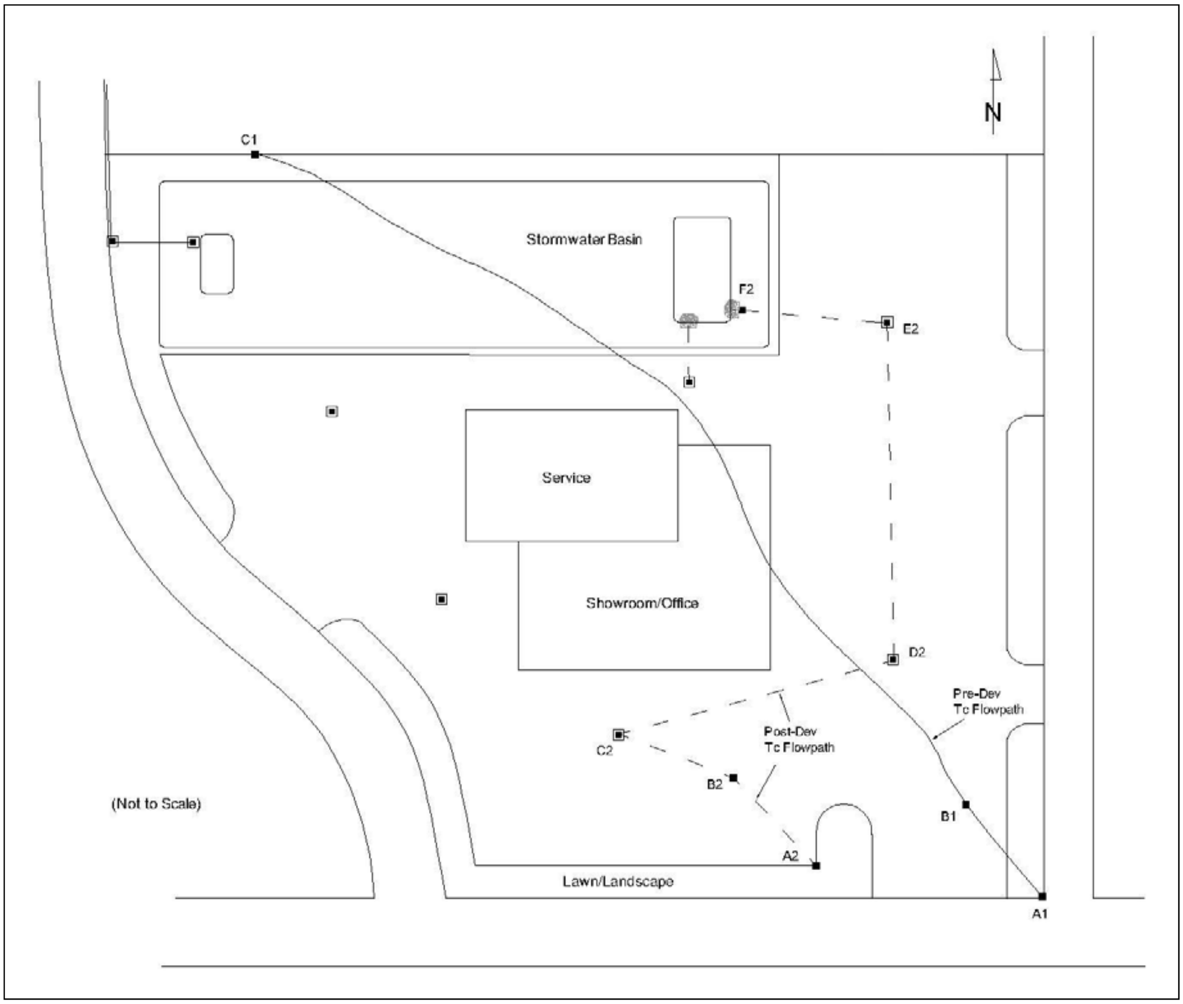


Figure 1.A.2. North Country Automotive Site Plan with pre-developed and post-developed flow paths.

<u>Site Data</u>
Total Drainage Area (A) = 7.7 ac
Estimated Impervious Area = 5.3 ac
Soil Types
Existing: 100% HSG-C
Proposed: 100% HSG-D

<u>Summary Hydrologic Data</u>
WQv = 0.26 ac-ft

	Pre	Post
CN =	70	96
Tc =	33.3 min	3.5 min

Calculation of Preliminary Stormwater Storage Volumes and Peak Discharges

Step 1 - Calculate Water Quality Volume (WQv)

The water quality volume (WQv) is a post-construction stormwater control requirement in Ohio (NPDES Storm Water Construction General Permit; OEPA, 2008). The WQv is determined according to the following equation:

$$WQv = C*P*A \qquad \text{Equation 1.A.1}$$

where:

C = runoff coefficient

P = 0.75 inch precipitation depth

A = drainage area

For open water, C = 1. At this site, the surface area of the detention basin at full WQv is estimated to be 0.4 acres.

For the remainder of the site, the runoff coefficient can either be selected from Table 1 of the NPDES Storm Water Permit, or calculated using the following equation:

$$C = 0.858i^3 - 0.78i^2 + 0.774i + 0.04 \qquad \text{Equation 1.A.2}$$

where i is the fraction of post-construction impervious surface.
For this site, total impervious area = 5.3 acres from a drainage area of 7.3 acres (i.e., total site drainage area - surface area of detention basin at full WQv).

$$i = 5.3/7.3 = 0.73 \qquad \text{Equation 1.A.3}$$

$$C = 0.858(0.73)^3 - 0.78(0.73)^2 + 0.774(0.73) + 0.04 = 0.52 \qquad \text{Equation 1.A.4}$$

Therefore, the WQv is:

$$WQv = [1.0*0.75 \text{ in}*0.4 \text{ ac} + 0.52*0.75 \text{ in}*7.3 \text{ ac}]*(1 \text{ ft}/12 \text{ in}) \qquad \text{Equation 1.A.5}$$

= <u>0.26 ac-ft</u>

$= 11,400 \text{ ft}^3$

Peak Discharge Summary
Project: North Country Automotive

	Cover Description	Soil Name	Hydrologic Group	Drainage Y/N	CN	Area (acres)
Existing Condition	Woods (good condition)	Ellsworth	C		70	7.7
	Existing Conditions				**70**	**7.7**
Proposed Condition	**Cover Description**	**Soil Name**	**Hydrologic Group**		**CN**	**Area (acres)**
	Impervious Area				98	5.3
	Open space (good condition)	Ellsworth	D		80	0.8
	Detention Basin				98	1.6
	Proposed Conditions				**96**	**7.7**

Table 1.A.1. Curve Number (CN) for existing (pre-developed) and proposed (post-developed) condition.

	Segment	Flow Type	Surface Cover	Mannings n	Length ft	Slope %	Velocity ft/s	Tt min
Existing Condition	A_1 to B_1	Overland - sheet	Woods - Light Underbrush	0.40	100	3		21.0
	B_1 to C_1	Overland - shallow conc	Woods - Light Underbrush	0.10	700	3.5	0.95	12.3
	Total	**Existing**			**800**			**33.3**
Proposed Condition	**Segment**	**Flow Type**	**Surface Cover**	**Mannings n**	**Length ft**	**Slope %**	**Velocity ft/s**	**Tt min**
	A_2 to B_2	Overland - sheet	Pavement	0.011	100	2.0		1.4
	B_2 to C_2	Overland - shallow conc	Pavement	0.025	100	2.0	2.9	0.6
	C_2 to D_2	Pipe - storm drain (12”)	Pipe	0.013	250	2.0	6.4	0.6
	D_2 to E_2	Pipe - storm drain (15”)	Pipe	0.013	300	2.0	7.4	0.7
	E_2 to F_2	Pipe - storm drain (18”)	Pipe	0.013	150	4.0	11.9	0.2
	Total	**Proposed**			**900**			**3.5**

Table 1.A..2. Time of Concentration (Tc) for existing ((pre-developed) and proposed (post-developed) condition.

RI years	P in	Q_{pre} in	Q_{post} in	Percent Increase Q	q_{pre} cfs	q_{post} cfs
1	2.00	0.24	1.57	554	0.86	21.3
2	2.40	0.41	1.96	378	1.8	26.1
5	2.98	0.70	2.53	261	3.7	33.1
10	3.47	0.99	3.02	205	5.5	39.0
25	4.17	1.44	3.71	158	8.5	47.3
50	4.76	1.86	4.29	131	11.2	54.3
100	5.38	2.32	4.91	112	14.1	61.6

Table 1.A.3. Summary runoff depth (Q) and peak discharge (q) for existing (pre-developed) and proposed (post-developed) conditions with critical storm (bold type).

Step 2 - Compute Peak Discharge Requirements

> Note: Peak discharge control is typically regulated through local entities (e.g. stormwater district, municipal, township or county governments). The state of Ohio recommends use of the Critical Storm Method[1] for peak discharge control, but the requirements will vary by community. Check local stormwater regulations to determine which peak discharge control method you must use.

This example uses the NRCS Curve Number Methodology to perform hydrologic calculations. TR-20, HEC-HMS or other software that uses NRCS procedures should provide similar results.

Tables 1-1 and 1-2 summarize the inputs necessary to determine the curve number (CN) and time of concentration (Tc) for the existing (pre-development) and proposed (post-development) conditions. Table 1-3 summarizes the existing and proposed runoff depths and peak discharges for the 1-year, 24-hr through 100-year, 24-hr rainfall events.

The *critical storm* is determined from the percent increase in runoff volume from the 1-year, 24-hr storm for the proposed (post-developed) conditions when compared to the existing (pre-developed) conditions (Goettemoeller et al., 1980):

$$Percent\ Increase = \frac{Q_{post} - Q_{pre}}{Q_{pre}} \, x\, 100 \qquad \text{Equation 1.A.6}$$

From Table 1-3, the percent increase in the 1-year, 24-hr runoff for the proposed development is:

$$Percent\ Increase = \frac{1.57 - 0.24}{0.24} \, x\ \ 100 = 554\% \qquad \text{Equation 1.A.7}$$

For an increase greater than 500%, the *critical storm* for peak discharge control is the 100-year, 24-hr event - i.e., the 100-year, 24-hr post-developed peak discharge must be less than the existing (pre-developed) 1-year, 24-hr peak discharge. These values are shown in bold type in Table 1.A.3.

Step 3 - Identify Other Local Development Criteria/Requirements

Commercial development in this community is subject to a 5% minimum landscaped area requirement - the proposed design meets this requirement. No additional setback or stormwater requirements were identified.

Step 4 - Determine if the Development Site and Soils Are Appropriate for the Use of a Dry Extended Detention Basin

The site drainage area is 7.7 acres, all of which is mapped as Ellsworth silt loam soil in the county soil survey. Ellsworth silt loam soils are suitable for creation of an extended detention basin with a wet forebay and permanent micropool. The subsoil is silty clay loam derived from glacial till and has slow permeability. Because the soil has slow permeability, there may be extended periods when the basin cannot be mowed. This subsoil is suitable material for construction of the embankment for the stormwater basin.

[1] The Critical Storm Method is a set of criteria for controlling the peak discharge of stormwater from large storm events (1 - 100 yr recurrence interval) recommended by ODNR-DSWC since 1980. See Goettemoeller, R.L., D.P. Hanselmann, and J.H. Bassett. 1980. Ohio Stormwater Control Guidebook. Ohio Department of Natural Resources, Division of Soil and Water Districts, p47.

Step 5 - Determine Pond Location and Develop Preliminary Geometry to Meet WQv and Peak Discharge Requirements

The proposed location of the stormwater basin (see Figure 1.A.2) reflects the best combination of characteristics (landscape position, access to outlet, minimized earth moving, appropriate soils, etc.) for siting the basin. Existing ground elevation at the proposed pond outlet is 935 MSL. An existing 24" storm sewer runs along the west edge of the property, with an invert elevation of 928 MSL at the proposed discharge point. [For more information on siting and planning an extended detention basin, see section 2.6.]

The basin will be designed to include a permanent micropool and wet forebay, an extended detention volume to protect water quality and stream channel stability, and storage necessary to control the peak discharge rate.

The NPDES Storm Water Permit (OEPA, 2008) specifies a dry extended detention basin include a water quality volume (WQv) with a drawdown time of 48 hours. The permit also requires an additional sediment storage volume equal to 20% of the WQv which, for a dry extended detention basin, should consist of a permanent micropool and forebay each sized at 10% of the WQv.

$$V_{micropool} \text{ and } V_{forebay} \geq 0.1{*}WQv = 0.1{*}0.26 \text{ ac-ft} = 0.026 \text{ ac-ft} = 1140 \text{ ft}^3 \qquad \text{Equation 1.A.8}$$

A plan view of the basin layout (Figure 1.A.3) reflects the following:

- extended detention water quality volume (WQv)
- a wet forebay with a minimum volume of 0.1*WQv and 3' depth
- permanent micropool with a minimum volume of 0.1*WQv and 4' depth
- a flow length to flow width ratio of 4:1, exceeding the 3:1 requirement
- positive slope (~0.8%) toward the outlet to facilitate surface drainage [Note: this is not enough slope to prevent extended periods of soil wetness.]
- 4:1 side slopes for safety and ease of maintenance
- an emergency spillway constructed in native soil (i.e., not in the constructed embankment)

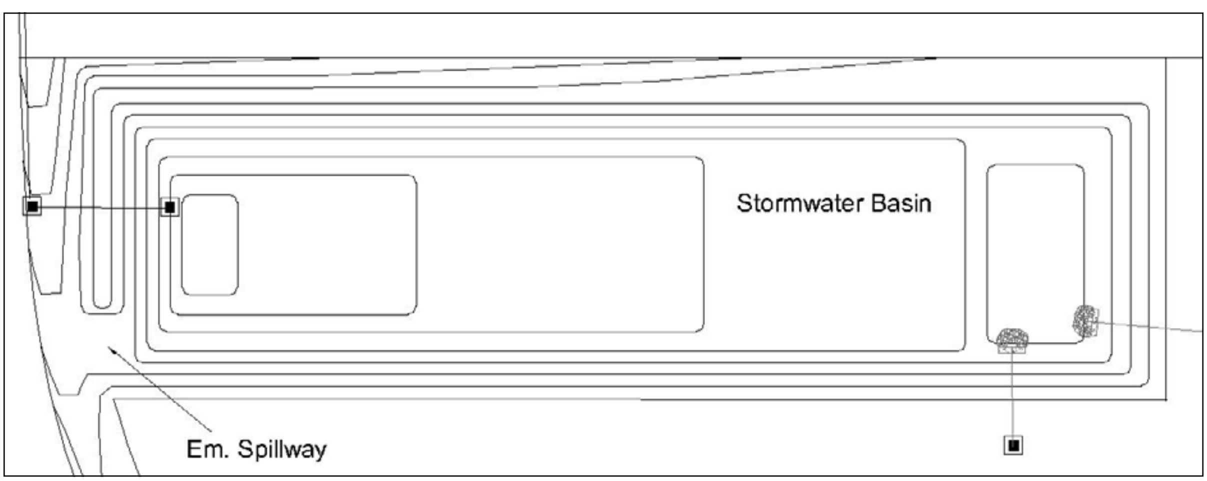


Figure 1.A.3. Plan View of the Basin Layout.

Set elevations for pond structures

- The pond bottom is set at elevation 930.0 and the riser invert is set at 929
- A manhole will be installed in the sewer main with a barrel invert (outfall) elevation at 928 ft

Establish permanent micropool and WQv water surface elevations

A stage-area-storage table (Table 1.A.4) reflects the geometry of the stormwater basin (Figure 1.A.3) designed to meet permanent micropool, forebay, extended detention (WQv) and peak discharge control requirements.

- The permanent micropool volume of 0.05 ac-ft (surface elevation 934.0) exceeds 0.1*WQv
- The extended detention water quality volume WQv of 0.26 ac-ft above the permanent micropool has a top elevation of approximately 935.5

Elevation MSL (ft)	Surface Area (acre)	Average Area (acre)	Incremental Depth (ft)	Incremental Volume (ac-ft)	Cumulative Volume (ac-ft)	Vol above Perm Pool (ac-ft)
930.0	0.004	-	-	-	-	-
934.0	0.02	0.01	4.0	0.05	0.05	-
934.5	0.10	0.06	0.5	0.03	0.08	0.03
935.0	0.22	0.16	0.5	0.08	0.16	0.11
935.5	0.40	0.31	0.5	0.15	0.31	0.26
936.0	0.52	0.39	0.5	0.20	0.48	0.43
937.0	1.05	0.76	1.0	0.76	1.24	1.19
938.0	1.25	1.15	1.0	1.15	2.39	2.34
939.0	1.36	1.30	1.0	1.30	3.69	3.64
940.0	1.47	1.41	1.0	1.41	5.10	5.05
941.0	1.58	1.53	1.0	1.53	6.63	6.58

Table 1.A.4. Stage-Area-Storage Information for Dry Extended Detention Basin.

Determine orifice size for 48-hour drawdown of WQv

The controlling parameters are WQv = 0.26 ac-ft, depth of WQv = 1.5 ft, and minimum drain time, T_d = 48 hours. Note that "the outlet structure for the post-construction BMP must not discharge more than the first half of the WQv in less than one-third of the drain time" (p22, NPDES Storm Water Construction General Permit; OEPA, 2008).

The average discharge rate for the WQv is:

$$Q_{avg} = \frac{WQv}{T_d} = \frac{(0.26\ ac \cdot ft)\left(\frac{43560\ ft^2}{1\ ac}\right)}{(48\ hr)\left(3600\ \frac{s}{hr}\right)} = 0.065\ cfs \qquad \text{Equation 1.A.9}$$

The discharge equation for an orifice is:

$$Q = ca\sqrt{2gh} \qquad \text{Equation 1.A.10}$$

By rearranging, we can estimate needed orifice area, as:

$$a = \frac{Q}{c\sqrt{2gh}} \qquad \text{Equation 1.A.11}$$

Using an orifice coefficient of c = 0.6, and average head, h = d/2 = (1.5 ft)/2 = 0.75 ft, the required orifice size is:

$$a = \frac{0.065\ \frac{ft^3}{s}}{0.6\sqrt{2(32.2\ \frac{ft}{s^2})(0.75\ ft)}} = 0.0156\ \ ft^2 \qquad \text{Equation 1.A.12}$$

resulting in an estimated orifice diameter of:

$$d = \left(\frac{4a}{3.14}\right)^{0.5} = \left[\frac{4(0.0156\,ft^2)}{3.14}\right]^{0.5} = 0.141\,ft \times \frac{12\,in}{1\ ft} = 1.7\,in \qquad \text{Equation 1.A.13}$$

This estimate is a good starting point for selecting the WQv orifice size, because it will always meet the two drawdown requirements: (1) the specified minimum drain time, T_d; and (2) the outlet must not discharge more than the first half of the WQv (or EDv) in less than one-third of the drain time. A larger or smaller orifice should be considered if it will help meet other environmental, cost, or maintenance goals, but must be tested for the two drawdown criteria.

Choosing the largest orifice size meeting the criteria lowers the likelihood of a clogged orifice and slightly lessens the storage volume required to meet the peak discharge requirement. In this situation, a 1.7" diameter orifice was the largest orifice that met the above two drawdown requirements (see Figure 1.A.4) and, thus, will be used as the extended detention (WQv) outlet.

Determine storage and outlet configuration to meet peak discharge requirements

As noted under Step 2, this dry detention basin is designed to meet the Critical Storm Method (CSM) for peak discharge control as well as the WQv requirement. Additional storage volume must be added that, with appropriate outlet design, will allow the basin to meet the following requirement:

- The peak rate of discharge from the post-construction 100-year, 24-hour event (the critical storm) must be released at the existing (pre-development) 1-year, 24-hour discharge rate

Proprietary stormwater modeling software was used to try a combination of stage-storage and outlet configurations until the critical storm requirement was satisfied while considering the following:

- use best practices outlined in Section 2.6 of the Rainwater and Land Development manual
- minimize cut/fill and grading

The resulting detention basin geometry is presented in Figure 1.A.3 and Table 1.A.4. The resulting outlet configuration is shown in Figure 1.A.5.

The outlet structure consists of a 3 ft by 3 ft concrete catch basin (e.g., ODOT No. 2-3) with invert at 929 MSL and 2.5'x2.5' iron grate at 938.1 MSL. The following comprise the outlets:

- 1.7" diameter extended detention water quality volume (WQv) orifice (invert 934.0 MSL) drilled into 6" PVC pipe using a non-clogging design
- 4.2" diameter orifice (invert 935.5 MSL) that controls release of the critical storm (100-year, 24-hour)
- 2.5'x2.5' iron grate (invert 938.1 MSL) for emergency overflow and maintenance access

The catch basin will be connected - using a 12" diameter conduit - to the 2' diameter storm sewer at the road along the west property boundary. A tailwater analysis was performed using the modeling software and the storm sewer's design elevation (invert at 928 MSL; 25-yr full pipe flow at 930 MSL) and assumed elevation for the 100-yr event (935 MSL).

In addition, this design includes an emergency spillway excavated into native soil with the following characteristics:

- Invert (crest) elevation of 938.5 MSL
- Level section length of 25 ft, weir length (i.e., crest width) of 25 ft
- Spillway crest perpendicular to flow
- With all other outlets blocked and starting from the permanent pool elevation of 934.0 MSL, will safely convey the 100-yr, 24-hr event with at least 1 ft freeboard below top of embankment

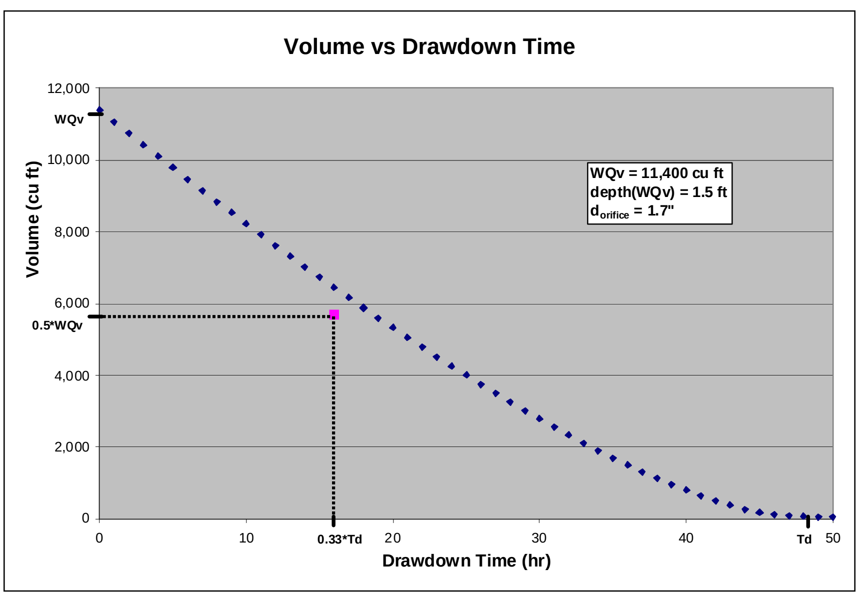


Figure 1.A.4. Dry Extended Detention Basin - Drawdown from Full WQv.

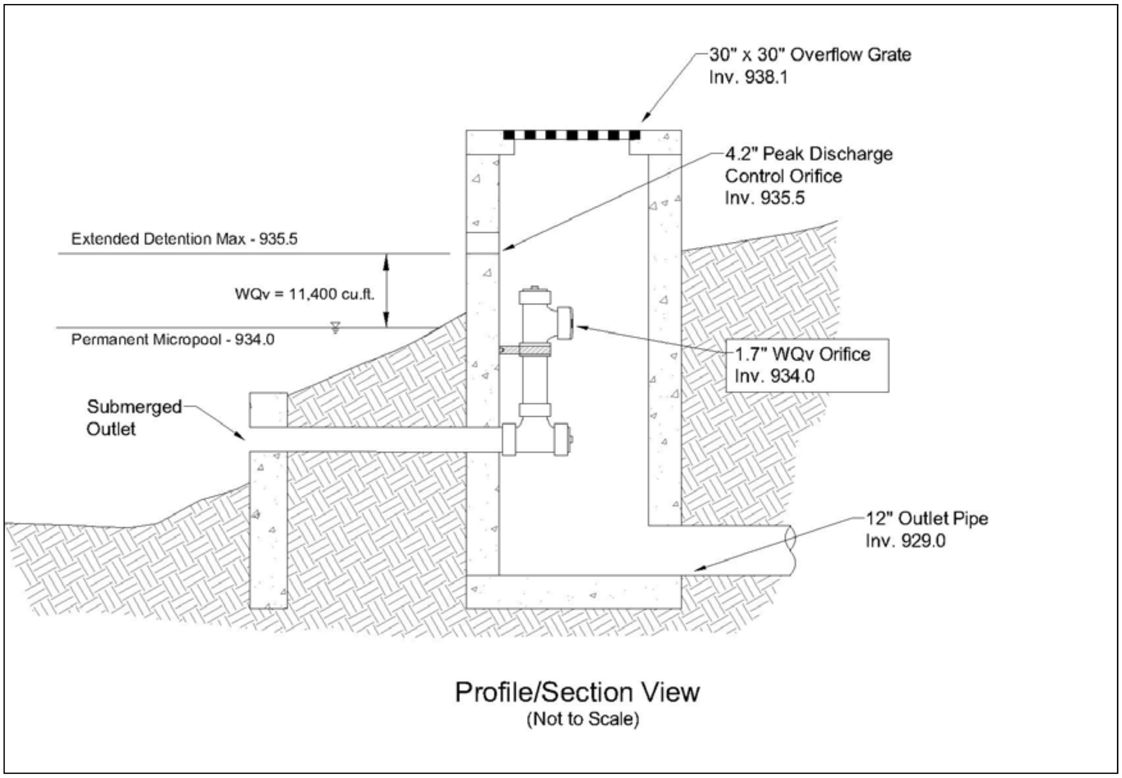


Figure 1.A.5. Outlet Configuration for Dry Extended Detention Basin (not to scale).

Step 6 - Check Design to Ensure All Requirements Are Met

From full WQv, check that WQv meets minimum 48 hour drain time, and discharges no more than 1/2 the water quality volume, 0.5*WQv (5050 ft^3), in the first 1/3 of the drain time, 0.33*T_d (16 hr). This requirement is met and illustrated in Figure 1.A.4.

Check peak discharge for all events (see Table 1.A.5).

RI years	P in	$q_{post\text{-}in}$ cfs	Allowed $q_{post\text{-}out}$ cfs	Estimated $q_{post\text{-}out}$ cfs
1	2.00	21.3	0.86	0.48
2	2.40	26.1	0.86	0.55
5	2.98	33.1	0.86	0.63
10	3.47	39.0	0.86	0.68
25	4.17	47.3	0.86	0.75
50	4.76	54.3	0.86	0.80
100	5.38	61.6	0.86	0.85

Table 1.A.5. Critical Storm Method Peak Discharge Check.

Section B: Wet Extended Detention Basin

This design example illustrates the design of a wet extended detention stormwater basin that provides water quality treatment and peak discharge control within a condominium development. This residential development will consist of 74 units of “active senior” living units and a well-equipped clubhouse for recreation, exercise and social functions. The layout of the development is shown in figure 1.B.1.

The development site consists of 24.2 acres having 10.2 acres of impervious area. An additional 8.5 acres of off-site area drains to the development site. The pre-developed site soils and flow paths are shown in Figure 1.B.2, while the post-developed flow paths (limited to that used for calculations) are shown in figure 1.B.3.

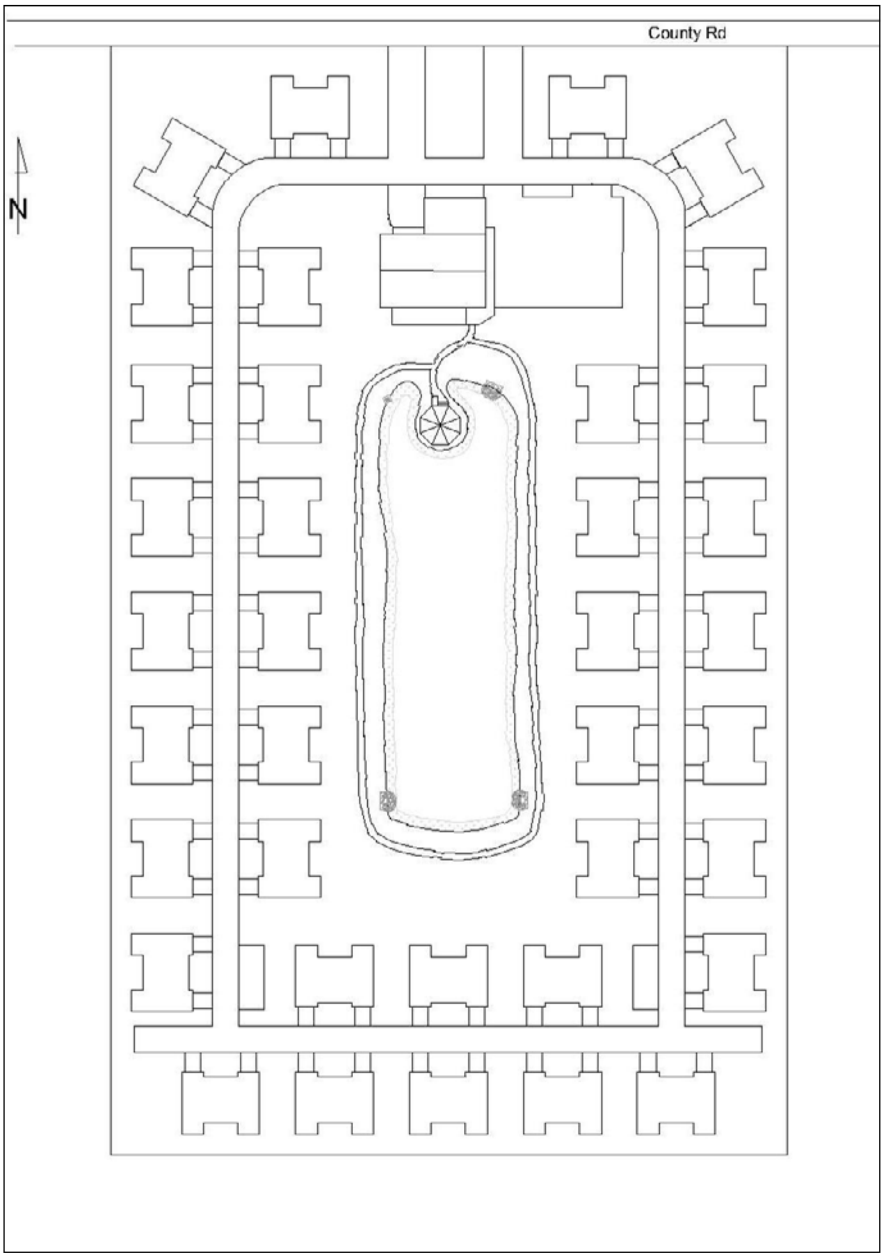


Figure 1.B.1. Autumn Knoll Subdivision Site Plan.

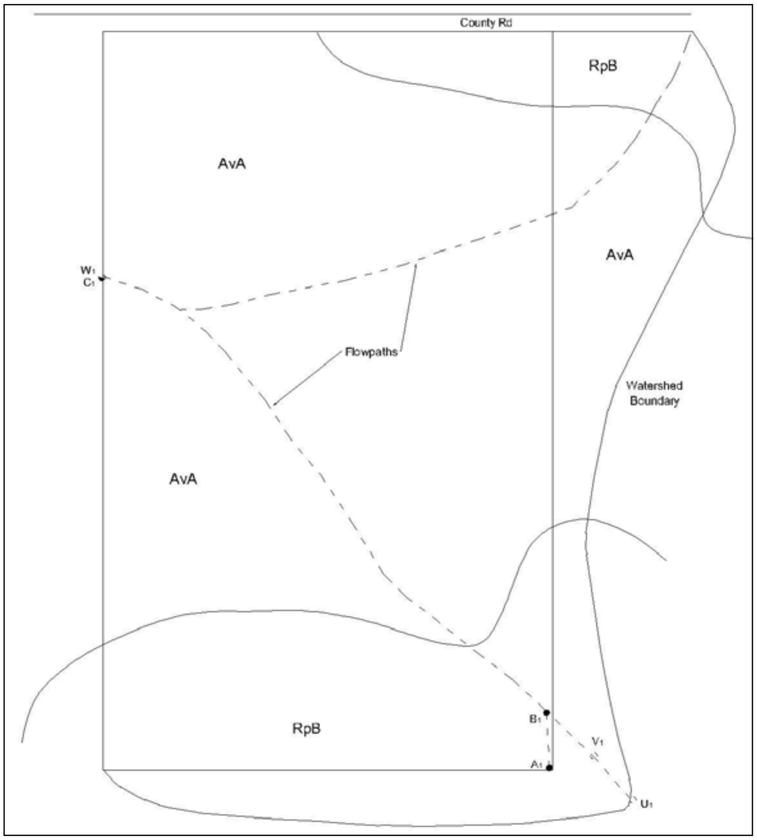


Figure 1.B.2. Pre-Development On-site and Off-site Soils and Drainage.

<u>Development Site Data</u>
Total On-Site Drainage Area (A) = 24.2 ac
Estimated Impervious Area = 10.2 ac
Soil Types
Existing: 25% HSG-C, 75% HSG-D
Proposed: 100% HSG-D

<u>Drainage from Off-site</u>
Off-site Drainage Area (A) = 8.5 ac
Estimated Impervious Area = 0 ac
Soil Types: 60% HSG-C, 40% HSG-D

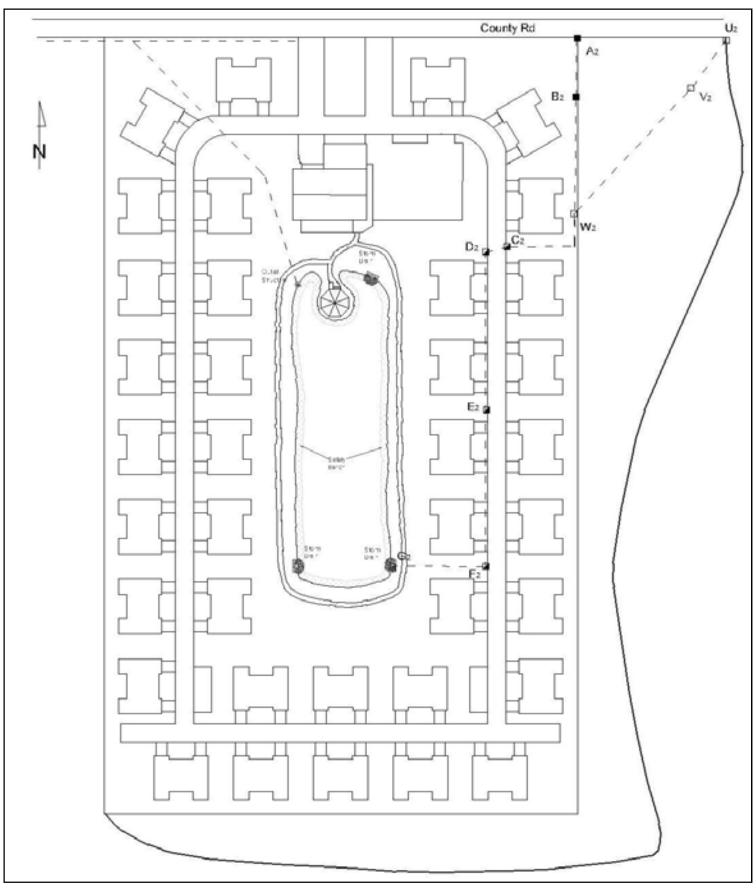


Figure 1.B.3. Post-Development On-site and Off-site Drainage.

<u>Summary Hydrologic Data</u>
WQv = 0.53 ac-ft
EDv = 0.40 ac-ft

	Pre	Post
CN(site) =	84	89
Tc(site) =	66 min	27 min
CN(offsite) =	83	83
Tc(offsite) =	71 min	30 min

Calculation of Preliminary Stormwater Storage Volumes and Peak Discharges

Step 1 - Calculate Water Quality Volume (WQv)

The water quality volume (WQv) is a post-construction stormwater control requirement in Ohio (NPDES Storm Water Construction General Permit; OEPA, 2008). The WQv is determined according to the following equation:

$$WQv = C*P*A \quad \text{Equation 1.B.1}$$

where:

C = runoff coefficient

P = 0.75 inch precipitation depth

A = drainage area

For open water, C = 1. At this site, the surface area of the detention basin at full extended detention volume is estimated to be 1.5 acres.

For the remainder of the site, the runoff coefficient can either be selected from Table 1 of the NPDES Storm Water Permit, or calculated using the following equation:

$$C = 0.858i^3 - 0.78i^2 + 0.774i + 0.04 \quad \text{Equation 1.B.2}$$

where i is the fraction of post-construction impervious surface.

For this site, total impervious area = 10.2 acres from a drainage area of 22.7 acres (i.e., total site drainage area - surface area of detention basin).

$$i = 10.2/22.7 = 0.45 \quad \text{Equation 1.B.3}$$

$$C = 0.858(0.45)^3 - 0.78(0.45)^2 + 0.774(0.45) + 0.04 = 0.31 \quad \text{Equation 1.B.4}$$

Therefore, the WQv is:

$$WQv = [1.0*0.75\text{ in}*1.5\text{ ac} + 0.31*0.75\text{ in}*22.7\text{ ac}]*(1\text{ ft}/12\text{ in}) \quad \text{Equation 1.B.5}$$

= <u>0.53 ac-ft</u>

= 23,200 ft^3

Step 2 - Compute Peak Discharge Requirements

> Note: Peak discharge control is typically regulated through local entities (e.g. stormwater district, municipal, township or county governments). The state of Ohio recommends use of the Critical Storm Method[1] for peak discharge control, but the requirements will vary by community. Check local stormwater regulations to determine which peak discharge control method you must use.

This example uses the NRCS Curve Number Methodology to perform hydrologic calculations. TR-20, HEC-HMS or other software that uses NRCS procedures should provide similar results.

Tables 1.B.1 and 1.B.2 (a and b) summarize the inputs necessary to determine the curve number (CN) and time of concentration (Tc) for the existing (pre-development) and proposed (post-development) conditions. Table 1.B.3 summarizes the existing and proposed runoff depths and peak discharges for the 1-year, 24-hr through 100-year, 24-hr rainfall events.

The *critical storm* is determined from the percent increase in runoff volume generated from the development site for the 1-year, 24-hr storm, comparing proposed (post-developed) conditions to the existing (pre-developed) conditions (Goettemoeller et al., 1980):

$$Percent\ \ Increase = \frac{Q_{post} - Q_{pre}}{Q_{pre}} x\, 100$$ Equation 1.B.6

Using data from Table 1.B.3, the percent increase in the 1-year, 24-hr runoff volume for the proposed development site is:

$$Percent\ \ \ Increase\ \ = \frac{1.38 - 1.05}{1.05} x\ \ 100\ \ = 31\,\%$$ Equation 1.B.7

For an increase greater than 20% but less than 50%, the *critical storm* for peak discharge control is the 5-year, 24-hr event - i.e., the stormwater detention facility must be designed such that the 5-year, 24-hr post-developed peak discharge does not exceed the existing (pre-developed) 1-year, 24-hr peak discharge. These values are shown in bold type in Table 1.B.3. In addition, the proposed peak discharge for the 10-year through 100-year events must not exceed the existing (pre-developed) discharge for like year events.

Step 3 - Identify Other Local Development Criteria/Requirements

The local subdivision regulations included lot size, lot width, road width and setback requirements that affected site layout. No additional stormwater requirements were identified.

Step 4 - Determine if the Development Site and Soils Are Appropriate for the Use of a Wet Extended Detention Basin

The site drainage area is 24.2 acres, all of which is mapped as Rossmoyne silt loam or Avonburg silt loam soil in the county soil survey[2]. The wet basin will be located in area mapped solely as Avonburg silt loam. Avonburg silt loam soils are suitable for creation of an extended detention basin with a permanent pool. The subsoil is clay loam derived from glacial till and has slow permeability. The constructed basin will lie predominantly below existing grade; a small amount of soil material will be used for construction of an embankment along the western and northern edges of the stormwater basin.

[1] The Critical Storm Method is a set of criteria for controlling the peak discharge of stormwater from large storm events (1 - 100 yr recurrence interval) recommended by ODNR-DSWC since 1980. See Goettemoeller, R.L., D.P. Hanselmann, and J.H. Bassett. 1980. Ohio Stormwater Control Guidebook. Ohio Department of Natural Resources, Division of Soil and Water Districts, p47.

[2] Note - Readily available county soil survey data provide excellent planning level information but typically are not accurate enough for engineering design. As part of site evaluation, a certified soil scientist should be contracted to perform an on-site soil investigation to provide an accurate representation of soil conditions and limitations at the development site.

Peak Discharge Summary
Project: Autumn Knoll Senior Living Residential Development

	Cover Description	Soil Name	Hydrologic Group	CN	Area (acres)
Existing Condition Site Only	Agriculture - Row Crop SR & CR	Rossmoyne	C	82	6.1
	Agriculture - Row Crop SR & CR	Avonburg	D	85	18.1
	Existing Conditions			**84**	**24.2**
	Cover Description	**Soil Name**	**Hydrologic Group**	**CN**	**Area (acres)**
Proposed Condition Site Only	Impervious Area			98	10.2
	Open space (good cond)	Rossmoyne/Avonburg	D	80	12.0
	Detention Basin			98	2.0
	Proposed Conditions			**89**	**24.2**
	Cover Description	**Soil Name**	**Hydrologic Group**	**CN**	**Area (acres)**
Off-site Condition	Agriculture - Row Crop SR & CR	Rossmoyne	C	82	5.1
	Agriculture - Row Crop SR & CR	Avonburg	D	85	3.4
	Existing Conditions			**83**	**8.5**

Table 1-B-1. Curve Number for existing and proposed conditions, as well as off-site area.

	Segment	Flow Type	Surface Cover	Mannings n	Length ft	Slope %	Veloctiy ft/s	Tt min
Existing Condition Site Only	A_1 to B_1	Overland - sheet	Cultivated Residue Cover >20%	0.17	100	1.2		14.0
	B_1 to C_1	Overland - shallow conc	Cultivated - Minimum Tillage	0.101	1130	0.5	0.4	52.3
	Total	**Existing**			**1230**			**66.3**
	Segment	**Flow Type**	**Surface Cover**	**Mannings n**	**Length ft**	**Slope %**	**Veloctiy ft/s**	**Tt min**
Proposed Condition Site Only	A_2 to B_2	Overland - sheet	Dense Grass	0.24	100	1.0		19.9
	B_2 to C_2	Overland - shallow conc	Grass Swale	0.050	380	1.0	1.6	3.9
	C_2 to D_2	Pipe - storm drain (18”)	Pipe	0.013	40	0.6	3.3	0.2
	D_2 to E_2	Pipe - storm drain (18”)	Pipe	0.013	260	0.3	3.3	1.3
	E_2 to F_2	Pipe - storm drain (24”)	Pipe	0.013	260	0.2	3.9	1.1
	F_2 to G_2	Pipe - storm drain (30”)	Pipe	0.013	170	0.3	4.6	0.7
	Total	**Proposed**			**910**			**27.0**

Table 1-B-2a. Time of Concentration (Tc) for existing and proposed conditions, as well as drainage from the off-site area.

Peak Discharge Summary (cont'd)
Project: Autumn Knoll Senior Living Residential Development

	Segment	Flow Type	Surface Cover	Mannings n	Length ft	Slope %	Veloctiy ft/s	Tt min
Existing Condition Off-site Condition	U_1 to V_1	Overland - sheet	Cultivated Residue Cover >20%	0.17	100	1.5		12.8
	V_1 to W_1	Overland - shallow conc	Cultivated - Minimum Tillage	0.101	1250	0.5	0.4	57.9
	Total	**Existing**			**1350**			**70.7**
Proposed Condition Off-site Condition	**Segment**	**Flow Type**	**Surface Cover**	**Mannings n**	**Length ft**	**Slope %**	**Veloctiy ft/s**	**Tt min**
	U_2 to V_2	Overland - sheet	Cultivated Residue Cover >20%	0.17	100	1.5		12.8
	V_2 to W_2	Overland - shallow conc	Cultivated - Minimum tillage	0.101	300	0.7	0.4	11.9
	W_2 to C_2	Overland - shallow conc	Grass Swale	0.050	180	1.0	1.6	1.9
	C_2 to D_2	Pipe - storm drain (18")	Pipe	0.013	40	0.3	3.3	0.2
	D_2 to E_2	Pipe - storm drain (18")	Pipe	0.013	260	0.3	3.3	1.3
	E_2 to F_2	Pipe - storm drain (24")	Pipe	0.013	260	0.3	3.9	1.1
	F_2 to G_2	Pipe - storm drain (30")	Pipe	0.013	170	0.3	4.6	0.6
	Total	**Proposed**			**1310**			**29.8**

Table 1-B-2b. Time of Concentration (Tc) for existing and proposed condition for off-site drainage only.

RI years	P in	Q_{pre} in	Q_{post} in	$Q_{off\text{-}site}$ in	Q_{pre} Ac-ft	Q_{post} Ac-ft	q_{pre} cfs	q_{post} cfs
1	2.42	1.05	1.38	1.00	2.8	3.5	16.1	38.1
2	2.90	1.43	1.81	1.37	3.9	4.6	22.4	50.4
5	3.56	1.99	2.41	1.91	5.4	6.2	31.4	67.7
10	4.07	2.43	2.89	2.35	6.6	7.5	38.6	81.3
25	4.77	3.06	3.55	2.97	8.3	9.3	48.6	100.0
50	5.32	3.56	4.08	3.47	9.6	10.7	56.6	114.7
100	5.89	4.09	4.63	3.99	11.1	12.2	65.0	130.0

Table 1-B-3. Summary runoff depth or volume (Q) and peak discharge (q) for existing (pre-developed) and proposed (post-developed) conditions with critical storm (bold type).

Step 5 - Determine Pond Location and Develop Preliminary Geometry to Meet WQv and Peak Discharge Requirements

The proposed location of the stormwater basin (see Figure 1.B.1) reflects several goals for this development project (including appropriate soils). In particular, the wet basin is considered the centerpiece of this development, with "waterfront condos" selling for a premium. The basin will also be over-excavated to provide fill material to raise the elevation of the condo structures. Existing ground elevation at the proposed pond outlet is 829 MSL. As part of this development, a storm sewer will be installed along the county road to convey site runoff to a receiving stream to the west. At the connection point, the storm sewer is 36" and has an invert elevation of 818.5 MSL. [For more information on siting and planning an extended detention basin, see section 2.6.]

The stormwater basin includes a permanent pool, an extended detention volume to protect water quality and stream channel stability, and storage necessary to control the peak discharge rate.

The NPDES Storm Water Permit (OEPA, 2008) specifies a wet extended detention basin must include both a permanent pool (designated PPv below) and an extended detention volume (EDv) equal to 75% of the water quality volume (WQv), with an EDv drawdown time of 24 hours. The permit also requires that the permanent pool contain an additional sediment storage volume equal to 20% of the WQv.

EDv = 0.75* WQv = 0.75*0.53 ac-ft = 0.40 ac-ft = 17,400 ft^3 Equation 1-B-8

PPv $\geq$ (0.75 + 0.2)*WQv = 0.95*0.53 ac-ft = 0.50 ac-ft = 22,000 ft^3 Equation 1-B-9

A plan view of the basin layout (Figure 1-B-4) reflects the following:

- extended detention volume equal to 0.75*WQv
- permanent pool with a minimum volume of (0.75+0.2)*WQv and 6 foot minimum depth
- 4:1 sideslopes for safety and ease of maintenance
- shallow, submerged wetland safety benches around the perimeter
- an emergency spillway constructed in native soil
- 3 storm drain outlets draining subareas within the development site (note: the length to width ratio for each of the two drains at the far end of the basin (draining approximately 90% of the site) exceeds 3:1, whereas the storm drain for the clubhouse/parking lot (drains approximately 10% of the site) was located on the other side of the gazebo peninsula to extend flow pathway to minimize short-cutting.

<u>Set elevations for pond structures</u>

- The pond bottom and riser invert are set at elevation 820 MSL
- A pond drain will be included to facilitate drawdown for maintenance or repairs.

<u>Establish permanent pool and WQv water surface elevations</u>

A stage-area-storage table (Table 1.B.4) reflects the geometry of the stormwater basin (Figure 1.B.4) designed to meet permanent pool, extended detention (EDv) and peak discharge control requirements.

- The permanent pool volume (PPv) of 4.5 ac-ft (surface elevation 826.0) exceeds 0.95*WQv
- The extended detention volume (EDv) of 0.40 ac-ft above the permanent pool has a top elevation of approximately 826.26.

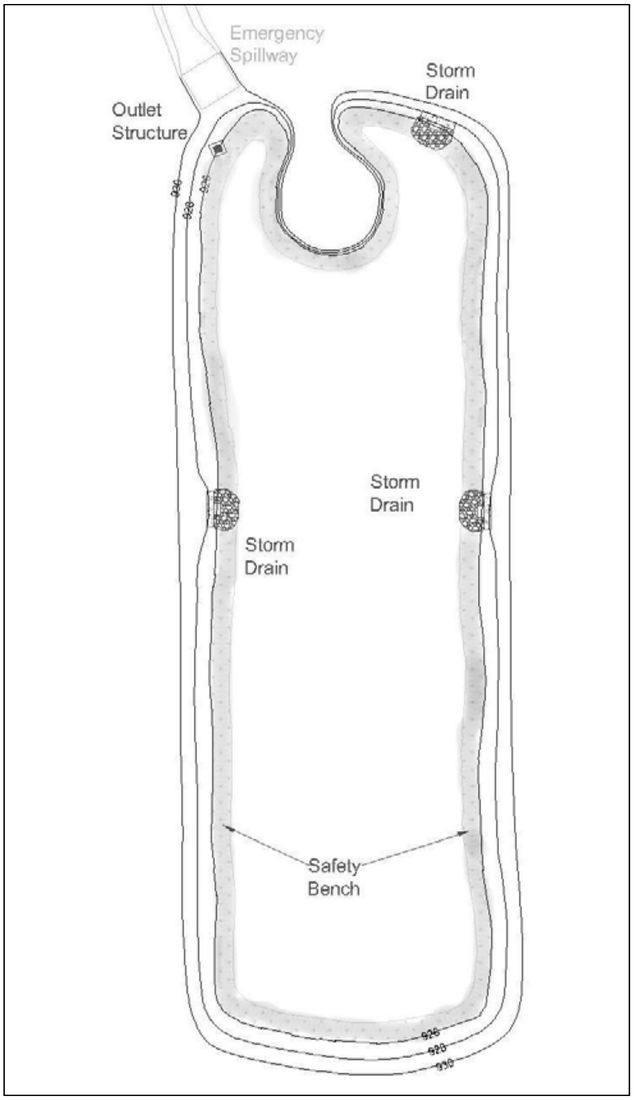


Figure 1.B.4. Preliminary Plan View of Wet Extended Detention Basin (not to scale).

Elevation MSL (ft)	Surface Area (acre)	Average Area (acre)	Incremental Depth (ft)	Incremental Volume (ac-ft)	Cumulative Volume (ac-ft)	Vol above Perm Pool (ac-ft)
820.0	0.004	-	-	-	-	-
826.0	1.50	0.75	6.0	4.5	4.5	-
826.3	1.53	1.51	0.3	0.4	4.9	0.4
827.0	1.61	1.57	0.7	1.2	6.1	1.6
828.0	1.72	1.66	1.0	1.7	7.8	3.3
829.0	1.83	1.78	1.0	1.8	9.6	5.1
830.0	1.98	1.90	1.0	1.9	11.5	7.0
831.0	2.20	2.09	1.0	2.1	13.6	9.1

Table 1.B.4. Stage-Area-Storage Information for Wet Extended Detention Basin.

Determine outlet geometry for 24-hour drawdown of EDv

The controlling parameters are EDv = 0.40 ac-ft, depth of EDv = 0.26 ft, and minimum drain time, T_d = 24 hours. Note that "the outlet structure for the post-construction BMP must not discharge more than the first half of the WQv in less than one-third of the drain time" (p22, NPDES Storm Water Construction General Permit; OEPA, 2008). This same criterion applies to the EDv.

When a wet detention basin has a large surface area (and thus the EDv depth is small), the designer has a wide variety of outlet options that will meet the two criteria above[3]. In this situation, combining a v-notch weir ("V" depth equal to or exceeding the depth of the EDv) with the peak discharge (critical storm) outlet, the designer was able to simplify and optimize the outlet while meeting both EDv criteria (see Figure 1.B.5) and peak discharge criteria (Table 1.B.5).

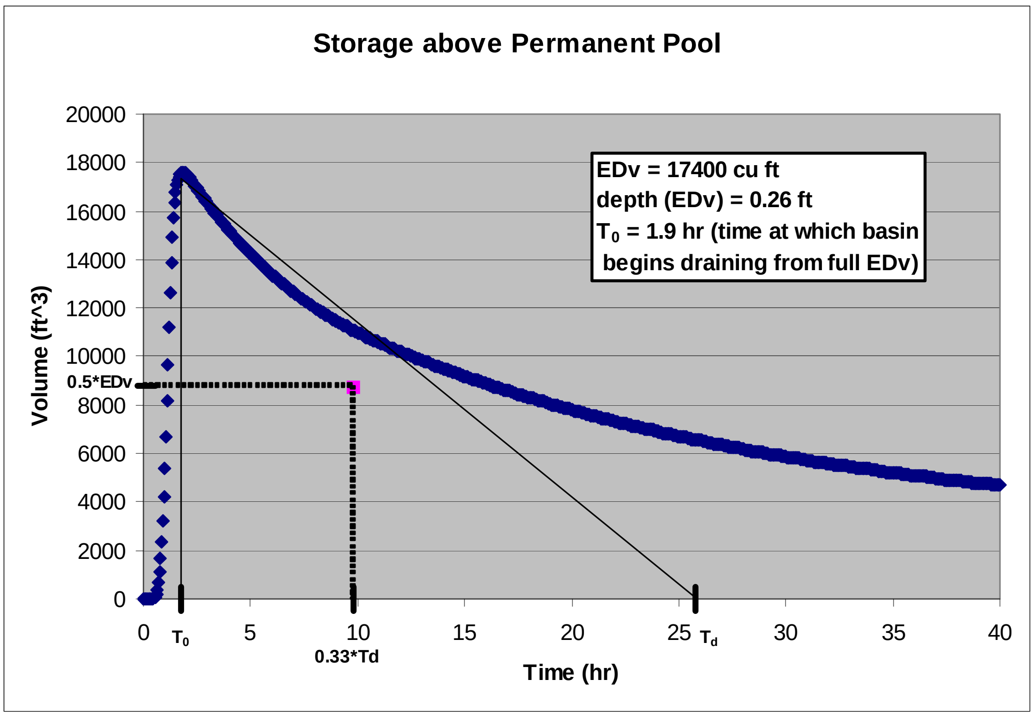


Figure 1.B.5. Wet Extended Detention Basin - Drawdown from Full EDv.

Determine storage and outlet configuration to meet peak discharge requirements

As noted under Step 2, this wet detention basin is designed to meet the Critical Storm Method (CSM) for peak discharge control as well as the WQv requirement. Storage volume must be incorporated that, with appropriate outlet design, will allow the basin to meet the following requirements:

- The peak rate of discharge from the post-construction 5-year, 24-hour event (the *critical storm*) must be less than the existing (pre-development) 1-year, 24-hour discharge peak rate
- The peak rate of discharge from the post-construction 10-, 25-, 50- and 100-year, 24-hour events must be no more than the existing (pre-development) discharge peak rate for the corresponding recurrence interval events

Proprietary stormwater modeling software was used to try a combination of stage-storage and outlet

[3] The methodology laid out in the Ohio NPDES Post Construction Q&A Document (Guidance Regarding Post-Construction Storm Water Management Requirements of Ohio; Ohio EPA, 2007) item #22.is a good starting point for selecting the EDv orifice size because it will always meet the two drawdown requirements: (1) the specified minimum drain time, Td; and (2) the outlet must not discharge more than the first half of the WQv (or EDv) in less than one-third of the drain time. A larger or smaller orifice should be considered if it will help meet other environmental, cost, or maintenance goals but must be tested for the two drawdown criteria.

configurations until the *critical storm* requirement was satisfied while considering the following:

- use best practices outlined in Section 2.6 of the Rainwater and Land Development manual
- optimize cut/fill and grading
- meet safety and aesthetic goals for the "lake" and waterfront properties

The resulting detention basin geometry is presented in Figure 1.B.4 and Table 1.B.4. The resulting outlet configuration is shown in Figure 1.B.6.

The outlet structure (see Figure 1.B.6) consists of a 3 ft by 3 ft concrete catch basin (e.g., ODOT No. 2-3) with invert at 820 MSL and 2.5'x2.5' iron grate at 828 MSL. The following comprise the outlets:

- A 30" wide orifice combined with a V-notch weir (invert 826 MSL) that controls release of both the extended detention volume (EDv) and the *critical storm* (5-year, 24-hour)
- 2.5'x2.5' iron grate (effective orifice area 490 sq. in.; invert 828 MSL) for maintenance access and to help manage discharge between the 10-yr and 100-yr, 24-hr events

The catch basin will be connected - using a 30" diameter conduit - to the 36" diameter storm sewer at the road along the north property boundary. A tailwater analysis was performed using the modeling software and the storm sewer's design elevation (invert at 818.5 MSL; 10-yr full pipe flow at 821.5 MSL) and assumed elevation for the 100-yr event (827.5 MSL).

In addition, this design includes an emergency spillway excavated into native soil with the following characteristics:

- Invert (crest) elevation of 829.2 MSL
- Spillway crest perpendicular to flow
- Level section length of 25 ft, weir length (i.e., width of crest perpendicular to flow) of 25 ft
- Exit channel flows to road ditch at elevation 827.5 MSL
- With all other outlets blocked and starting from the permanent pool elevation of 826 MSL, will safely convey the 100-yr, 24-hr event

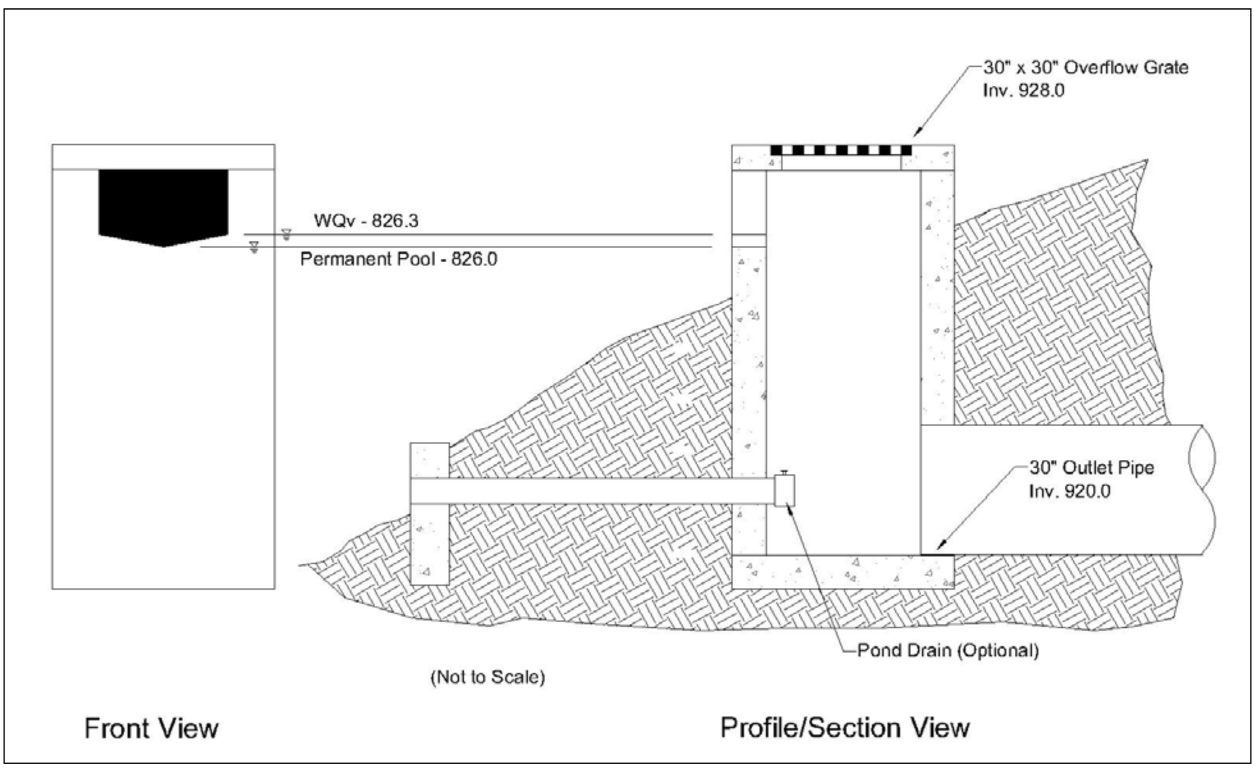


Figure 1.B.6. Outlet Configuration for Wet Extended Detention Basin (not to scale).

Step 6 - Check Design to Ensure All Requirements Are Met

From full EDv, check that EDv meets minimum 24 hour drain time, and discharges no more than 1/2 the extended detention volume, 0.5*EDv (9150 ft^3), in the first 1/3 of the drain time, 0.33*T_d (8 hr). This requirement is met and illustrated in Figure 1.A.5[4].

Check peak discharge for all events (see Table 1.B.5).

RI years	P in	$q_{post\text{-}in}$ cfs	Allowed $q_{post\text{-}out}$ cfs	Estimated $q_{post\text{-}out}$ cfs
1	2.42	38.1	16.1	7.2
2	2.90	50.4	16.1	11.1
5	3.56	67.7	16.1	15.9
10	4.07	81.3	38.6	24.0
25	4.77	100.0	48.6	32.3
50	5.32	114.7	56.6	37.3
100	5.89	130.0	65.0	41.8

Table 1.B.5. Critical Storm Method Peak Discharge Check.

[4] Note - Through trial and error, it was determined using a constant intensity 1-hour rainfall event of 0.83" depth in the hydrologic model would raise the water surface elevation of the wet basin to 826.26 providing a just-full EDv of 0.40 ac-ft (17,400 ft^3) above permanent pool, allowing evaluation of the drawdown from a full EDv (Figure 1.B.5). The depth of rainfall event necessary to just fill the EDv or WQv for other stormwater basins using CN methodology will vary based on watershed characteristics, pond geometry and outlet configuration, but can be determined through trial and error.

Section C: Extended Detention Wetland Basin

This design example illustrates the design of a extended detention wetland basin that provides water quality treatment and peak discharge control for a single family residential development, consisting of 101 residential lots on 46.0 acres (parcel and drainage area). The layout of the Beech Ridge subdivision is shown below in Figure 1.C.1.

The impervious area of the site at completion of construction is estimated to be 13.2 acres. The pre-developed site soils and flow paths are shown in Figure 1.C.2, while the post-developed flow path (limited to that used for calculations) is shown in Figure 1.C.3. This example assumes that the local community has adopted the Critical Storm Method criteria to control peak discharges.

Figure 1.C.1. Beech Ridge Subdivision Site Plan.

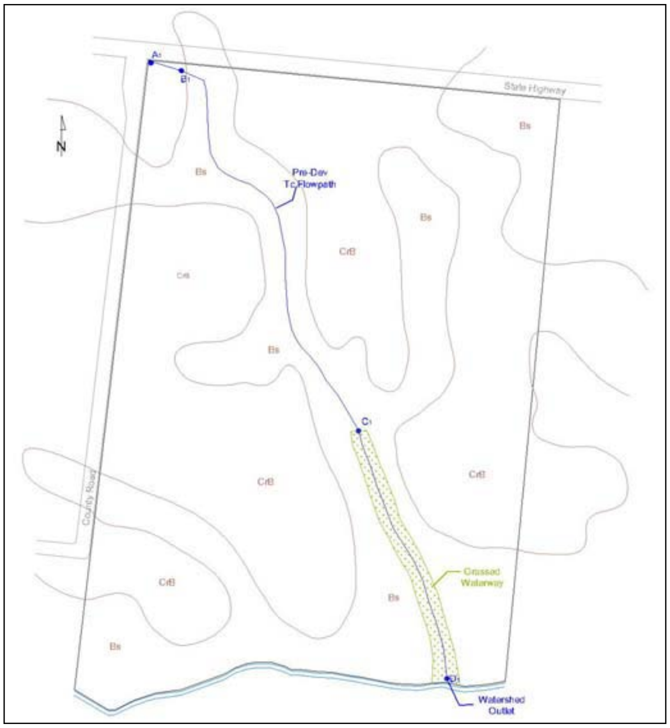


Figure 1.C.2.   Pre-Development and Soils and Flow Path.

Figure 1.C.3.   Post-Development Flow Path and Proposed Basin Location

<u>Site Data</u>
Zoning: Residential, 16,000 ft^2 minimum lot size (0.37 ac)
Total Drainage Area (A) = 46.0 ac
Estimated Impervious Area = 13.2 ac
Pre-Development Soil Types: 60% HSG-C, 40% HSG-B/D

<u>Summary Hydrologic Data</u>
WQv = 0.69 ac-ft

	Pre	Post
CN =	79	86
Tc =	69.5 min	25.4 min

Calculation of Preliminary Stormwater Storage Volumes and Peak Discharges

Step 1 - Calculate Water Quality Volume (WQv)

The water quality volume (WQv) is a post-construction stormwater control requirement in Ohio (OEPA-CGP, 2008). The WQv is determined according to the following equation:

$$WQv = C*P*A \qquad \text{Equation 1.C.1}$$

where:

C = runoff coefficient

P = 0.75 inch precipitation depth

A = drainage area

For open water, C = 1. At this site, the surface area of the detention basin at full WQv is estimated to be 1.2 acres.

For the remainder of the site, the runoff coefficient can either be selected from Table 1 of the NPDES Storm Water Permit, or calculated using the following equation:

$$C = 0.858i^3 - 0.78i^2 + 0.774i + 0.04 \qquad \text{Equation 1.C.2}$$

where i is the fraction of post-construction impervious surface.

For this site, total impervious area = 13.2 acres from a drainage area of 44.8 acres (i.e., total site drainage area - surface area of detention basin).

$$i = 13.2/44.8 = 0.295 \qquad \text{Equation 1.C.3}$$

$$C = 0.858(0.295)^3 - 0.78(0.295)^2 + 0.774(0.295) + 0.04 = 0.22 \qquad \text{Equation 1.C.4}$$

Therefore, the WQv is:

$$WQv = [(1.0 * 0.75 \text{ in} * 1.2 \text{ ac}) + (0.22 * 0.75 \text{ in} * 44.8 \text{ ac})] * (1 \text{ ft}/12 \text{ in}) \qquad \text{Equation 1.C.5}$$

$$= \underline{0.69 \text{ ac-ft}}$$

$$= 30{,}100 \text{ ft}^3$$

Step 2 - Compute Peak Discharge Requirements

> Note: Peak discharge control is typically regulated through local entities (e.g. stormwater district, municipal, township or county governments). The state of Ohio recommends use of the Critical Storm Method[1] for peak discharge control, but the requirements will vary by community. Check local stormwater regulations to determine which peak discharge control method you must use.

This example uses the SCS Curve Number Methodology to perform hydrologic calculations. TR-20, HEC-HMS or other software that uses SCS procedures should provide similar results.

Tables 1.C.1 and 1.C.2 summarize the inputs necessary to determine the curve number (CN) and time of concentration (Tc) for the existing (pre-development) and proposed (post-development) conditions. The property receives no runoff from off-site. Table 1.C.3 summarizes the existing and proposed runoff depths and peak discharges for the 1-year, 24-hr through 100-year, 24-hr rainfall events.

The *critical storm* is determined from the percent increase in runoff volume from the 1-year, 24-hr storm for the proposed (post-developed) conditions when compared to the existing (pre-developed) conditions (Goettemoeller et al., 1980):

$$Percent\ Increase = \frac{Q_{post} - Q_{pre}}{Q_{pre}}\, x\, 100$$

Equation 1.C.6

From Table 1.C.3, the percent increase in the 1-year, 24-hr runoff for the proposed development is:

$$Percent\ Increase = \frac{0.98 - 0.62}{0.62}\, x\ 100 = 58.1\%$$

Equation 1.C.7

For a percentage increase between 50% and 100%, the critical storm for peak discharge control is the 10-year, 24-hr event—that is, the 10-year, 24-hr post-developed peak discharge must be less than the existing (pre-developed) 1-year, 24-hr peak discharge. These values are shown in bold type in Table 1.C.3. In addition, the post-developed peak discharge from the 25, 50 and 100 year events must be less than the existing peak discharge for each of those events.

Step 3 - Identify Other Local Development Criteria/Requirements

This site is located within a community that has incorporated a stream corridor protection requirement (i.e., stream setback) in its subdivision regulations. Review of the regulations has determined that the stream protection zone at this site extends 100 ft from the ordinary high water mark of the adjacent stream. This protection zone is noted on the map in Figure 1.C.1. All construction activities, including the wetland stormwater basin and embankment, must be outside of the stream protection zone. Also note this stream protection area, since it does not drain to the detention facility, was excluded from the hydrologic analysis.

[1] The Critical Storm Method is a set of criteria for controlling the peak discharge of stormwater from large storm events (1 - 100 yr recurrence interval) recommended by ODNR-DSWC since 1980. See Goettemoeller, R.L., D.P. Hanselmann, and J.H. Bassett. 1980. Ohio Stormwater Control Guidebook. Ohio Department of Natural Resources, Division of Soil and Water Districts, p47.

Peak Discharge Summary
Project: Beech Ridge Subdivision

	Cover Description	Soil Name	Hydrologic Group	Drainage Y/N	CN	Area (acres)
Existing Condition Site Only	Row crop, SR + CR (good condition)	Crosby	C		82	27.6
	Row crop, SR + CR (good condition)	Brookston	B/D	Y	75	18.4
	Pre-development Conditions - All				**79**	**46.0**
	Cover Description	**Soil Name**	**Hydrologic Group**		**CN**	**Area (acres)**
Proposed Condition Site Only	Impervious Area				98	13.2
	Open space (good condition)	Crosby	D		80	18.5
	Open space (good condition)	Brookston	D		80	12.3
	Open Water				98	2.0
	Post-development Conditions - All				**86**	**46.0**

Table 1.C.1. Curve Number (CN) for existing (pre-developed) condition.

	Segment	Flow Type	Surface Cover	Mannings n	Length ft	Slope %	Velocity ft/s	Tt min
Existing Condition Site Only	A_1 to B_1	Overland - sheet	Min Tillage	0.17	100	1.2		14.8
	B_1 to C_1	Overland - shallow conc	Min Tillage	0.1	1250	0.75	0.44	47.6
	C_1 to D_1	Overland - shallow conc	Grassed waterway	0.05	750	1.2	1.8	7.1
	Total	**Pre-developed**			**2100**			**69.5**
	Segment	**Flow Type**	**Surface Cover**	**Mannings n**	**Length ft**	**Slope %**	**Velociiy ft/s**	**Tt min**
Proposed Condition Site Only	A_2 to B_2	Overland - sheet	Grass	0.24	100	1.5		17.8
	B_2 to C_2	Overland - shallow conc	Grassed waterway	0.05	160	1.5	2.0	1.3
	C_2 to D_2	Pipe - storm drain (15”)	Pipe	0.013	250	0.5	3.7	1.1
	D_2 to E_2	Pipe - storm drain (18”)	Pipe	0.013	270	0.5	4.2	1.1
	E_2 to F_2	Pipe - storm drain (24”)	Pipe	0.013	750	0.5	5.1	2.5
	F_2 to G_2	Pipe - storm drain (30”)	Pipe	0.013	450	0.5	5.9	1.3
	G_2 to H_2	Pipe - storm drain (36”)	Pipe	0.013	130	0.5	6.7	0.3
	Total	**Post-developed**			**2150**			**25.4**

Table 1.C.2. Time of Concentration (Tc) for existing (pre-developed) and proposed (post-developed) condition.

RI years	P in	Q_{pre} in/acre	Q_{post} in/acre	q_{pre} cfs	q_{post} cfs
1	2.17	0.62	0.98	12.1	42.8
2	2.59	0.90	1.32	18.3	58.0
5	3.18	1.32	1.82	28.1	80.4
10	3.67	1.70	2.25	36.7	99.4
25	4.35	2.25	2.87	49.2	126.2
50	4.91	2.72	3.38	59.9	148.4
100	5.50	3.24	3.94	71.3	171.9

Table 1.C.3. Summary runoff depth (Q) and peak discharge (q) for existing (pre-developed) and proposed (post-developed) condition with critical storm (bold type).

Step 4 - Determine if the Development Site and Soils Are Appropriate for the Use of an Extended Detention Wetland Basin

The site drainage area is 46.0 acres. Brookston and Crosby soils are suitable for creation of an extended detention wetland. The subsoil is silty clay loam derived from high-lime glacial till and has slow permeability. This subsoil is suitable material for construction of the embankment for the stormwater basin.

It is known that subsurface tiles currently drain the proposed property. All tiles need to be removed from the wetland basin site[2].

Step 5 - Determine Pond Location and Develop Preliminary Geometry to Meet WQv and Peak Discharge Requirements

The proposed location of the stormwater basin (see Figure 1.C.3) reflects the best combination of characteristics (landscape position, access to outlet, minimize earth moving, appropriate soils, etc.) for siting the basin. Existing ground elevation at the proposed pond outlet is 907 MSL. The invert of the receiving stream at the proposed discharge point is 896 MSL.

The basin will be designed to include a permanent pool, an extended detention volume equivalent to the WQv, and the storage necessary to control the peak discharge rate. [For more information on siting and planning a wetland basin, see section 2.6.]

An analysis of site hydrology (drainage area/wetland surface area ratio >>20, HSG-D soil with seasonal high water table, etc.) has determined that a permanent pool equivalent to the WQv (~0.69 ac-ft) up to 2 ac-ft should be sufficient to maintain basic wetland hydrology and function. In addition, an additional sediment storage volume equal to 20% of the WQv (0.2*WQv = 0.2*0.69 = 0.14 ac-ft) is added to the permanent pool with this volume concentrated in the forebay.

A preliminary plan view of the basin layout (Figure 1.C.4) reflects the following:

- permanent pool (includes forebay and outlet micropool) with a volume in excess of 1.2*WQv
- permanent pool forebay equal to 0.2* WQv and a minimum depth of 3 ft
- permanent micropool at outlet with a minimum depth of 3 ft
- total area of deep pools (including forebay and outlet mircropool) representing between 20 and 25 percent of total permanent pool surface area with deep pools interspersed through wetland to provide refugia and wetland function during drought periods - depth of deep pools should range between 18 and 36 (or more) inches[3]
- balance of permanent pool with average depth of 0.75 ft, and range of depths from 6” to 18”
- a low constructed peninsula, with an elevation approximately 1 ft above the extended detention (WQv) storage volume, to extend the flow path and minimize short-circuiting during the WQv event
- maximum 4:1 side slopes for safety and maintenance
- an emergency spillway constructed in native soil (i.e., not located in the constructed embankment)

Note: The high organic matter topsoil should be removed and stockpiled before excavation and construction of the wetland, and then replaced on peninsulas and benches.

[2] Functional drainage systems are essential for the productivity of agriculture in much of Ohio, and to prevent flooding of upgradient property. It is the responsibility of the developer to maintain drainage infrastructure (surface and subsurface drainage mains) disrupted by construction activities. As an example, if a subsurface tile main conveys water from upgradient properties, that main should be protected or re-routed to maintain the same drainage capacity.

[3] Recent guidance from North Carolina (Hunt et al, 2007) recommends “deep pools (including the forebay) should occupy between 20 and 25 percent of the total wetland surface area”. For most wetlands this will result in a permanent pool volume (ac-ft) between about 1.1 and 1.3 times the surface area (acres) of the permanent pool.

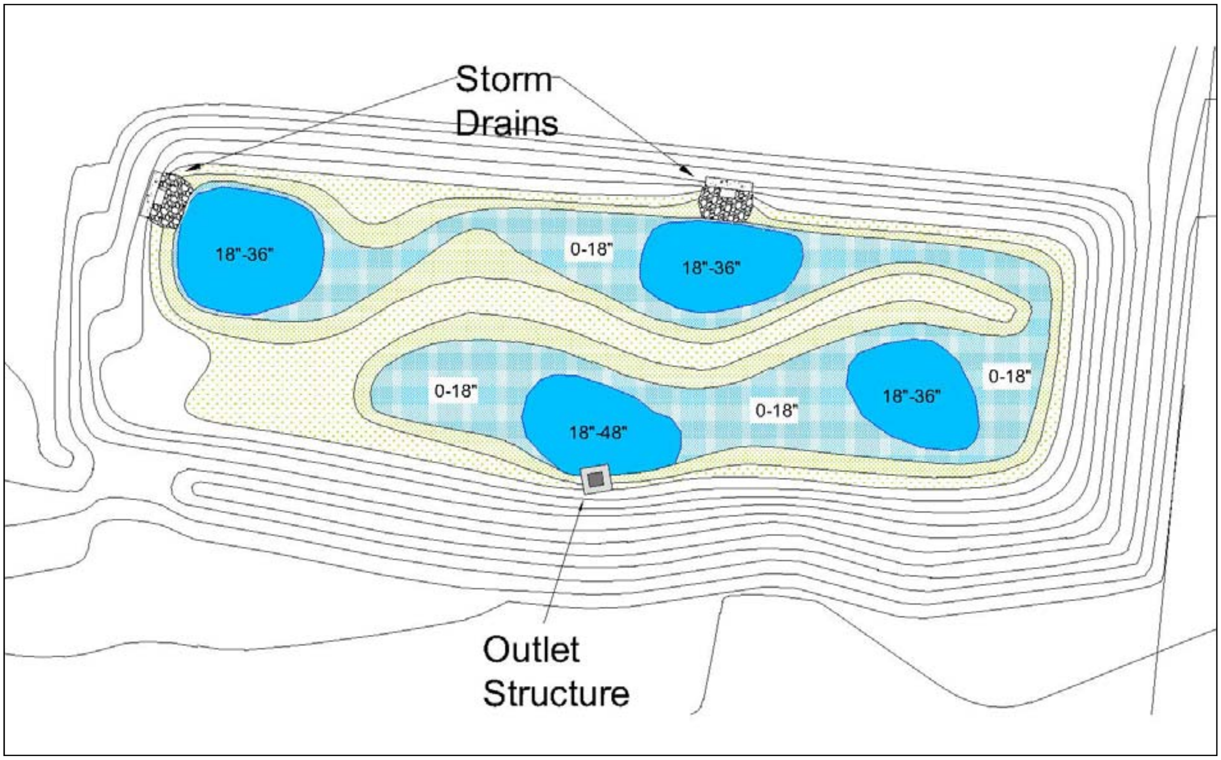


Figure 1.C.4. Preliminary Plan View of Wetland (not to scale).

Elevation MSL (ft)	Surface Area (acre)	Average Area (acre)	Incremental Depth (ft)	Incremental Volume (ac-ft)	Cumulative Volume (ac-ft)	Vol above Perm Pool (ac-ft)
900.0	0.08	-	-	-	-	-
902.5	0.16	0.12	2.5	0.30	0.30	-
903.0	0.24	0.20	0.5	0.10	0.40	-
903.5	0.48	0.36	0.5	0.18	0.58	-
904.0	0.74	0.61	0.5	0.31	0.89	-
904.7	1.24	0.99	0.7	0.69	1.58	0.69
905.0	1.50	1.37	0.3	0.41	1.99	1.10
906.0	1.60	1.55	1.0	1.55	3.54	2.65
907.0	1.71	1.65	1.0	1.65	5.19	4.30
908.0	1.82	1.76	1.0	1.76	6.95	6.07
909.0	1.93	1.87	1.0	1.87	8.82	7.94
910.0	2.05	1.99	1.0	1.99	10.81	9.93
911.0	2.20	2.13	1.0	2.13	12.94	12.05

Table 1.C.4. Stage-Area-Storage Information for Wetland Basin.

Set elevations for pond structures

- The basin bottom is set at elevation 900.0
- To allow gravity flow for the pond drain, set the riser invert at 898.0
- The outfall at the receiving stream has invert elevation 896.5

Set permanent pool and WQv water surface elevations

A stage-area-storage table (Table 1-4) reflects geometry of the stormwater wetland basin (Figure 1-3) designed to meet permanent pool, extended detention WQv and peak discharge control requirements.

- To meet NPDES Construction Stormwater Permit minimums, the permanent pool, surface elevation 904.0, is sized to exceed 1.2*WQv = 1.2*0.69 ac-ft = 0.83 ac-ft (see footnote below)
- The extended detention WQv of 0.69 ac-ft above permanent pool has a top elevation of approximately 904.7

Calculate required orifice size for 24-hour drawdown of WQv

The controlling parameters are WQv = 0.69 ac-ft, depth of WQv = 0.7 ft, and minimum drain time, t_d, of 24 hours. Note that "the outlet structure for the post-construction BMP must not discharge more than the first half of the WQv in less than one-third of the drain time" (p22, NPDES Storm Water Construction General Permit).

The average discharge rate for the WQv is:

$$Q_{avg} = \frac{WQv}{t_d} = \frac{(0.69\ ac \cdot ft)\left(\frac{43560\ ft^2}{1\ ac}\right)}{(24\ hr)\left(3600\ \frac{s}{hr}\right)} = 0.35\ cfs$$

Equation 1-C-8

The discharge equation for an orifice is:

$$Q = ca\sqrt{2gh}$$

Equation 1-C-9

By rearranging, we can estimate needed orifice area:

$$a = \frac{Q}{c\sqrt{2gh}}$$

Equation 1-C-10

Using an orifice coefficient, c = 0.6, and average head, h = d/2 = (0.7 ft)/2 = 0.35 ft, the required orifice size is:

$$a = \frac{0.35\ \frac{ft^3}{s}}{0.6\sqrt{2(32.2\ \frac{ft}{s^2})(0.35\ ft)}} = 0.12\ ft^2$$

Equation 1-C-11

Resulting in an orifice diameter of:

$$d = \left(\frac{4a}{3.14}\right)^{0.5} = \left[\frac{4(0.12\ ft^2)}{3.14}\right]^{0.5} = 0.39\ ft \times \frac{12}{1\ ft} = 4.7\ in$$

Equation 1-C-12

This estimate is a good starting point for selecting the WQv or EDv orifice size because it will always meet the two drawdown requirements: (1) the specified minimum drain time, Td; and (2) the outlet must discharge less than the first half of the WQv in the first one-third of the drain time (8 hours in this case). A larger or smaller orifice should be considered if it will help meet other environmental, cost, or maintenance goals but must be tested for the two drawdown criteria. In this situation, trial and error showed that a 6.0" diameter orifice will meet the above two drawdown requirements (see Figure 1.C.4) and will be used as the WQv outlet.

Determine storage and outlet configuration to meet peak discharge requirements

As noted under Step 2, this wetland basin is designed to meet the Critical Storm Method (CSM) for peak discharge control as well as the WQv requirement. Additional storage volume must be added that, with appropriate outlet design, will allow the basin to meet the following requirements:

- The peak rate of discharge from the post-construction 10-year, 24-hour event (the *critical storm*) must be released at the existing (pre-development) 1-year, 24-hour discharge rate
- The peak rate of discharge from the post-construction 25-, 50- and 100-year, 24-hour events must be released at the existing (pre-development) discharge rate for the corresponding recurrence interval events

Proprietary stormwater modeling software was used to try a combination of stage-storage and outlet configurations until the Table 1.C.5 requirements were satisfied while considering the following:

- maximize wetland function
- minimize the "footprint" of the basin
- optimize cut/fill

RI years	P in	$q_{post-in}$ cfs	Allowed $q_{post-out}$ cfs
1	2.17	42.7	12.1
2	2.59	57.9	12.1
5	3.18	80.2	12.1
10	3.67	99.1	12.1
25	4.35	125.9	49.2
50	4.91	148.0	59.9
100	5.50	171.4	71.3

Table 1.C.5. Critical Storm Method Peak Discharge Requirements.

The resulting wetland basin geometry is presented in Figure 1.C.3 and Table 1.C.4. The resulting outlet configuration is shown in Figure 1.C.5.

The outlet structure consists of a 4 ft by 4 ft concrete catch basin (e.g., ODOT No 2-4) with invert at 899 MSL and 3.7'x3.7' iron grate at 908.33 MSL. The following comprise the outlets:

- 36" barrel outlet with invert at 899 MSL
- 6.0" extended detention (WQv) orifice (invert 904 MSL) with submerged entrance
- Two (2) 12" diameter orifices (invert 904.7 MSL) that control release of the *critical storm* (10-year, 24-hour)
- Four 36" L x 9" H rectangular orifices (invert 907.25 MSL) and 3.7'x3.7' iron grate (invert 908.33) with 868 in^2 of clear opening area that control release of the 25- through 100-year, 24-hour events

In addition, this design includes an emergency spillway excavated into native soil that has the following characteristics:

- Invert (crest) elevation of 909.3 MSL
- Level section length of 25 ft, weir length (i.e., crest width) of 30 ft
- Spillway crest perpendicular to flow
- Exit channel aligned with level section well beyond downstream toe of dam, and a 4 percent slope
- With all other outlets blocked and starting from the permanent pool elevation of 904 MSL, will safely convey the 100-yr, 24-hr event with 1 ft freeboard from top of embankment

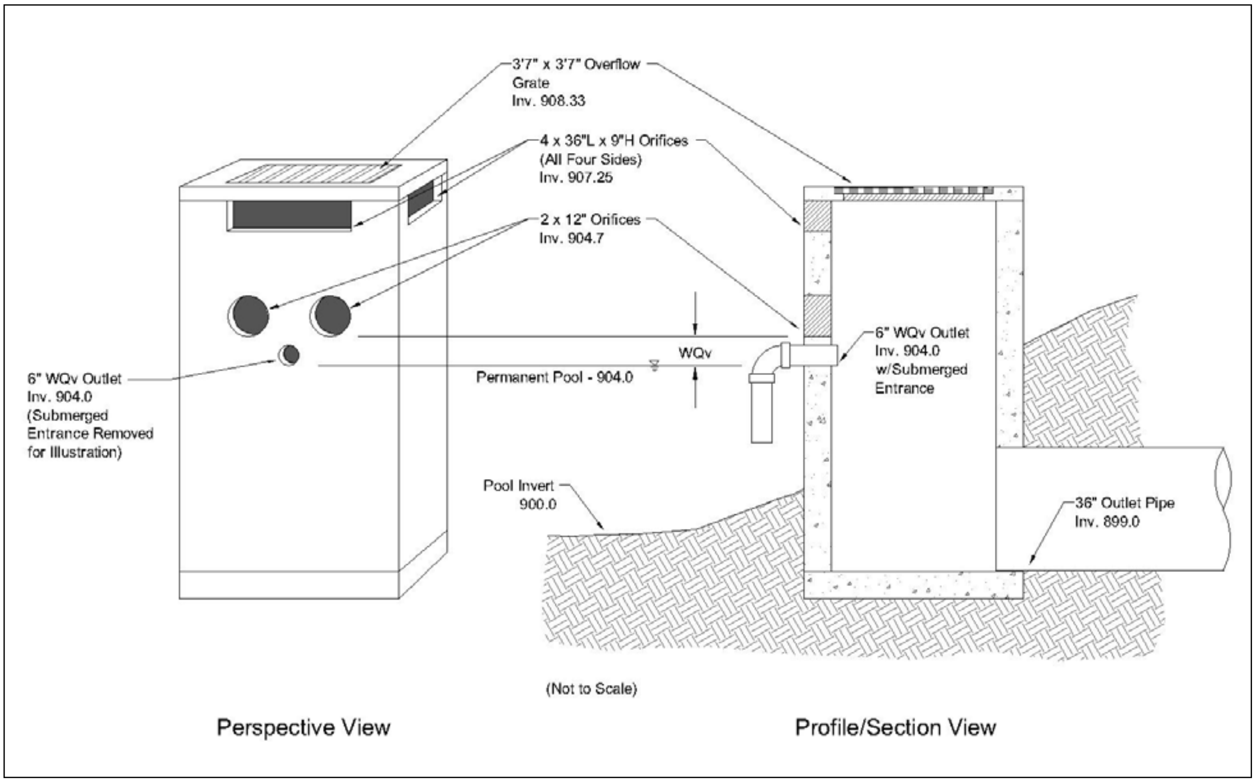


Figure 1.C.5. Outlet configuration for Wetland Basin (not to scale).

Step 6 - Check Design to Ensure All Requirements Are Met

From "brimfull", check that WQv meets minimum 24 hour drain time, and discharges no more than 1/2 the water quality volume, 0.5*WQv (= 15,550 ft^3), in the first 1/3 of the drain time, 0.33*T_d (8 hr). Figure 1.C.4 shows the wetland basin meets this requirement.

Check peak discharge for all events. Table 1.C.6 shows the wetland basin meets the peak discharge requirements.

RI years	P in	$q_{post-in}$ cfs	Allowed $q_{post-out}$ cfs	Estimated $q_{post-out}$ cfs
1	2.17	42.7	12.1	5.1
2	2.59	57.9	12.1	7.6
5	3.18	80.2	12.1	10.2
10	3.67	99.1	12.1	12.1
25	4.35	125.9	49.2	26.5
50	4.91	148.0	59.9	42.7
100	5.50	171.4	71.3	62.4

Table 1.C.6. Critical Storm Method Peak Discharge Check.

References:

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Appendix 4: Overview of Stream/Wetland Regulations

According to the federal Clean Water Act, anyone who wishes to discharge dredged or fill material into the waters of the U.S., must obtain a Section 404 permit from the U.S. Army Corps of Engineers (Corps) and a Section 401 Water Quality Certification (WQC) from the state. The Corps will also require a Section 10 permit if the fill is located in a navigable water.

Section 404 Permits

Section 404 of the Clean Water Act requires approval prior to discharging dredged or fill material into the waters of the United States. Typical activities requiring Section 404 permits are:

- Depositing of fill or dredged material in waters of the U.S. or adjacent wetlands.
- Site development fill for residential, commercial, or recreational developments.
- Construction of revetments, groins, breakwaters, levees, dams, dikes, and weirs.
- Placement of riprap and road fills.

Waters of the United States

Waters of the United States includes essentially all surface waters such as all navigable waters and their tributaries, all interstate waters and their tributaries, all wetlands adjacent to these waters, and all impoundments of these waters.

"Wetlands" are areas characterized by growth of wetland vegetation (bulrush, cattails, rushes, sedges, willows, pickleweed) where the soil is saturated during a portion of the growing season or the surface is flooded during some part of most years. Wetlands generally include swamps, marshes, bogs, and similar areas.

The landward regulatory limit for non-tidal waters (in the absence of adjacent wetlands) is the *ordinary high water mark*. The ordinary high water mark is the line on the shores established by the fluctuations of water and indicated by physical characteristics such as:

- a clear natural line impressed on the bank;
- shelving;
- changes in the character of the soil;
- destruction of terrestrial vegetation;
- the presence of litter and debris;
- or other appropriate means that consider the characteristics of the surrounding areas.

Navigable Waters

Navigable waters are defined as waters that have been used in the past, are now used, or are susceptible to use as a means to transport interstate or foreign commerce up to the head of navigation. Section 10 and/or Section 404 permits are required for construction activities in these waters. A complete list is available from the Army Corps of Engineers District Office.

Section 401 Water Quality Certification

The 401 Water Quality Certification (WQC) is required from Ohio EPA prior to the Corps approval of a Section 404 permit. Essentially these permitting processes work in tandem and include much of the same information. The 401 WQC requires an anti-degradation analysis investigating three alternatives: preferred alternative, minimum degradation alternative, and non-degradation alternative. The preferred alternative would include impacts that allow the applicant to develop the property in a preferred development plan. The minimum degradation alternative must minimize the impacts to water resources while still allowing the project to be constructed in an economically viable fashion. The non-degradation alternative must propose a site development plan, which includes zero water quality impacts to surface waters of the state. 401 WQC will be reviewed with varying levels of scrutiny based on the amount of impacts and quality of water resources. For example, a public need must be demonstrated to allow for impacts to category 3 wetlands, but this review is not necessary for impacts to category 1 or 2 wetlands. Fees are required at the time of application and for review of Ohio 401 Water Quality Certification applications.

Generally there are two types of 404 permits applicable to most entities in the State of Ohio, depending on the amount of linear feet of stream, linear feet of shoreline or acres of wetland proposed to be impacted. The types of permits include Individual Permits and Nationwide Permits. Additionally the Ohio Department of Transportation has been issued a Regional General Permits for for transportation projects meeting prescribed conditions.

Individual Permits

Individual permits are issued following a full public interest review of an individual application for a Department of the Army permit. A public notice is distributed to all known interested persons. After evaluating all comments and information received, final decision on the application is made.

The permit decision is generally based on the outcome of a public interest balancing process where the benefits of the project are balanced against the detriments. A permit will be granted unless the proposal is found to be contrary to the public interest. Processing time may take at least 120 days, although the Army Corps of Engineers is allowed up to 1 year to process permits.

Individual permits will require an individual 401 WQC from the Ohio EPA including a full antidegradation review.

Nationwide Permits

A nationwide permit is a form of general permit, which authorizes a category of activities throughout the nation. Nationwide Permits are for certain types of projects that are similar in nature and cause minimal degradation to waters of the state. These permits substantially expedite the permitting process. These permits are valid only if the conditions applicable to the permits are met. If the conditions cannot be met, an individual permit will be required.

Ohio EPA has pre-granted Section 401 Water Quality Certifications to Nationwide Permits with general and specific conditions. To determine if your project qualifies for Nationwide Permit coverage, or requires an individual Section 401 WQC from Ohio EPA, applicants should contact the Corps first to discuss the project.

Isolated Wetland Permits

In January 2001, the United States Supreme Court Decision in the case of Solid Waste Agency of

Northern Cook County (SWANCC) v. United States Army Corps of Engineers stated that the Corps did not have authority to regulate isolated wetlands under Section 404 of the Clean Water Act. Prior to that ruling, the Corps regulated activities in all streams and wetlands through the issuance of 404 Permits.

As a result of this decision, the Ohio EPA adopted emergency rules in April of 2001 to establish a state-permitting program, but these rules were effective for only ninety days. On July 17, 2001, Governor Bob Taft signed House Bill 231 into law. The bill establishes a permanent permitting process for isolated wetlands. The Army Corps of Engineers has maintained the authority to determine whether a wetland is isolated. If the determination by the Corps is that the wetland is isolated, applicants must contact the Ohio EPA to determine the correct level of Isolated Wetland Permit. More information can be found on the Ohio EPA web site.

Pre-Application Consultation

Applicants are encouraged to contact the Corps of Engineers and the Ohio EPA for proposed work in waters of the state. By discussing all information prior to application submittal, the application will be processed more efficiently. If an applicant is unsure if an application is required, the Corps will provide an official determination as to the need for a Department of the Army permit upon request.

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Contacts for Ohio EPA and Army Corps of Engineers

Ohio EPA, 401 Water Quality Certifications

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Columbus, Ohio 43216-1049
(614) 644-2013
Tom.Harcarik@epa.state.oh.us
www.epa.state.oh.us/dsw/401/401section.html

For Questions about the Ohio Rapid Assessment Method, contact Brian Gara at the above address or at (614) 836-8787, Brian.Gara@epa.state.oh.us

U.S. Army Corps of Engineers, Section 404 Permits

Buffalo District

1776 Niagara Street
Buffalo, NY 14207-3199
FAX (716) 879-4310
http://www.lrb.usace.army.mil/orgs/reg/index.htm

Louisville District

Attention: Regulatory Branch, OP-F
P.O. Box 59
Louisville, KY 40201-0059
Phone: (502) 315-6733
http://www.lrl.usace.army.mil/

Huntington District

502 Eighth Street
Huntington, WV 25701
(604) 529-5210
http://www.lrh.usace.army.mil/or/permits/

Pittsburgh District

William S. Moorhead Federal Building
1000 Liberty Avenue
Pittsburgh, PA 15222
http://www.lrp.usace.army.mil/or/or-f/permits.htm

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Definitions Associated with 404/401 and Isolated Wetland Permits

Isolated Wetlands – per OAC 3745-1-50

"Hydrologically isolated wetlands" means those wetlands which;

(1) Have no surface water connection to a surface water of the state;

(2) Are outside of, and not contiguous to, any one hundred year "floodplain" as that term is defined in this rule; and

(3) Have no contiguous hydric soil between the wetland and any surface water of the state.

Ordinary High Water Mark

Landward regulatory limit for non-tidal waters (in the absence of adjacent wetlands). Line on the shores or river banks established by the fluctuations of water and indicated by physical characteristics such as:

- a clear natural line impressed on the bank;
- shelving;
- changes in the character of the soil;
- destruction of terrestrial vegetation;
- the presence of litter and debris;
- or other appropriate means that consider the characteristics of the surrounding areas.

Navigable Waters

Waters that have been used in the past, are now used, or are susceptible to use as a means to transport interstate or foreign commerce up to the head of navigation. Section 10 and/or Section 404 permits are required for construction activities in these waters. A complete list is available in the Corps District Offices.

Ohio Rapid Assessment Method (ORAM)

Method which allows an applicant to assess the quality of the wetland without completing detailed vegetative or hydrologic analyses. The outcome of applying this method is the categorization of wetlands as either Category 1, 2 or 3. The Ohio EPA reviews categorization of wetlands. The current manual is ORAM Version 5.0.

Waters of the State

"Surface waters of the state" or "water bodies" mean all streams, lakes, reservoirs, ponds, marshes, wetlands or other waterways which are situated wholly or partially within the boundaries of the state, except those private waters which do not combine or effect a junction with natural surface or underground waters. Waters defined as sewerage system, treatment works or disposal system in section 6111.01 of the Revised Code are not included.

Wetlands – Effective 12/30/2002, Per OAC 3745-1-02

"Wetlands" means those areas that are inundated or saturated by surface or ground water at a frequency and duration that are sufficient to support, and that under normal circumstances do support, a prevalence of vegetation typically adapted for life in saturated soil conditions. "Wetlands" includes swamps, marshes, bogs, and similar areas that are delineated in accordance with the 1987 United States Army Corps of Engineers wetland delineation manual and any other procedures and requirements adopted by the United States army corps of engineers for delineating wetlands.

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Wetland Categories – Per, OAC 3745-1-54(C)

Category 1 Wetlands

a) support minimal wildlife habitat, and minimal hydrological and recreational functions as determined by an appropriate wetland evaluation methodology acceptable to the director. Wetlands assigned to category 1 do not provide critical habitat for threatened or endangered species or contain rare, threatened or endangered species.

b) Wetlands assigned to category 1 may be typified by some or all of the following characteristics: hydrologic isolation, low species diversity, a predominance of non-native species (greater than fifty per cent areal cover for vegetative species), no significant habitat or wildlife use, and limited potential to achieve beneficial wetland functions.

c) may include, but are not limited to, wetlands that are acidic ponds created or excavated on mined lands without a connection to other surface waters throughout the year and that have little or no vegetation and wetlands that are hydrologically isolated and comprised of vegetation that is dominated (greater than eighty per cent areal cover) by species including, but not limited to: *Lythrum salicaria; Phalaris arundinacea;* and *Phragmites australis.*

Category 2 Wetlands

a) support moderate wildlife habitat, or hydrological or recreational functions as determined by an appropriate wetland evaluation methodology acceptable to the director or his authorized representative.

b) may include, but are not limited to: wetlands dominated by native species but generally without the presence of, or habitat for, rare, threatened or endangered species; and wetlands which are degraded but have a reasonable potential for reestablishing lost wetland functions.

Category 3 Wetlands

a) support superior habitat, or hydrological or recreational functions as determined by an appropriate wetland evaluation methodology acceptable to the director or his authorized representative.

b) may be typified by some or all of the following characteristics: high levels of diversity, a high proportion of native species, or high functional values.

c) may include, but are not limited to: wetlands which contain or provide habitat for threatened or endangered species; high quality forested wetlands, including old growth forested wetlands, and mature forested riparian wetlands; vernal pools; and wetlands which are scarce regionally and/or statewide including, but not limited to, bogs and fens.

Wetland Delineation

Process utilized to determine the areal extent and boundaries of a jurisdictional wetland. Currently, the 1987 U.S. Army Corps of Engineers Manual details the procedures for performing a wetland delineation. The results of a wetland delineation are reviewed by the Army Corps of Engineers.

Appendix 6: Soils with Greatest Potential Use for Infiltration

The following is a list of Ohio soil map units that have the optimum soil characteristics for infiltration. These soils have a natural drainage class that is well drained, depths to bedrock over 100 inches and an appropriate saturated hydraulic conductivity between the depths of 20-60 inches.

Saturated hydraulic conductivity is the amount of water that would move vertically through a unit of saturated soil per unit time under hydraulic gradient, described in the National Soil Survey Handbook (http://soils.usda.gov/technical/hand¬book/contents/part618p3.html#50).

Of course, site designers must realize that soil map units are not enough information for design. For example, soil map units may have inclusions of other soils types. Some soil map units not listed here, such as the urban soil complex, are too disturbed to characterize consistently in this format. Also note that some of the following soils may have other limitations such as steep slopes and although they may receive water well, these may limit the potential of siting an infiltration practice at the particular area. Therefore on-site measures of soil and site characteristics are always recommended.

The following tables are listed by county, showing the soil map units that meet the 3 criteria for 'greatest potential use' for infiltration. If a county is not listed, that county does not have soil map units that meet all of the criteria. Assistance to identify the potential for infiltration of soils not included in this table can be obtained by contacting soil scientists with the ODNR-Division of Soil & Water Conservation or USDA-Natural Resources Conservation Service.

Adams County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
EkB	Elkinsville silt loam, 1 to 6 percent slopes	4,642	1.2
Ge	Gessie loam, frequently flooded	2,762	0.7
	Total	7,404	2.0

Allen County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
KnA	Knoxdale silt loam, 0 to 2 percent slopes, occasionally flooded	2,750	1.1
	Total	2,750	1.1

Ashland County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
WuB	Wooster-Riddles silt loams, 2 to 6 percent slopes	---	*
WuC	Wooster-Riddles silt loams, 6 to 12 percent slopes	---	*
WuD2	Wooster-Riddles silt loams, 12 to 18 percent slopes, eroded	---	*
	Total	0	0.0

* Less than 0.1 percent.

Ashtabula County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Ch	Chagrin silt loam	2,319	0.5
Sm	Steep land, loamy	6,428	1.4
	Total	8,747	1.9

Athens County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Cd	Chagrin loam, rarely flooded	2,090	0.6
Cg	Chagrin silt loam, frequently flooded	14,250	4.4
CmC	Clymer loam, 8 to 15 percent slopes	1,000	0.3
HcA	Hackers silt loam, 0 to 3 percent slopes	820	0.3
Mp	Moshannon silt loam, frequently flooded	470	0.1
PaB	Parke silt loam, 2 to 6 percent slopes	450	0.1
RcC	Richland loam, 8 to 15 percent slopes	310	*
RcD	Richland loam, 15 to 25 percent slopes	3,640	1.1
	Total	23,030	7.1

* Less than 0.1 percent.

Auglaize County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Gn	Genesee silt loam, occasionally flooded	2,890	1.1
	Total	2,890	1.1

Belmont County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
As	Ashton silt loam, occasionally flooded	319	*
Cf	Chagrin loam, occasionally flooded	2	*
Cg	Chagrin silt loam, occasionally flooded	2,240	0.6
DuB	Duncannon-Urban land complex, 0 to 15 percent slopes	514	0.1
No	Nolin variant silt loam, occasionally flooded	1,813	0.5
Nu	Nolin variant-Urban land complex	291	*
RcC	Richland loam, 8 to 15 percent slopes	684	0.2
RcD	Richland loam, 15 to 25 percent slopes	2,658	0.8
RcE	Richland moderately stony loam, 25 to 40 percent slopes	788	0.2
RkC	Richland channery loam, 8 to 15 percent slopes	39	*
RkD	Richland channery loam, 15 to 25 percent slopes	292	*
	Total	9,640	2.8

* Less than 0.1 percent.

Brown County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
EkB	Elkinsville silt loam, 2 to 6 percent slopes	1,802	0.6
EkC2	Elkinsville silt loam, 6 to 12 percent slopes, eroded	433	0.1
Ge	Genesee silt loam, occasionally flooded	5,982	1.9
Gn	Gessie loam, frequently flooded	10	*
HyC3	Hickory clay loam, 6 to 12 percent slopes, severely eroded	6	*
Ju	Jules silt loam, frequently flooded	755	0.2
	Total	8,988	2.8

* Less than 0.1 percent.

Butler County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Gn	Genesee loam	9,161	3.0
Go	Genesee-Urban land complex	1,727	0.6
UnA	Uniontown silt loam, 0 to 2 percent slopes	278	*
UnB	Uniontown silt loam, 2 to 6 percent slopes	612	0.2
	Total	11,778	3.9

* Less than 0.1 percent.

Carroll County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Ek	Elkinsville silt loam, rarely flooded	419	0.2
WrC	Westmoreland silt loam, 6 to 15 percent slopes	33	*
	Total	452	0.2

* Less than 0.1 percent.

Champaign County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Go	Genesee silt loam, till substratum, occasionally flooded	84	*
RuA	Rush silt loam, 0 to 2 percent slopes	97	*
WsA	Wea silt loam, 0 to 3 percent slopes	838	0.3
	Total	1,019	0.4

* Less than 0.1 percent.

Clark County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Ge	Genesee silt loam, till substratum, rarely flooded	246	*
Gn	Genesee silt loam, till substratum, occasionally flooded	1,637	0.6
Rn	Ross silt loam, occasionally flooded	2,385	0.9
RuA	Rush silt loam, 0 to 2 percent slope	1,756	0.7
	Total	6,024	2.3

* Less than 0.1 percent.

Clermont County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Gn	Genesee silt loam	8,144	2.8
HkD2	Hickory loam, 12 to 18 percent slopes, moderately eroded	1,442	0.5
HkF2	Hickory loam, 18 to 35 percent slopes, moderately eroded	7,602	2.6
HlG3	Hickory clay loam, 25 to 50 percent slopes, severely eroded	1,423	0.5
Hu	Huntington silt loam	1,777	0.6
	Total	20,388	6.9

Clinton County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
CuC2	Crouse-Miamian silt loams, 6 to 12 percent slopes, eroded	5,146	2.0
CuD2	Crouse-Miamian silt loams, 12 to 18 percent slopes, eroded	1,044	0.4
HkD2	Hickory silt loam, 12 to 18 percent slopes, eroded	1,704	0.6
HkE2	Hickory silt loam, 18 to 25 percent slopes, eroded	523	0.2
HkF2	Hickory silt loam, 25 to 35 percent slopes, eroded	641	0.2
HnE2	Hickory-Morrisville silt loams, 18 to 25 percent slopes, eroded	119	*
MoE2	Miamian-Crouse silt loams, 18 to 25 percent slopes, eroded	770	0.3
MoF2	Miamian-Crouse silt loams, 25 to 50 percent slopes, eroded	1,363	0.5
WmA	Williamsburg silt loam, 0 to 2 percent slopes	171	*
WmB	Williamsburg silt loam, 2 to 6 percent slopes	95	*
	Total	11,576	4.4

*Less than 0.1 percent.

Columbiana County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
RhD	Richland silt loam, 15 to 25 percent slopes, stony	272	*
RhE	Richland silt loam, 25 to 40 percent slopes, stony	204	*
	Total	476	0.1

* Less than 0.1 percent.

Coshocton County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
AfB	Alford silt loam 2 to 6 percent slopes	198	*
AfC2	Alford silt loam, 6 to 15 percent slopes, eroded	616	0.2
Ht	Huntington silt loam, rarely flooded	344	*
MnA	Mentor silt loam, 0 to 2 percent slopes	787	0.2
MnB	Mentor silt loam, 2 to 6 percent slopes	829	0.2
MnC	Mentor silt loam, 6 to 15 percent slopes	1,675	0.5
MnD	Mentor silt loam, 15 to 25 percent slopes	783	0.2
RcC	Richland silt loam, 6 to 15 percent slopes	229	*
RcD	Richland silt loam, 15 to 25 percent slopes	947	0.3
WeC	Wellston silt loam, 6 to 15 percent slopes	557	0.2

WhC	Westmoreland silt loam, 6 to 15 percent slopes	3,142	0.9
WhD	Westmoreland silt loam, 15 to 25 percent slopes	13,234	3.6
WhE	Westmoreland silt loam, 25 to 35 percent slopes	13,957	3.8
	Total	37,298	10.3

* Less than 0.1 percent.

Crawford County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
AdB	Alexandria silt loam, 2 to 6 percent slopes	1,038	0.4
AdC2	Alexandria silt loam, 6 to 12 percent slopes, moderately eroded	2,066	0.8
AdD2	Alexandria silt loam, 12 to 18 percent slopes, moderately eroded	573	0.2
HpE	Hennepin-Alexandria silt loams, 18 to 50 percent slopes	775	0.3
	Total	4,452	1.7

* Less than 0.1 percent.

Cuyahoga County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Ch	Chagrin silt loam, occasionally flooded	4,252	1.4
GeF	Geeburg-Mentor silt loams, 25 to 70 percent slopes	5,194	1.8
	Total	9,446	3.2

Defiance County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Ge	Genesee loam, occasionally flooded	3,299	1.2
	Total	3,299	1.2

Delaware County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
MaB	Martinsville loam, 2 to 6 percent slopes	24	*
MbB	Martinsville loam, till substratum, 2 to 6 percent slopes	959	0.3
McD2	Mentor silt loam, 12 to 18 percent slopes, eroded	63	*
RoA	Rossburg silt loam, 0 to 2 percent slopes, occasionally flooded	1,464	0.5
	Total	2,510	0.9

* Less than 0.1 percent.

Erie County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
NoA	Nolin silt loam, 0 to 2 percent slopes, occasionally flooded	576	0.3
	Total	576	0.3

Fairfield County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
AfB	Alford silt loam, 2 to 6 percent slopes	2,163	0.7
AfC2	Alford silt loam, 6 to 12 percent slopes, eroded	1,860	0.6

Cg	Chagrin silt loam, frequently flooded	625	0.2
Gf	Gessie silt loam, occasionally flooded	1,748	0.5
Gg	Gessie silt loam, frequently flooded	1,841	0.6
HhC2	Hickory silt loam, 6 to 12 percent slopes, eroded	810	0.2
HkE	Hickory-Germano complex, 20 to 35 percent slopes	583	0.2
HmD2	Hickory-Gilpin complex, 12 to 20 percent slopes, eroded	2,889	0.9
PkB	Pike silt loam, 2 to 6 percent slopes	432	0.1
PkC2	Pike silt loam, 6 to 12 percent slopes, eroded	559	0.2
	Total	13,510	4.2

Fayette County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Gn	Genesee silt loam	826	0.3
Rs	Ross silt loam	1,393	0.5
	Total	2,219	0.9

Franklin County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Gn	Genesee silt loam, occasionally flooded	2,424	0.7
Uw	Urban land-Genesee complex, occasionally flooded	1,370	0.4
	Total	3,794	1.1

Gallia County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
AkB	Allegheny loam, 3 to 8 percent slopes	550	0.2
AkC	Allegheny loam, 8 to 15 percent slopes	652	0.2
AkD	Allegheny loam, 15 to 25 percent slopes	587	0.2
Cg	Chagrin silt loam, frequently flooded	6,780	2.2
Cu	Cuba silt loam, occasionally flooded	1,226	0.4
EkB	Elkinsville silt loam, 1 to 6 percent slopes	2,129	0.7
	Total	11,924	4.0

Greene County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Gn	Genesee loam	1,831	0.7
Rs	Ross loam	3,601	1.4
RtA	Rush silt loam, 0 to 2 percent slopes	2,036	0.8
RtB	Rush silt loam, 2 to 6 percent slopes	1,932	0.7
	Total	9,400	3.5

Guernsey County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
AgC	Allegheny loam, 8 to 15 percent slopes	407	0.1
MeB	Mentor silt loam, 2 to 8 percent slopes	2,595	0.8

MeC	Mentor silt loam, 8 to 15 percent slopes	1,863	0.6
MeD	Mentor silt loam, 15 to 25 percent slopes	1,567	0.5
MfB	Mentor-Urban land complex, 2 to 8 percent slopes	152	*
MgB	Mentor silt loam, 2 to 6 percent slopes	8	*
RcC	Richland channery loam, 8 to 15 percent slopes	19	*
RcD	Richland channery loam, 15 to 25 percent slopes	471	0.1
	Total	7,082	2.1

* Less than 0.1 percent.

Hamilton County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Gn	Genesee loam, occasionally flooded	3,912	1.5
Go	Genesee-Urban land complex, occasionally flooded	1,888	0.7
Hu	Huntington silt loam, occasionally flooded	875	0.3
Ju	Jules silt loam, occasionally flooded	5,635	2.1
McA	Martinsville silt loam, 0 to 2 percent slopes	2,073	0.8
McB	Martinsville silt loam, 2 to 6 percent slopes	616	0.2
PbB2	Parke silt loam, 3 to 8 percent slopes, eroded	575	0.2
PbC2	Parke silt loam, 8 to 15 percent slopes, eroded	914	0.3
PbD	Parke silt loam, 15 to 25 percent slopes	381	0.1
PbE	Parke silt loam, 25 to 35 percent slopes	381	0.1
PcB	Parke-Urban land complex, 3 to 8 percent slopes	519	0.2
PcC	Parke-Urban land complex, 8 to 15 percent slopes	320	0.1
RwB2	Russell silt loam, 3 to 8 percent slopes, eroded	1,621	0.6
RxB	Russell-Urban land complex, 3 to 8 percent slopes	8,304	3.1
UgB	Urban land-Elkinsville complex, 3 to 8 percent slopes	1,117	0.4
UgC	Urban land-Elkinsville complex, 8 to 15 percent slopes	722	0.3
Uh	Urban land-Huntington complex, frequently flooded	4,627	1.8
UmB	Urban land-Martinsville complex, 3 to 8 percent slopes	5,253	2.0
UmC	Urban land-Martinsville complex, 8 to 15 percent slopes	431	0.2
	Total	40,164	15.2

Hardin County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Gn	Genesee silt loam	14	*
MaB	Martinsville loam, 1 to 4 percent slopes	397	0.1
No	Nolin silt loam, occasionally flooded	810	0.3
	Total	1,221	0.4

* Less than 0.1 percent.

Henry County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Gm	Genesee loam	372	0.1
Rs	Ross loam	547	0.2
	Total	919	0.3

Highland County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
EkB	Elkinsville silt loam, 1 to 6 percent slopes	14	*
Gd	Gessie loam, frequently flooded	77	*
Ge	Gessie silt loam, occasionally flooded	8	*
Gn	Genesee silt loam	5,829	1.6
HkC2	Hickory silt loam, 6 to 12 percent slopes, moderately eroded	1,741	0.5
HkD2	Hickory silt loam, 12 to 18 percent slopes, moderately eroded	4,538	1.3
HkE2	Hickory silt loam, 18 to 25 percent slopes, moderately eroded	2,235	0.6
HkF2	Hickory silt loam, 25 to 35 percent slopes, moderately eroded	758	0.2
HyC3	Hickory clay loam, 6 to 12 percent slopes, severely eroded	352	*
HyD3	Hickory clay loam, 12 to 18 percent slopes, severely eroded	2,016	0.6
HyE3	Hickory clay loam, 18 to 25 percent slopes, severely eroded	201	*
OcA	Ockley silt loam, 0 to 2 percent slopes	141	*
OcB	Ockley silt loam, 2 to 6 percent slopes	566	0.2
OcC2	Ockley silt loam, 6 to 12 percent slopes, moderately eroded	444	0.1
OdB	Ockley-Urban land complex, gently sloping	40	*
Rn	Ross silt loam	2,944	0.8
RuB	Russell silt loam, 2 to 6 percent slopes	210	*
WvA	Williamsburg silt loam, 0 to 2 percent slopes	91	*
WvB	Williamsburg silt loam, 2 to 6 percent slopes	350	*
WvC	Williamsburg silt loam, 6 to 12 percent slopes	256	*
	Total	22,811	6.4

* Less than 0.1 percent.

Hocking County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
AfB	Alford silt loam, 2 to 6 percent slopes	269	*
AfC	Alford silt loam, 6 to 12 percent slopes	638	0.2
AgB	Allegheny loam, 2 to 6 percent slopes	235	*
AgC	Allegheny loam, 6 to 12 percent slopes	242	*
Cg	Chagrin silt loam, frequently flooded	13,498	5.0
HcD2	Hickory-Gilpin complex, 12 to 20 percent slopes, eroded	62	*
HkD2	Hickory silt loam, 12 to 20 percent slopes, eroded	2	*
HkE2	Hickory silt loam, 20 to 35 percent slopes, eroded	46	*
HmC2	Hickory silt loam, 6 to 12 percent slopes, eroded	1	*
HmD2	Hickory silt loam, 12 to 18 percent slopes, eroded	1,380	0.5
HmE	Hickory silt loam, 20 to 35 percent slopes, eroded	746	0.3
HmF	Hickory silt loam, 25 to 40 percent slopes	464	0.2
HrE	Hickory-Germano complex, 20 to 35 percent slopes	13	*
PkC2	Pike silt loam, 6 to 12 percent slopes, eroded	2	*
Po	Pope loam, occasionally flooded	2,169	0.8
RcD	Richland loam, 15 to 25 percent slopes	5	*
	Total	19,772	7.3

* Less than 0.1 percent.

Jackson County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
AkB	Allegheny loam, 3 to 8 percent slopes	319	0.1
AkC	Allegheny loam, 8 to 15 percent slopes	766	0.3
AkD	Allegheny loam, 15 to 25 percent slopes	2,166	0.8
Cu	Cuba silt loam, occasionally flooded	752	0.3
Ha	Haymond silt loam, occasionally flooded	10	*
	Total	4,013	1.5

* Less than 0.1 percent.

Jefferson County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
RaB	Richland silt loam, 2 to 6 percent slopes	68	*
RcB	Richland silt loam, 1 to 7 percent slopes	3,975	1.5
RcC	Richland silt loam, 7 to 15 percent slopes	200	*
	Total	4,243	1.6

* Less than 0.1 percent.

Lawrence County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Cg	Chagrin loam, frequently flooded	3,863	1.3
Ch	Chagrin silt loam, frequently flooded	63	*
Cu	Cuba silt loam, occasionally flooded	3,570	1.2
EkB	Elkinsville silt loam, 1 to 6 percent slopes	3,050	1.0
EkE	Elkinsville silt loam, 15 to 40 percent slopes	366	0.1
EmB	Elkinsville-Urban land complex, 1 to 8 percent slopes	3,657	1.3
	Total	14,569	5.0

* Less than 0.1 percent.

Licking County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
AcB	Alford silt loam, 2 to 8 percent slopes	35	*
AcC2	Alford silt loam, 8 to 15 percent slopes, eroded	5	*
AfA	Alford silt loam, 0 to 2 percent slopes	610	0.1
AfB	Alford silt loam, 2 to 6 percent slopes	3,105	0.7
AfC2	Alford silt loam, 6 to 12 percent slopes, eroded	705	0.2
AhB	Alford-Urban land complex, 2 to 6 percent slopes	500	0.1
HkC2	Hickory silt loam, 6 to 12 percent slopes, eroded	490	0.1
HkD2	Hickory silt loam, 12 to 18 percent slopes, eroded	265	*
MnA	Mentor silt loam, 0 to 2 percent slopes	520	0.1
MnB	Mentor silt loam, 2 to 6 percent slopes	3,405	0.8
MnC2	Mentor silt loam, 6 to 12 percent slopes, eroded	4,080	0.9
MnD2	Mentor silt loam, 12 to 18 percent slopes, eroded	370	*
PaC2	Parke silt loam, 6 to 12 percent slopes, eroded	2,250	0.5
RsA	Rush silt loam, 0 to 2 percent slopes	975	0.2
	Total	17,315	3.9

* Less than 0.1 percent.

Logan County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Gn	Genesee silt loam	1,371	0.5
	Total	1,371	0.5

Lorain County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
MnB	Mentor silt loam, 2 to 6 percent slopes	434	0.1
MnC	Mentor silt loam, 6 to 12 percent slopes	127	*
MnE	Mentor silt loam, 12 to 25 percent slopes	104	*
	Total	665	0.2

* Less than 0.1 percent.

Lucas County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
SmB	Sisson loam, 2 to 6 percent slopes	451	0.2
SmC	Sisson loam, 6 to 12 percent slopes	614	0.3
SmD	Sisson loam, 12 to 18 percent slopes	826	0.4
SnB	Sisson-Urban land complex, 2 to 12 percent slopes	1,546	0.7
	Total	3,437	1.5

Madison County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Rs	Ross silt loam, occasionally flooded	987	0.3
	Total	987	0.3

Mahoning County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
WrF2	Wooster loam, 25 to 50 percent slopes, moderately eroded	247	*
WsB	Wooster silt loam, 2 to 6 percent slopes	2,068	0.8
WsC2	Wooster silt loam, 6 to 12 percent slopes, moderately eroded	3,837	1.4
WsD2	Wooster silt loam, 12 to 18 percent slopes, moderately eroded	571	0.2
WsE2	Wooster silt loam, 18 to 25 percent slopes, moderately eroded	88	*
	Total	6,811	2.5

* Less than 0.1 percent.

Marion County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
MaA	Martinsville loam, 0 to 2 percent slopes	880	0.3
MaB	Martinsville loam, 2 to 6 percent slopes	477	0.2
No	Nolin silt loam, occasionally flooded	3,773	1.5
Ro	Rossburg silt loam, 0 to 2 percent slopes, occasionally flooded	4	*
	Total	5,134	2.0

* Less than 0.1 percent.

Medina County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Cr	Chagrin silt loam, occasionally flooded	59	*
MoB	Mentor silt loam, 2 to 6 percent slopes	4	*
WvB	Wooster-Riddles silt loams, 2 to 6 percent slopes	49	*
WvC2	Wooster-Riddles silt loams, 6 to 12 percent slopes, eroded	188	*
WvD2	Wooster-Riddles silt loams, 12 to 18 percent slopes, eroded	11	*
	Total	311	0.1

* Less than 0.1 percent.

Meigs County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Cg	Chagrin silt loam, frequently flooded	10,689	3.9
DuC	Duncannon silt loam, 6 to 12 percent slopes	227	*
EkA	Elkinsville silt loam, 0 to 2 percent slopes	261	*
GaC	Gallia loam, 6 to 12 percent slopes	802	0.3
GaD	Gallia loam, 12 to 18 percent slopes	255	*
Mo	Moshannon silt loam, frequently flooded	1,264	0.5
RcB	Richland silt loam, 2 to 6 percent slopes	1,071	0.4
RdD	Richland loam, 15 to 25 percent slopes	3	*
RdE	Richland loam, 25 to 40 percent slopes	1	*
	Total	14,573	5.3

* Less than 0.1 percent.

Mercer County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Gn	Genesee silt loam	1,816	0.6
	Total	1,816	0.6

Miami County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Rs	Ross silt loam	2,876	1.1
	Total	2,876	1.1

Monroe County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
AlD	Allegheny silt loam, 12 to 18 percent slopes	1	*
AsA	Ashton silt loam, 0 to 3 percent slopes	192	*
Cg	Chagrin silt loam	5,942	2.0
Hu	Huntington silt loam	737	0.3
	Total	6,872	2.3

* Less than 0.1 percent.

Montgomery County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Rs	Ross silt loam	10,731	3.6
Rt	Ross-Urban land complex	3,786	1.3
	Total	14,517	4.9

Morgan County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Ca	Chagrin silt loam, frequently flooded	327	0.1
RvE	Richland-Vandalia complex, 20 to 35 percent slopes	53	*
	Total	380	0.1

* Less than 0.1 percent.

Morrow County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
ObA	Ockley loam, 0 to 2 percent slopes	3	*
ObB	Ockley loam, 2 to 6 percent slopes	69	*
	Total	72	0.0

* Less than 0.1 percent.

Muskingum County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
AfB	Alford silt loam, 2 to 8 percent slopes	5,395	1.3
AfC2	Alford silt loam, 8 to 15 percent slopes, eroded	5,545	1.3
Cb	Chagrin loam, rarely flooded	2,277	0.5
LcD	Lakin-Alford complex, 15 to 25 percent slopes	541	0.1
No	Nolin silt loam, occasionally flooded	4,638	1.1
UtA	Urban land-Nolin complex, rarely flooded	593	0.1
	Total	18,989	4.4

Noble County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
AlD	Allegheny silt loam, 12 to 18 percent slopes	9	*
Ch	Chagrin silt loam, occasionally flooded	1,990	0.8
RcD	Richland channery loam, 15 to 25 percent slopes	16	*
	Total	2,015	0.8

* Less than 0.1 percent.

Ottawa County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Gn	Genesee silt loam, frequently flooded	1,041	0.6
	Total	1,041	0.6

Perry County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
AfB	Alford silt loam, 1 to 8 percent slopes	6,773	2.6
AfC	Alford silt loam, 8 to 15 percent slopes	1,862	0.7
AfC2	Alford silt loam, 8 to 15 percent slopes, eroded	107	*
AfD	Alford silt loam, 15 to 25 percent slopes	282	0.1
AgB	Alford silt loam, 2 to 8 percent slopes	3	*
MeB	Mentor silt loam, gravelly substratum, 1 to 8 percent slopes	836	0.3
MeC	Mentor silt loam, gravelly substratum, 8 to 15 percent slopes	1,137	0.4
No	Nolin silt loam, occasionally flooded	3,510	1.3
SfD	Shelocta-Cruze complex, 15 to 25 percent slopes	1	*
SfE	Shelocta-Cruze complex, 25 to 40 percent slopes	26	*
	Total	14,537	5.5

* Less than 0.1 percent.

Pickaway County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Gn	Genesee silt loam, occasionally flooded	9,332	2.9
Gs	Gessie silt loam, occasionally flooded	47	*
Rt	Ross silt loam, overwash, frequently flooded	801	0.2
WeA	Wea silt loam, 0 to 2 percent slopes	1,965	0.6
WeB	Wea silt loam, 2 to 6 percent slopes	476	0.1
	Total	12,621	3.9

* Less than 0.1 percent.

Pike County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
En	Elkinsville silt loam, rarely flooded	2,182	0.8
Ge	Genesee silt loam, occasionally flooded	6,699	2.4
Gf	Gessie silt loam, occasionally flooded	72	*
Ha	Haymond silt loam, occasionally flooded	2,705	1.0
Hu	Huntington silt loam, occasionally flooded	3,637	1.3
Mh	Martinsville loam, rarely flooded	727	0.3
Mt	Mentor silt loam, rarely flooded	117	*
PaA	Parke silt loam, 0 to 3 percent slopes	639	0.2
PaB	Parke silt loam, 3 to 8 percent slopes	212	*
SuB	Spargus channery silt loam, 2 to 6 percent slopes	7	*
	Total	16,997	6.0

* Less than 0.1 percent.

Portage County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Tg	Tioga loam	1,055	0.3
	Total	1,055	0.3

Preble County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
RuB	Russell silt loam, 2 to 6 percent slopes	2,857	1.0
	Total	2,857	1.0

Putnam County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Gn	Genesee silt loam	1,807	0.6
Kw	Knoxdale silt loam, occasionally flooded	10	*
Rw	Rossburg silt loam, occasionally flooded	33	*
	Total	1,850	0.6

* Less than 0.1 percent.

Richland County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
MeB	Mentor silt loam, 2 to 6 percent slopes	267	*
MeC	Mentor silt loam, 6 to 12 percent slopes	198	*
WeD	Westmoreland silt loam, 12 to 18 percent slopes	102	*
WmD	Wheeling and Mentor silt loams, 12 to 18 percent slopes	301	*
	Total	868	0.3

* Less than 0.1 percent

Ross County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Ge	Gessie silt loam, occasionally flooded	17,914	4.0
Gf	Gessie silt loam, frequently flooded	5,601	1.3
Hd	Haymond silt loam, occasionally flooded	2,911	0.7
HkD2	Hickory silt loam, 12 to 20 percent slopes, eroded	131	*
HkE2	Hickory silt loam, 20 to 35 percent slopes, eroded	329	*
Ht	Huntington silt loam, occasionally flooded	245	*
McA	Martinsville loam, rarely flooded	166	*
MeC2	Mentor silt loam, 6 to 12 percent slopes, eroded	702	0.2
MeD2	Mentor silt loam, 12 to 20 percent slopes, eroded	512	0.1
MfA	Mentor silt loam, rarely flooded	561	0.1
MgA	Mentor silt loam, gravelly substratum, 0 to 2 percent slopes	2,914	0.7
MgB	Mentor silt loam, gravelly substratum, 2 to 6 percent slopes	657	0.1
PkA	Pike silt loam, 0 to 2 percent slopes	1,873	0.4
PkB	Pike silt loam, 2 to 6 percent slopes	1,355	0.3
SuB	Spargus channery silt loam, 2 to 6 percent slopes	1,259	0.3
	Total	37,130	8.4

* Less than 0.1 percent.

Sandusky County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
MeB	Mentor silt loam, 1 to 4 percent slopes	1,277	0.5
MeF	Mentor silt loam, 25 to 50 percent slopes	756	0.3

	Total	2,033	0.8

Scioto County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
AfD	Alford silt loam, 10 to 25 percent slopes	660	0.2
Cu	Cuba silt loam, occasionally flooded	1,280	0.3
EhB	Elkinsville silt loam, 1 to 6 percent slopes	12	*
EkB	Elkinsville silt loam, 1 to 8 percent slopes	2,768	0.7
EkE	Elkinsville silt loam, 25 to 40 percent slopes	1,679	0.4
EmB	Elkinsville-Urban land complex, 1 to 8 percent slopes	1,541	0.4
Ge	Genesee silt loam, occasionally flooded	2,365	0.6
Ha	Haymond silt loam, occasionally flooded	3,054	0.8
Hu	Huntington silt loam, occasionally flooded	522	0.1
No	Nolin silt loam, occasionally flooded	12,086	3.1
SbB	Shelocta silt loam, 3 to 8 percent slopes	10,880	2.8
SbC	Shelocta silt loam, 8 to 15 percent slopes	2,119	0.5
SbD	Shelocta silt loam, 15 to 25 percent slopes	3,584	0.9
WmB	Wheeling silt loam, 1 to 8 percent slopes	1,450	0.4
	Total	44,000	11.2

* Less than 0.1 percent.

Seneca County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Ch	Chagrin silt loam, occasionally flooded	5,427	1.5
Ge	Genesee silt loam, occasionally flooded	157	*
Ru	Ross silt loam, occasionally flooded	1,170	0.3
	Total	6,754	1.9

* Less than 0.1 percent.

Shelby County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Ge	Genesee silt loam, occasionally flooded	1,108	0.4
	Total	1,108	0.4

Stark County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
MeA	Mentor silt loam, 0 to 2 percent slopes	270	*
MeB	Mentor silt loam, 2 to 6 percent slopes	447	0.1
MeC	Mentor silt loam, 6 to 12 percent slopes	237	*
MeD	Mentor silt loam, 12 to 18 percent slopes	176	*
RuA	Rush silt loam, 0 to 3 percent slopes	---	*
WuB	Wooster silt loam, 2 to 6 percent slopes	6,487	1.7
WuC	Wooster silt loam, 6 to 12 percent slopes	3,816	1.0

WuC2	Wooster silt loam, 6 to 12 percent slopes, moderately eroded	10,791	2.9
WuD2	Wooster silt loam, 12 to 18 percent slopes, moderately eroded	6,137	1.7
WuE2	Wooster silt loam, 18 to 25 percent slopes, moderately eroded	1,538	0.4
WuF2	Wooster silt loam, 25 to 50 percent slopes, moderately eroded	143	*
WvD	Wooster-Urban land complex, steep	305	*
WxB	Wooster-Riddles silt loams, 2 to 6 percent slopes	---	*
WxC	Wooster-Riddles silt loams, 6 to 12 percent slopes	---	*
WxC2	Wooster-Riddles silt loams, 6 to 12 percent slopes, eroded	---	*
WxD2	Wooster-Riddles silt loams, 12 to 18 percent slopes, eroded	---	*
	Total	30,347	8.2

* Less than 0.1 percent.

Summit County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
CwC2	Chili-Wooster complex 6 to 12 percent slopes, moderately eroded	449	0.2
CwD2	Chili-Wooster complex, 12 to 18 percent slopes, moderately eroded	275	0.1
CwE2	Chili-Wooster complex, 18 to 25 percent slopes, moderately eroded	232	*
WwD	Wooster-Urban land complex, hilly	300	0.1
WyC2	Wooster-Riddles silt loams, 6 to 12 percent slopes, eroded	4	*
	Total	1,260	0.5

* Less than 0.1 percent.

Tuscarawas County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
EkA	Elkinsville silt loam, 0 to 3 percent slopes	600	0.2
MeB	Mentor silt loam, 2 to 6 percent slopes	2	*
RuA	Rush silt loam, 0 to 3 percent slopes	3,322	0.9
	Total	3,924	1.1

* Less than 0.1 percent.

Union County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Gn	Genesee silt loam	3,006	1.1
No	Nolin silt loam, 0 to 2 percent slopes, occasionally flooded	35	*
RpA	Rossburg silt loam, 0 to 2 percent slopes, occasionally flooded	2	*
	Total	3,043	1.1

* Less than 0.1 percent.

Vinton County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Cg	Chagrin silt loam, 0 to 2 percent slopes, frequently flooded	4,434	1.7
RcD	Richland loam, 15 to 25 percent slopes	29	*
RcE	Richland loam, 25 to 40 percent slopes	48	*
	Total	4,511	1.7

* Less than 0.1 percent.

Warren County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
CqC2	Crouse-Miamian silt loams, 6 to 12 percent slopes, eroded	94	*
CrB	Crider silt loam, 2 to 6 percent slopes	333	0.1
Gd	Genesee fine sandy loam	4,515	1.7
Gn	Genesee loam	4,612	1.8
HiD2	Hickory silt loam, 12 to 18 percent slopes, eroded	220	*
HiE2	Hickory silt loam, 18 to 25 percent slopes, eroded	7	*
HiF2	Hickory silt loam, 25 to 35 percent slopes, eroded	279	0.1
HmE	Hennepin-Miamian silt loams, 18 to 25 percent slopes	240	*
HmE2	Hennepin-Miamian silt loams, 18 to 25 percent slopes, moderately eroded	1,654	0.6
HnD3	Hennepin-Miamian complex, 12 to 18 percent slopes, severely eroded	399	0.2
HuE2	Hickory-Morrisville silt loams, 18 to 25 percent slopes, eroded	27	*
PaB	Parke silt loam, 2 to 6 percent slopes	224	*
PaD2	Parke silt loam, 6 to 18 percent slopes, moderately eroded	183	*
Rn	Ross loam	3,598	1.4
WlA	Williamsburg silt loam, 0 to 2 percent slopes	156	*
WlB	Williamsburg silt loam, 2 to 6 percent slopes	529	0.2
WlC2	Williamsburg silt loam, 6 to 12 percent slopes, moderately eroded	166	*
	Total	17,236	6.6

* Less than 0.1 percent.

Washington County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
AlB	Allegheny silt loam, 2 to 6 percent slopes	536	0.1
AlC	Allegheny silt loam, 6 to 12 percent slopes	1,801	0.4
AlD	Allegheny silt loam, 12 to 18 percent slopes	1,479	0.4
AlG	Allegheny silt loam, 18 to 50 percent slopes	497	0.1
AsA	Ashton silt loam, 0 to 2 percent slopes	631	0.2
AsB	Ashton silt loam, 2 to 6 percent slopes	101	*
Cg	Chagrin silt loam	7,284	1.8
DtB	Duncannon silt loam, 2 to 6 percent slopes	156	*
DtC	Duncannon silt loam, 6 to 12 percent slopes	147	*
DuD	Duncannon-Lakin complex, 12 to 18 percent slopes	205	*
DuE	Duncannon-Lakin complex, 18 to 25 percent slopes	373	*
GaB	Gallia silt loam, 2 to 6 percent slopes	441	0.1
GaC	Gallia silt loam, 6 to 12 percent slopes	1,433	0.4
GaD	Gallia silt loam, 12 to 18 percent slopes	341	*
HcA	Hackers silt loam, 0 to 2 percent slopes	948	0.2
HcB	Hackers silt loam, 2 to 6 percent slopes	1,758	0.4
HcC	Hackers silt loam, 6 to 12 percent slopes	198	*
Hu	Huntington silt loam	852	0.2
MeA	Mentor silt loam, 0 to 2 percent slopes	2,182	0.5
MeB	Mentor silt loam, 2 to 6 percent slopes	1,991	0.5
MeC	Mentor silt loam, 6 to 12 percent slopes	611	0.1
Mp	Moshannon silt loam	6,621	1.6
No	Nolin silt loam	2,891	0.7
	Total	33,477	8.2

* Less than 0.1 percent.

Wayne County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
RhB	Riddles silt loam, 2 to 6 percent slopes	2,444	0.7
RhC	Riddles silt loam, 6 to 12 percent slopes	2,359	0.7
RhD2	Riddles silt loam, 12 to 18 percent, eroded	1,069	0.3
RhE	Riddles silt loam, 18 to 25 percent slopes	2,500	0.7
WuB	Wooster-Riddles silt loams, 2 to 6 percent slopes	23,623	6.6
WuC	Wooster-Riddles silt loams, 6 to 12 percent slopes	6,927	1.9
WuC2	Wooster-Riddles silt loams, 6 to 12 percent slopes, eroded	15,191	4.3
WuD2	Wooster-Riddles silt loams, 12 to 18 percent slopes, eroded	6,816	1.9
	Total	60,929	17.1

Williams County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Ge	Genesee loam	1,396	0.5
	Total	1,396	0.5

Wood County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
Gm	Genesee loam	385	*
Gn	Genesee silt loam	777	0.2
	Total	1,162	0.3

* Less than 0.1 percent.

Wyandot County, Ohio

Map Symbol	Soil Name	Acres	Percent of County
AdC2	Alexandria silt loam, 6 to 12 percent slopes, moderately eroded	1	*
Cm	Chagrin silt loam, rarely flooded	871	0.3
Ge	Genesee silt loam, occasionally flooded	4,143	1.6
HpE	Hennepin-Alexandria silt loams, 18 to 50 percent slopes	1	*
MaB	Martinsville fine sandy loam, 2 to 6 percent slopes	591	0.2
SfC2	Shinrock-Martinsville complex, 6 to 12 percent slopes, eroded	1,638	0.6
SfD2	Shinrock-Martinsville complex, 12 to 18 percent slopes, eroded	246	*
	Total	7,501	2.9

* Less than 0.1 percent.

Appendix 9: Adjusting Hydrologic Soil Group for Construction

This appendix provides hydrologic soil group (HSG) values for undisturbed Ohio soils and predictable HSG values for Ohio soils that are altered by construction practices.

Hydrologic soil groups are used to assign a Curve Number (CN) when performing runoff calculations or in hydrologic models. Soil map units have been assigned to the four Hydrologic Soil Groups in technical resources and soil resources published by the USDA Natural Resource Conservation Service[1] (NRCS). NRCS HSG values are based on undisturbed, naturally-occurring soils. In contrast, soils at development sites are typically changed dramatically by construction practices that remove topsoil, change the soil profile and compact soils with heavy equipment. The runoff potential of a site is significantly impacted by these changes and should be reflected in hydrologic modeling and runoff calculations.

The following tables contain the HSGs and predicted HSGs for post-construction that were developed by applying the HSG criteria to modeled representative post-construction soil profiles. The modeled scenario consisted of the removal of the topsoil and subsoil to a depth of 18 inches and the compaction of the zone from 0 to 6 inches at the new surface. A fuller explanation of this process is available at the end of this appendix.

Soil Map Unit Component	HSG[1]	Post-Const HSG
Aaron	C	D
Abscota Variant (Warren)	A	No Eval.
Adrian	A/D	D
Aetna	B/D	D
Alexandria	C	D
Alford	B	D
Algansee	A/D	D
Algiers	B/D	D
Allegheny	B	C
Allegheny Variant (Belmont, Pike)	B	No Eval.
Allis	D	D
Alvada	B/D	D
Amanda	C	D
Amanda Variant (Licking)	B	No Eval.
Arkport	A	A
Ashton	B	D
Atlas	D	D
Aurand	C/D	D
Ava	C	D
Avonburg	D	D
Barkcamp	A	No Eval.

Soil Map Unit Component	HSG[1]	Post-Const HSG
Barkcamp (CL surface)	A	A
Barkcamp (L surface)	A	B
Beasley	C	No Eval.
Beaucoup	C/D	D
Belmore	B	C
Belpre	C	No Eval.
Bennington	C/D	D
Berks	B	D
Bethesda	C	D
Biglick	D	D
Birkbeck	B	D
Bixler	B	D
Blairton	C	No Eval.
Blakeslee	B/D	D
Blanchester	C/D	D
Blount	C/D	D
Bogart	B/D	D
Bogart Variant (Mahoning)	C	No Eval.
Bonnell	C	D
Bonnie	C/D	D
Bono	C/D	D

Notes: CL = clay loam; L = loam; substr = substratum; limestne substr = limestone substratum; Dual classes in Ohio, such as A/D, B/D, C/D are given for drained or undrained condition; No Eval. = No evaluation performed.

1. Hydrologic Soil Groups (HSGs) for Ohio (for undisturbed naturally-occuring sites) were updated in 2008 and should be used rather than HSGs from earlier publications (http://www.oh.nrcs.usda.gov/technical/soils/OH_hsg.pdf or contact the USDA Natural Resources Conservation Service in Columbus, Ohio). You may also utilize www.OhioERIN.com to find site specific HSG (unaltered).

Soil Map Unit Component	HSG[1]	Post-Const HSG
Boston	C	D
Boyer	A	B
Braceville	C/D	D
Brady	B	No Eval.
Bratton	C	D
Brecksville	D	D
Brenton	B	No Eval.
Bronson	B	No Eval.
Brooke	D	D
Brookside	C	D
Brookston	B/D	D
Broughton	D	D
Brownsville	A	D
Brushcreek	C	D
Calcutta	C/D	D
Cambridge	D	D
Cana	C	D
Cana Variant	C	No Eval.
Canadice	D	D
Canal	C/D	D
Caneadea	D	D
Canfield	C	D
Canfield (Summit)	D	D
Canfield Variant (Stark)	C	No Eval.
Captina	C	No Eval.
Cardinal	C/D	D
Cardington	C	D
Carlisle	A/D	D
Casco	B	A
Castalia	A	D
Cedarfalls	A	No Eval.
Celina	C	D
Celina Variant	C	No Eval.
Centerburg	C	D
Ceresco	A/D	D
Chagrin	B	C
Channahon	D	D
Chavies	A	B
Chenango	A	A
Chili	B	C
Cidermill	B	D
Cincinnati	C	D
Clarksburg	C	D
Claysville	C/D	D

Soil Map Unit Component	HSG[1]	Post-Const HSG
Clermont	D	D
Clifty	A	C
Clymer	B	C
Coblen	B	No Eval.
Cohoctah	A/D	D
Colonie	A	A
Colwood	B/D	D
Colwood (Erie)	C/D	D
Colyer	D	D
Colyer Variant	C	No Eval.
Condit	C/D	D
Conneaut	C/D	D
Conotton	A	C
Conotton Variant	A	No Eval.
Coolville	C	D
Corwin	C	D
Coshocton	C	D
Crane	B/D	D
Crider	B	No Eval.
Crosby	C/D	D
Crouse	B	No Eval.
Cruze	C	D
Cuba	B	C
Culleoka	B	D
Cyclone	B/D	D
Cygnet	B/D	D
Damascus	B/D	D
Damascus (Stark)	C/D	D
Dana	B	D
Darien	C/D	D
Darroch	B/D	D
Defiance	C/D	D
Dekalb	B	D
Del Rey	C/D	D
Del Rey Variant	C/D	D
Digby	B/D	D
Digby (till substr) (Wood)	C/D	D
Digby Variant (Auglaize, Putnam)	C/D	D
Dixboro	B/D	D
Doles	C/D	D
Donnelsville	B	No Eval.
Drummer	B/D	D
Dubois	C/D	D
Dunbridge	B	D

Notes: CL = clay loam; L = loam; substr = substratum; limestne substr = limestone substratum; Dual classes in Ohio, such as A/D, B/D, C/D are given for drained or undrained condition; No Eval. = No evaluation performed.

1. Hydrologic Soil Groups (HSGs) for Ohio (for undisturbed naturally-occuring sites) were updated in 2008 and should be used rather than HSGs from earlier publications (http://www.oh.nrcs.usda.gov/technical/soils/OH_hsg.pdf or contact the USDA Natural Resources Conservation Service in Columbus, Ohio). You may also utilize www.OhioERIN.com to find site specific HSG (unaltered).

Soil Map Unit Component	HSG[1]	Post-Const HSG
Duncannon	B	C
Dunham	B/D	D
Eden	D	D
Edenton	C	D
Edwards	C/D	D
Eel	B/D	D
Eel moderately deep	C/D	D
Eel Variant (Shelby)	C	No Eval.
Elba	C	D
Eldean	B	D
Elkinsville	B	D
Elliott	C/D	D
Ellsworth	C	D
Elnora	A/D	D
Endoaquents	D	D
Enoch	C	C
Ernest	C	D
Euclid	C/D	D
Fairmount	D	D
Fairmount Variant (Greene)	C	No Eval.
Fairpoint	C	D
Farmerstown	C	C
Faywood	C	D
Fincastle	C/D	D
Fitchville	C/D	D
Fitchville Variant	C/D	D
Flatrock	B/D	D
Flatrock (limestne substr)	B/D	D
Fluvaquents	D	D
Fox	B	D
Frankstown Variant	C	No Eval.
Fredericktown	B	No Eval.
Frenchtown	D	D
Fries	D	D
Fulton	C/D	D
Fulton (till substr)	C/D	D
Fulton Variant	C/D	D
Fulton (till substr)	C/D	D
Gageville	C/D	D
Galen	A/D	D
Gallia	B	C
Gallipolis	C	C
Gallman	B	C
Gasconade	D	D

Soil Map Unit Component	HSG[1]	Post-Const HSG
Gavers	C/D	D
Geeburg	D	D
Genesee	B	C
Genesee Variant (Ottawa)	C	No Eval.
Germano	B	D
Gessie	B	C
Gilford	A/D	D
Gilpin	C	D
Ginat	C/D	D
Glendora	A/D	D
Glenford	C/D	D
Glynwood	D	D
Glynwood (limestne substr) (Hancock)	C/D	D
Gosport	D	D
Granby	A/D	D
Granby (till substr)	A/D	D
Grayford	B	No Eval.
Gresham	C/D	D
Guernsey	C	D
Hackers	B	D
Haney	B	D
Hanover	C	D
Harbor	B/D	D
Harrod	C/D	D
Hartshorn	B	D
Hartshorn Variant (Monroe)	B/D	D
Haskins	C/D	D
Haubstadt	D	D
Haymond	B	C
Hayter	A	C
Hazleton	A	C
Hennepin	D	D
Henshaw	C/D	D
Henshaw Variant	C/D	D
Heverlo	C	No Eval.
Hickory	B	C
Holly	B/D	D
Holton	B/D	D
Homer	B/D	D
Homewood	C	D
Homeworth	B/D	D
Hornell	D	D
Houcktown	C/D	D
Hoytville	C/D	D

Notes: CL = clay loam; L = loam; substr = substratum; limestne substr = limestone substratum; Dual classes in Ohio, such as A/D, B/D, C/D are given for drained or undrained condition; No Eval. = No evaluation performed.

1. Hydrologic Soil Groups (HSGs) for Ohio (for undisturbed naturally-occuring sites) were updated in 2008 and should be used rather than HSGs from earlier publications (http://www.oh.nrcs.usda.gov/technical/soils/OH_hsg.pdf or contact the USDA Natural Resources Conservation Service in Columbus, Ohio). You may also utilize www.OhioERIN.com to find site specific HSG (unaltered).

Soil Map Unit Component	HSG[1]	Post-Const HSG
Hoytville Variant	C/D	D
Huntington	B	D
Hyatts	C/D	D
Ionia	B	No Eval.
Iva	C/D	D
Jenera	C/D	D
Jeneva	B	No Eval.
Jessup	C	D
Jimtown	B/D	D
Johnsburg	D	D
Joliet	D	D
Jonesboro	C	D
Jules	B	No Eval.
Kanawha	B	C
Kane	B/D	D
Keene	C	D
Kendallville	C	C
Kensington	B	C/D
Kerston	C/D	D
Kibbie	B/D	D
Killbuck	C/D	D
Kings Variant	C/D	D
Kingsville	A/D	D
Kinn	B	No Eval.
Knoxdale	B	No Eval.
Kokomo	C/D	D
Kyger	A/D	D
Lakin	A	A
Lamberjack	B/D	D
Lamson	A/D	D
Landes	A	A
Lanier	A	A
Latham	D	D
Latty	C/D	D
Latty (till substr)	C/D	D
Lawshe	D	D
Lenawee	C/D	D
Lenawee Variant	C/D	D
Leoni	A	No Eval.
Lewisburg	D	D
Library Variant	C/D	D
Libre	C	No Eval.
Licking	C	D
Lily	B	D

Soil Map Unit Component	HSG[1]	Post-Const HSG
Lindside	C	D
Linwood	B/D	D
Lippincott	B/D	D
Lobdell	C	D
Lockport	D	D
Lorain	C/D	D
Lordstown	C	D
Lorenzo	A	No Eval.
Losantville	D	D
Loudon	C	D
Loudonville	C	D
Lowell	C	D
Lucas	D	D
Lumberton	B	D
Luray	C/D	D
Luray Variant (Stark)	B/D	D
Lybrand	C	D
Lykens	C	D
Mahalasville	B/D	D
Mahoning	C/D	D
Marblehead	D	D
Marengo	B/D	D
Markland	C	D
Martinsville	B	D
Martisco	B/D	D
Martisco Variant (Logan)	C/D	D
McGary	C/D	D
McGary Variant	C/D	D
McGuffey	D	D
Mechanicsburg	B	C
Medway	C	D
Medway Variant	C	D
Medway (limestne substr)	B/D	D
Melvin	B/D	D
Mentor	B	D
Mermill	C/D	D
Mermill Variant	C/D	D
Mertz	C	C
Metamora	B/D	D
Miami	C	D
Miami Variant	C	No Eval.
Miamian	C	D
Miamian Variant	C	No Eval.
Milford	C/D	D

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Soil Map Unit Component	HSG[1]	Post-Const HSG
Mill	C/D	D
Millgrove	B/D	D
Millsdale	C/D	D
Milton	C	D
Milton Variant	C	No Eval.
Miner	C/D	D
Minoa	B/D	D
Mitiwanga	C/D	D
Mitiwanga Variant	D	D
Monongahela	D	D
Montgomery	C/D	D
Montgomery Variant (Pike)	D	D
Morley	D	D
Morley (limestone substr)	C	No Eval.
Morningsun	B	No Eval.
Morristown	C	C
Morrisville	C	No Eval.
Mortimer	C/D	D
Moshannon	B	D
Muck	B/D	D
Muse	C	D
Muskego	C/D	D
Muskingum	C	C
Nappanee	D	D
Negley	A	C
Neotoma	A	No Eval.
Newark	B/D	D
Newark Variant	B/D	D
Nicely	C	No Eval.
Nicholson	C	No Eval.
Nineveh	B	No Eval.
Nolin	B	D
Nolin Variant	B	No Eval.
Oakville	A	A
Ockley	B	C
Odell	C/D	D
Ogontz	B	No Eval.
Olentangy	B/D	D
Olmsted	B/D	D
Omulga	D	D
Opequon	D	D
Orrville	B/D	D
Orrville Variant (Richland)	A/D	D
Orrville Variant (Ashland)	C/D	D

Soil Map Unit Component	HSG[1]	Post-Const HSG
Oshtemo	A	A
Oshtemo (till substr)	A	C
Otego	B/D	D
Otisville	A	A
Ottokee	A	D
Ottokee (till substr)	A	No Eval.
Otwell	D	D
Pacer	B	No Eval.
Painesville	C/D	D
Pandora	C/D	D
Papakating	C/D	D
Parke	B	D
Parr	B	No Eval.
Pate	D	D
Patton	B/D	D
Patton Variant	B/D	D
Paulding	D	D
Pekin	D	D
Peoga	C/D	D
Perrin	A	No Eval.
Pewamo	C/D	D
Pewamo Variant	C/D	D
Philo	B	D
Pierpont	C	D
Pike	B	D
Pinegrove	A	A
Pinnebog	A/D	D
Piopolis	C/D	D
Plainfield	A	A
Platea	D	D
Plattville	C	No Eval.
Plumbrook	A/D	D
Pope	B	A
Princeton	B	B
Prout	C/D	D
Purdy Variant	C/D	D
Pyrmont	D	D
Ragsdale	C/D	D
Rainsboro	C	D
Rainsville	C	No Eval.
Ramsey	D	D
Randolph	C/D	D
Rarden	D	D
Raub	B/D	D

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1. Hydrologic Soil Groups (HSGs) for Ohio (for undisturbed naturally-occuring sites) were updated in 2008 and should be used rather than HSGs from earlier publications (http://www.oh.nrcs.usda.gov/technical/soils/OH_hsg.pdf or contact the USDA Natural Resources Conservation Service in Columbus, Ohio). You may also utilize www.OhioERIN.com to find site specific HSG (unaltered).

Soil Map Unit Component	HSG[1]	Post-Const HSG
Ravenna	D	D
Rawson	D	D
Red Hook	B/D	D
Reesville	B/D	D
Remsen	D	D
Rensselaer	B/D	D
Rensselaer (till substr)	B/D	D
Richland	B	D
Riddles	B	C
Rigley	A	A
Rigley Variant	A	No Eval.
Rimer	A/D	D
Rimer (deep phase)	A/D	D
Risingsun	C/D	D
Ritchey	D	D
Rittman	D	D
Rockmill	B/D	D
Rodman	A	A
Rollersville	C/D	D
Romeo	D	D
Roselms	D	D
Ross	B	C
Ross Variant	D	D
Rossburg	B	D
Rossmoyne	C	D
Roundhead	C/D	D
Rush	B	D
Russell	B	D
Russell (bedrock substr)	B	No Eval.
Sandusky	B/D	D
Sarahsville	D	D
Saranac	C/D	D
Sardinia	B	D
Savona	B/D	D
Saylesville	C	D
Schaffenaker	A	D
Schaffer	C/D	D
Scioto	B	No Eval.
Sciotoville	C	D
Sebring	C/D	D
Sebring Variant	C/D	D
Secondcreek	C/D	D
Sees	C	D
Senecaville	C/D	D

Soil Map Unit Component	HSG[1]	Post-Const HSG
Seward	A	D
Sewell	A	No Eval.
Shawtown	B	No Eval.
Sheffield	D	D
Shelocta	B	D
Shinrock	C	D
Shinrock Variant (Henry)	C/D	D
Shinrock (till substr)	C/D	D
Shoals	B/D	D
Shoals (mod deep)	C/D	D
Shoals Variant	C/D	D
Sisson	B	D
Skidmore	A	C
Skidmore Variant	A	No Eval.
Sleeth	B/D	D
Sligo	B	No Eval.
Sloan	B/D	D
Sloan (mod deep)	B/D	D
Sloan Variant	B/D	D
Sloan (limestone substr)	B/D	D
Smothers	C/D	D
Spargus	B	No Eval.
Sparta	A	No Eval.
Spinks	A	A
Spinks (deep to limestone)	A	No Eval.
St. Clair	D	D
Stafford	A/D	D
Stanhope	B/D	D
Steinsburg	B	D
Stendal	B/D	D
Stone	C/D	D
Stonelick	A	B
Strawn	D	D
Stringley	A	No Eval.
Sugarvalley	B/D	D
Summitville	C	D
Swanton	B/D	D
Switzerland	B	No Eval.
Taggart	C/D	D
Tarhollow	C	D
Tarlton	C	No Eval.
Tedrow	A/D	D
Tedrow (till substr) (Wood)	C/D	D
Teegarden	C/D	D

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1. Hydrologic Soil Groups (HSGs) for Ohio (for undisturbed naturally-occuring sites) were updated in 2008 and should be used rather than HSGs from earlier publications (http://www.oh.nrcs.usda.gov/technical/soils/OH_hsg.pdf or contact the USDA Natural Resources Conservation Service in Columbus, Ohio). You may also utilize www.OhioERIN.com to find site specific HSG (unaltered).

Soil Map Unit Component	HSG[1]	Post-Const HSG
Thackery	B	D
Thackery Variant	B	No Eval.
Thackery (till substr)	B/D	D
Thrifton	D	D
Tiderishi	C/D	D
Tilsit	D	D
Tioga	A	C
Tioga variant (Cuyahoga)	A	No Eval.
Tioga variant (Lake)	B	No Eval.
Tippecanoe	B	D
Tiro	C/D	D
Titusville	D	D
Toledo	C/D	D
Towerville	C/D	D
Trappist	C	D
Treaty	B/D	D
Tremont	C	D
Trumbull	D	D
Tuscarawas	C	No Eval.
Tuscola	C	D
Tuscola Variant	C	No Eval.
Tygart	C/D	D
Tyler	D	D
Tyner	A	A
Tyner Variant	A	No Eval.
Typic Udorthents	C	No Eval.
Uniontown	C	D
Upshur	C	D
Valley	D	D
Vandalia	C	D
Vandergrift	C/D	D
Vanlue	C/D	D
Vaughnsville	C	D
Venango	C/D	D
Vincent	C	D
Wabasha	C/D	D
Wabasha Variant	D	D
Wadsworth	D	D
Wadsworth Variant	D	D
Wakeland	B	No Eval.
Wakeman	B	No Eval.
Wallkill	B/D	D
Wallkill Variant	C/D	D
Wapahani	D	D

Soil Map Unit Component	HSG[1]	Post-Const HSG
Wappinger	B	No Eval.
Warners	C/D	D
Warsaw	B	C
Warsaw Variant	B	No Eval.
Watertown	A	A
Waupecan	B	No Eval.
Wauseon	A/D	D
Wauseon (deep to till)	A/D	D
Wayland	C/D	D
Waynetown	B/D	D
Wea	B	B
Wea Variant	B	No Eval.
Weikert	D	D
Weinbach	D	D
Wellston	B	D
Wernock	C	No Eval.
Wernock Variant	C	No Eval.
Westboro	C/D	D
Westgate	C	D
Westland	B/D	D
Westmore	C	D
Westmoreland	B	C
Wetzel	C/D	D
Weyers	A/D	D
Wharton	C	D
Wheeling	B	C
Whitaker	B/D	D
Wick	B/D	D
Wilbur	B	No Eval.
Willette	C/D	D
Williamsburg	B	C
Wilmer Variant	C	No Eval.
Woodsfield	C	C
Woolper	C	No Eval.
Wooster	C	D
Wyatt	D	D
Wynn	C	D
Xenia	C	D
Zanesville	C	D
Zepernick	B/D	D
Zipp	C/D	D
Zurich	C	No Eval.

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Hydrologic Soil Groups for Post-construction Soils

Overview

Hydrologic soil groups were created as a simple means to categorize inherent soil runoff potential and are commonly used to assign an appropriate Curve Number (CN) for hydrologic modeling purposes. Soil types have been assigned to hydrologic soil groups (HSG) in soil survey publications. In Ohio the HSGs are based on undisturbed, naturally occurring soils in an agricultural field or woodland setting. Soils properties at development sites are often changed dramatically by construction practices. Topsoil is removed, soil profiles are truncated or covered by grading activities, and exposed surfaces are compacted by heavy equipment traffic. The runoff potential is significantly impacted by these changes to the soil. This project predicts changes to HSG for soils that are altered by standard construction practices by applying the HSG criteria to modeled post-construction soil profiles.

Data for soil horizons from the USDA National Soil Information System (NASIS[1]) database were used to represent pre-construction profiles. From soil series with HSG = A, B or C, 150 soil series of significant extent in Ohio were selected for evaluation. A representative component was selected from official data sets for each series from commonly occurring map units. The standard construction practices were defined as: the removal of 18 inches of soil material from the top of the soil profile and the compaction of the zone from 0 to 6 inches at the new surface. To mirror the impact of the construction practices, layer depths in the component soil moisture table data were adjusted to reflect the removal of 18 inches (46 cm.) of soil. Similar adjustments were made to layer depths for the component soil moisture (water table) table and the component restrictions (impermeable layers) table. At the new surface, the top 6-inch (15 cm.) layer was modified in the component horizon table to show changes in infiltration caused by compaction at the surface. The USDA SPAW[2] tool was used to populate infiltration rates for the compacted soils utilizing pedon transfer functions. A report generator in NASIS was programmed to assign HSG criteria to each component. A comparison of the model's pre-construction to post-construction HSG values showed that most soils are downgraded by 1 or 2 HSG classes as a result of standard construction practices.

Methods

To calculate post-construction HSG, standard construction practices were defined as: the removal of 18 inches of soil material from the top of the soil profile and the compaction of the zone from 0 to 6 inches at the new surface.

In 2008, USDA-NRCS soil scientists in Ohio revised the HSG assigned to soil map unit data in their NASIS database. HSG were revised because of changes to Part 630 Chapter 7 of the National Engineering Handbook. Criteria for assignment of HSG was revised in Chapter 7. The published data had been compiled from manual calculations of soil profile data for each map unit. The previously published HSGs were computed on a component (soil series) basis, with representative groups based on the series typical pedon description and Soil Interpretation Record (old Soil 5 form) depths. For the revi-

1 Information regarding the USDA National Soil Information System (NASIS) database is available at http://soils.usda.gov/technical/nasis/index.html.

2 SPAW is a daily hydrologic budget model for agricultural fields and ponds developed by Dr. Keith Saxton, USDA-ARS (retired). This model includes a Soil Water Characteristics Hydraulic Properties Calculator, a program developed by Saxton and Dr. Walter Rawls USDA-ARS (retired) that can be used to estimate soil water tension, conductivity and water holding capability based on soil texture, organic matter, gravel content, salinity, and compaction. The model is available at: http://hydrolab.arsusda.gov/SPAW/Index.htm (site last updated on Oct 29, 2009).

sion, they used a report generator that calculated HSGs from published soil layer data. A large number of map units had different groups when calculated with the report generator than what had been published in the official data set. The report generator, which uses the criteria from Chapter 7 of Part 630 NEH, is run on soil map units, not components (series). Because of variation in depth to restrictive features, similar map units could receive different HSG by using the report generator. The differences in HSGs were due to changes in criteria in addition to variations between map units of the same component. In 2008 and 2009, NRCS edited their official data to show the revised HSG values. From the revised HSG values, soil components (series) with HSG = A, B or C, 150 soil series of significant extent in Ohio were selected for evaluation.

Soil component data is published by county soil survey areas in Ohio. To reflect regional variations in soil properties for a single named component, each county's component data set is unique for the occurrence of that soil type in that county – and in some counties, the component data is unique for each occurrence in a map unit. For a single component soil type named, the statewide database may contain a few, several or many unique data sets. An effort was made to select a representative component data set for each component by reviewing map unit characteristics. Map unit extent and distribution was evaluated. Preference was given to map units with larger acreage and to map units centrally located to the geographic distribution.

Layer depths in the component horizon (CH) table data were adjusted to reflect the removal of 18 inches (46 cm.) of soil. Any layer where bottom depth is less than or equal to 46 cm was deleted. Any layer where the bottom depth was greater than 46 cm and the top depth was less than 46 cm, the top depth was set at 0 cm. and 46 cm. was subtracted from the bottom depth. If the resulting layer was less than 6 cm. thick, it was deleted and the top depth of the next lowest layer was set at 0 cm. Where top depth greater than 46 cm, 46 cm was subtracted from both top and bottom depth.

The depth of two soil features that influence HSG are tracked independently of the CH table: soil water tables and soil restrictive features. Depth to soil water tables is stored in the component soil moisture (CSM) table and depth to restrictive features is stored in the component restrictions (CR) table. In both tables, top and bottom layer depths for all layers were edited by subtracting 46 cm, and values less than 0 cm edited as 0 cm.

Layer depths and Ksat values in the CH table data were adjusted to reflect creation of a 6 in. (15 cm.) zone of compacted surface during construction. If the thickness of the surface layer of the cut-soil was less than or equal to 25 cm the entire layer was used to represent the compacted zone. If it was greater than 25 cm, the upper 15 cm was replicated and modified to show compaction. The surface layer of the cut soil was copied and pasted above the original layer. The depths of the pasted layer were set at top equal to 0 cm and bottom equal to 15 cm. The top depth for the copied layer was set at top equal to 15 cm.

The USDA-ARS pedon transfer function tool 'SPAW' was used to calculate the Ksat values for the compacted surface. Ksat low range values were calculated using high clay percent and low sand percent and gravel percent; and conversely Ksat high values were calculated using low clay percent and high sand and gravel percent. Organic matter and salinity were assumed to be 0 percent. The compaction level was set at 'dense' resulting in a 110 percent compaction value.

Data used in the post-construction calculations for HSG values can be viewed in NASIS.

Load data from Area Type equal to Ohio Urban; Area equal to Ohio Urban Land; and Area Symbol equal to OHUL. Legend status equal to 'non-project'. An edit setup in the MO13 directory named "Marietta Urban" was created to view layer data that was edited in the post-construction data map units. The standard report named "EXPORT HSG data;" in the MO11 Directory was used to generate HSGs.)

Site Data

As a companion project to the development of the post-construction data set for NASIS, ODNR-DSWC soil scientist planned to gather soil profile descriptions for post-construction soils. The goal was to see how accurately the standard construction practices, as defined in our model (the removal of 18 inches of soil material from the top of the soil profile and the compaction of the zone from 0 to 6 inches at the new surface), matched actual site data gathered from the field.

Urban sites and soil types were identified for sampling. In the field, site disturbances from construction practices were verified and profile descriptions were taken from small hand-dug pits. When site conditions permitted, adjacent, undisturbed soils were also described. The extent of sampling was curtailed by staff reductions that occurred during the project.

From 13 sites, 24 profile descriptions were collected: 14 descriptions were classified as 'post-construction' and the remaining 10 descriptions were natural soils adjacent to the construction sites. The post-construction soils were judged to be cut profiles at 4 sites; fill profiles at 9 sites and 1 site was undetermined. Compaction was evaluated at the sites with a hand held penetrometer and by physical observations. At most sites compaction was rated severe in at least one horizon. The compacted horizon was not always the surface horizon.

Appendix 10: Alternative Pre-treatment Options for Dry Extended Detention Ponds - Rationale and Expectations

Research has shown that of the various mainstream stormwater BMPs (wet ponds, dry ponds, media filters, bioretention, wetlands), the suspended solids removal efficiency of dry ponds is the lowest or worst. The National Pollutant Removal Performance Database for Stormwater Treatment Practice, 2nd Edition (Center for Watershed Protection, 2000) reports the median TSS removal efficiencies for end-of-pipe controls as shown in the table below. Because of their poor water quality performance, several states no longer allow the use of dry ponds.

BMP	Median TSS Removal (%)
Dry Pond	47
Wet Pond	80
Stormwater Wetland	76
Filtering Practices	86
Infiltration Practices	95

Table 1. Median total suspended solids removal efficiences (CWP, 2000).

Ohio EPA has been interested in providing the most flexibility/options to the site designer but, with a 80% TSS removal target, the traditional dry pond designs fall short. Forebays have been shown to be effective pretreatment for all types of end-of-the-pipe stormwater BMPs, improving performance numbers significantly. A WinSLAMM (Source Loading And Management Model) analysis using solely the required 0.1*WQv volume would allow a wet pool forebay to remove upwards of 50% of the annual TSS load from most development types. Needless to say, such a forebay would significantly improve the water quality performance of dry basins.

Ohio EPA and ODNR-DSWR recognize there may be sites where, because of concerns about standing water (e.g. for safety reasons), the designer needs alternatives to a dry basin having wet pool forebays and micropools.

Figure 1. "Dry" extended detention pond with a forebay and a micropool (near the dam and the outlet).

First, the designer should consider whether the WQv requirement can be met through the use of other structural BMPs such as bioretention, enhanced swales, and/ or pervious pavement. Bioretention and enhanced swales pond water only briefly and shallowly, and would not create the same perceived threat as wet forebays and micropools. Pervious pavement does not pond water. If these BMP alternatives can be used to meet the WQv requirement, a dry basin without permanent pools can still be used to meet local peak discharge requirements.

A site can usually be divided into smaller drainage areas for WQv requirements. Bioretention works extremely well for small drainage areas, and often parking lot islands or landscape requirements may offer the needed locations/ area. If these BMP alternatives are deemed unsuitable for the site, the alternative dry basin design used to meet the WQv requirement must show performance and maintainability equivalent to a dry basin with forebay and micropool. The key considerations to address would be:

- pretreatment of runoff such that 50% of the annual TSS load is removed before discharge enters the dry basin;
- the outlet design allows for long-term function of the extended detention volume with minimal maintenance and oversight.

Pretreatment Options

> **Both filter strips and grass channels provide "biofiltering" of stormwater runoff as it flows across the grass surface. However, by themselves these controls cannot the 80% TSS removal performance goal. Consequently, both filter strips and grass channels should only be used as pretreatment measure or as part of a treatment train approach.**
>
> **(*Georgia Stormwater Management Manual*, Page 3.1-3)**

Water quality pre-treatment is provided through practices that slow, spread, filter and/or infiltrate water along its flow path. The needed level of pretreatment can be attained by using a "treatment train" approach, i.e., combining practices such as impervious area disconnection, grass filter strips, and grass swales. Another strategy is to focus these practices on treating runoff from pollutant hot spots such as parking areas driveways and roads. Our observations suggest these opportunities exist on almost every site, in-spite of the engineer's or developer's initial concerns about space limitations.

Figure 2. Disconnecting parking and storm sewers in order to reduce pollutant loads.

Preliminary parking lot runoff modeling results using WinSLAMM show that disconnecting the parking lot from the storm sewer system (i.e., placing all storm drain inlets in vegetated/ grassed collection areas with a minimum 15 ft travel distance from the parking lot) reduce both the annual runoff volume and load of total particulate solids by about 25%[1].

Grass swales can be designed to remove upwards of 50% of total solids. To provide the desired water quality depths and residence times for the water quality event, and maintaining flow velocities that prevent erosion and resuspension.

Guidance for these practices is available in the Rainwater and Land Development Manual. In addition, the Iowa Stormwater Manual provides more detailed calculations for sizing/ designing filter strips (Section 21-4) and grass swales (Section 21-2) to meet water quality targets. The Georgia Stormwater Manual and Lake County, Ohio, Swale Guidance are other useful design references.

One alternative is to incorporate the pretreatment options noted above into the design of the basin itself. The resulting basin will look more like a low, wide swale than the traditional deep-sided detention basin, and can often times be incorporated into the lawn and landscaping of the site (see photo).

Figure 3. Disconnecting parking and storm sewers in order to reduce pollutant loads.

1 WinSLAMM, Dayton 1991 rainfall, 1 Ac parking lot, clay soil

For in-basin pre-treatment, the minimum requirements allow waiving of the requirements:

- flow length that would minimum residence time of 5 minutes above the top of the WQv (see the figure below)
- max flow depth of 4” (0.33 ft)
- use manning’s n=0.15
- for HSG C&D soils, an under drain should be used to help maintain appearance and function
- designs should ensure stability (i.e., maintain flows less than max velocity) for soil, grass mix and method of establishment
- storm drain outfalls should be properly designed for stability and energy dissipation.

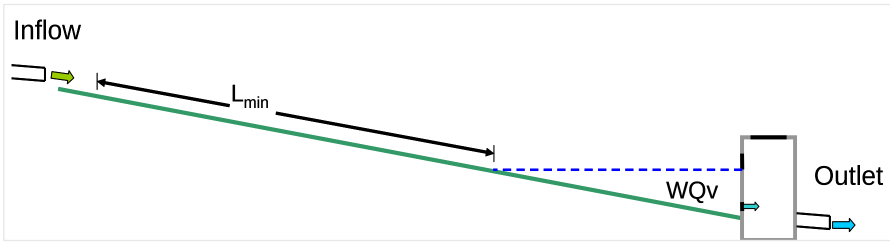

Figure 4. Alternative vegetative pre-treatment requires a flow length that allows a minimum of 5 minutes residence time above the water quality volume.

Outlet Protection

Incorporating a permanent micropool into a dry basin design allows the use of a reverse slope outlet pipe in addition to enhanced water quality treatment,. The advantage of the reverse slope pipe is that it moves the pipe entrance below the water surface protecting it from floatable debris (bottles, bags, styrofoam, leaves, etc.) that commonly blocks small (less than 4”) outlet openings at the water surface (see photos).

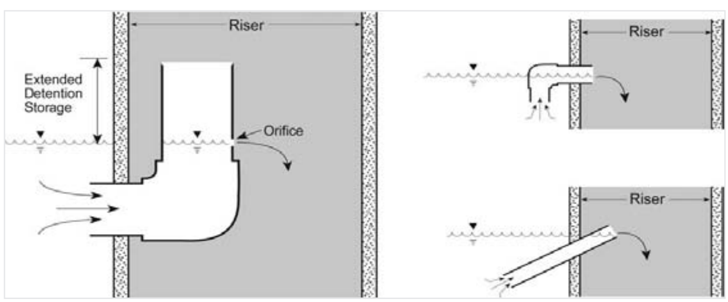

Figure 5. Reverse Slope Outlets

Figure 6. Unprotected Dry Basin Outlets

When eliminating the micropool from a WQv dry basin design, an alternative protected outlet design must be used. The protection comes from removing the controlling orifice inside the catch basin, and using a perforated lateral (or riser) and gravel filter to block any floatable materials (see the figure and photo).

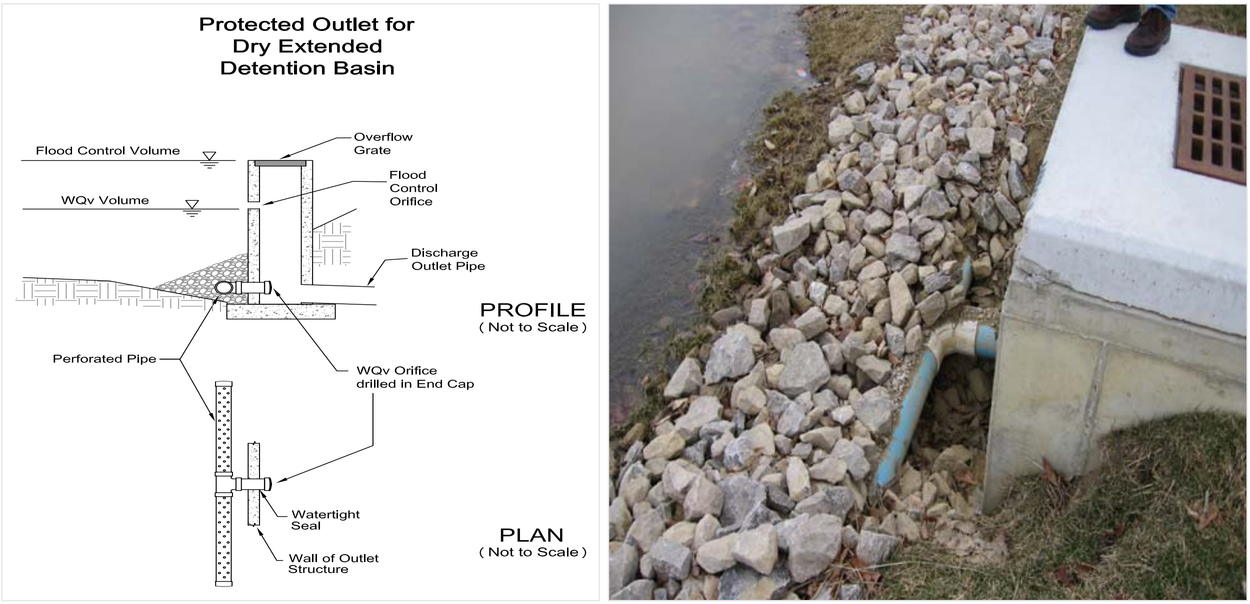

Figure 7. Protected Basin Outlets

Conclusion/Recommendation

There may be situations where a dry basin with: permanent pool forebay and micropool is not an option. In these situations, the designer should first consider alternative BMPs (bioretention, enhanced swales and/ or pervious pavement) for meetjng the WQv requirement.

Pre-treatment and outlet protection options are available that will provide equivalent performance to forebays and micropools. The designer must follow guidance to ensure that performance and maintenance goals are met.

Reference

Biohabitats. 2006. Swale Guidance, Lake County Stormwater Management Department.

CTRE. 2008. Iowa Stormwater Management Manual. Center for Transportation Research and Education, Iowa State University, Ames.

CWP. 2000. National Pollutant Removal Performance Database for Stormwater Treatment Practice, 2nd Edition. Center for Watershed Protection, Ellicott City, MD.

Haubner, S. (Editor). 2001. Georgia Stormwater Management Manual, Volume 2 - Technical Handbook. Atlanta Regional Commission.

Appendix 11: Critical Storm Method

The Critical Storm Method is a criteria recommended for controlling the peak discharge of stormwater from larger storm events (1 - 100 yr recurrence interval). It is recommended to protect property from flood damage and channel erosion, and to protect water resources from degradation resulting from accelerated stormwater flows.

In Ohio, most peak discharge control regulations reside in the requirements of a municipal, township or county government or in a stormwater . While the state of Ohio recommends the use of the Critical Storm Method for peak discharge control, actual requirements will vary according to what each community has adopted locally in conjunction with Ohio EPA NPDES permit requirements. This method has previously been included in the Ohio Stormwater Control Guidebook (ODNR, 1980), ODNR-DSWR model regulations and standards to prevent stream channel and floodplain erosion (Ohio Revised Code 1501:15-1-05).

Important Considerations

The use of this or other stormwater management criteria should assume certain conditions for adequate design, construction and continued function of stormwater management practices:

(1) Stormwater management systems must be designed for the ultimate use of the land. Areas developed for subdivisions must provide a stormwater management system for the ultimate plan of development for all of the subdivided lots.

(2) Stormwater management facilities and facilities must be designed so that they will continue to function with the least maintenance necessary.

(3) Stormwater management facilities should be designed to meet multiple objectives as much as possible. For instance pollution control, downstream channel stability, flood control, runoff reduction, and aesthetic quality are sample objectives.

(4) Stormwater management facilities and facilities shall be designed with specific regard to safety.

(5) The design criteria shall be applied to each watershed within the development area. All pre- and post-development runoff rates and volumes shall be calculated using their respective drainage divides.

The Critical Storm Method

A) In order to control pollution of public waters by soil sediment from accelerated stream channel erosion and flood plain erosion caused by accelerated stormwater runoff from development areas, the peak rates of runoff from an area after development may be no greater than the peak rates of runoff from the same area before development for all twenty-four-hour storms from one- to one-hundred-year frequency. Design and development to match the peak rate of runoff for the one, two, five, ten, twenty-five, fifty, and one-hundred year storms may be considered adequate to meet this rule.

(B)

(1) If the volume of runoff from an area after development will be greater than the volume of runoff from the same area before development, it shall be compensated by reducing the peak rate of runoff from the critical storm and all more-frequent storms occurring on the development area to the peak rate of runoff from a one-year frequency, twenty-four-hour storm occurring on the same area under predevelopment conditions. Storms of less-frequent occurrence (longer return periods) than the critical storm up to the one-hundred-year storm shall have peak runoff rates no greater than the peak runoff rates from equivalent size storms under predevelopment conditions.

(2) The critical storm for a specific development area is determined as follows:

(a) Determine the total volume of runoff from a one-year frequency, twenty-four-hour storm, occurring on the development area before and after development.

(b) From the volumes in paragraph (B)(2)(a) of this rule, determine the per cent of increase in volume of runoff due to development and, using this percentage, select the critical storm from this table:

If the percent of increase in runoff volume is		The critical storm for peak rate control will be
equal to or greater than	and less than	
-	10	1 year
10	20	2 year
20	50	5 year
50	100	10 year
100	250	25 year
250	500	50 year
500	-	100 year

Table 1-1 Critical storm determination using percent of increase in runoff volume.

(C) Methods for controlling increases in stormwater runoff peaks and volumes may include but are not limited to:

(1) Retarding flow velocities by increasing friction; for example, grassed road ditches rather than paved street gutters where practical, discharging roof water to vegetated areas, or grass and rock-lined drainage channels.

(2) Grading and use of grade control structure to provide a level of control in flow paths and stream gradients.

(3) Induced infiltration of increased stormwater runoff into the soil where practical; for example, constructing special infiltration areas where soils are suitable, retaining topsoil for all areas to be vegetated, or providing good infiltration areas with proper emergency overflow facilities.

(4) Provisions for detention and retention; for example, permanent ponds and lakes with stormwater basins provided with proper drainage, multiple-use areas for stormwater detention and recreation, wildlife, or transportation, or subsurface storage areas.

Reference: Goettemoeller, R.L., D.P. Hanselmann, and J.H. Bassett. 1980. Ohio Stormwater Control Guidebook. Ohio Department of Natural Resources, Division of Soil and Water Districts, Columbus. http://www.dnr.state.oh.us/soilandwater/water/urbanstormwater/default/tabid/9190/Default.aspx