Omaha Regional Stormwater Design Manual

Stormwater Best Management Practices

Chapter 8

Revised June 2014

City of Omaha Environmental Quality Control Division
www.omahastormwater.org

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List of Acronyms and Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
ADA	American Disabilities Act
ADT	Average Daily Traffic
BMPs	Best Management Practices
BSM	Bioretention Soil Mixture
CALTRANS	California Department of Transportation
CASQA	California Stormwater Quality Association
CFU	Colony Forming Unit
CO	Certificate of Occupancy
CRS	Community Rating System
CSO	Combined Sewer Overflow
CU	Copper
DOT	Department of Transportation
EDDB	Extended Dry Detention Basin
EMC	Event Mean Concentration
ET	Evapotranspiration
ETJ	Extra Territorial Jurisdiction
FPS	Feet per Second
FT	Feet
HEC-HMS	Hydrologic Engineering Centers – Hydrologic Modeling System
HR	Hour
HRT	Hydraulic Residence Time
HSG	Hydrologic Soil Group
IN	Inch(es)
ISBMPD	International Stormwater Best Management Practices (BMP) Database
ITE	Institute of Transportation Engineers
LEED	Leadership in Energy and Environmental Design
LEED-ND	Leadership in Energy and Environmental Design Neighborhood Development
LID	Low Impact Development
LTCP	Long Term Control Plan
MARC	Mid America Regional Council
mg/L	milligram/liter
mm	millimeter
MPN	Most Probable Number
MS4	Municipal Separate Storm Sewer Systems
NAVD 88	North American Vertical Datum 1988
NFIP	National Flood Insurance Program
NOAA	National Oceanic and Atmospheric Administration
NPDES	National Pollutant Discharge Elimination System
NRCS	Natural Resources Conservation Service
NWS	National Weather Service
PCSMP	Post-Construction Stormwater Management Plan
PCWP	Papillion Creek Watershed Partnership
PVC	Polyvinyl Chloride
Q_{WQ}	Water Quality Discharge

SQ FT	Square Feet
SSI	Sustainable Sites Initiative
SWMM	EPA's Storm Water Management Model
TKN	Total Kjeldahl Nitrogen
TMDL	Total Maximum Daily Load
TN	Total Nitrogen
TP	Total Phosphate
TSS	Total Suspended Solids
UDFCD	Urban Drainage and Flood Control District of Denver, Colorado
UIC	Underground Injection Control
USACE	U.S. Army Corp of Engineers
USDA	U.S. Department of Agriculture
USEPA	United States Environmental Protection Agency
USGS	U.S. Geological Survey
WinSLAMM	Source Loading and Management Model for Windows
WMM	Watershed Management Model
WQCV	Water Quality Control Volume
µg/L	microgram/liter
µm	micrometer

List of Calculation Variables

A	=	Area
A_F	=	Filter bed surface area
A_{FB}	=	Forebay surface area
A_{OT}	=	Outlet area for cage openings
A_T	=	Tributary area
A_{TR}	=	Minimum required trash rack coverage area
C	=	Median effluent concentration of BMP (as relates to pollutant load exiting BMP)
C	=	EMC (both - as relates to pollutant load entering BMP)
C	=	Overland flow runoff coefficient for cover type
C_V	=	V-notch weir coefficient
D	=	Overland flow distance parallel to slope
D_{AR}	=	Depth of the aggregate reservoir
d_f	=	Planting soil bed depth
D_O	=	Orifice diameter
D_P	=	Diameter of the outlet pipe
E	=	Effluent pollutant load
FS_{MIN}	=	Minimum filter length
G_{pipe}	=	Underdrain pipe
$H_{\#yr}$	=	Average head for storm (#) event
h_{avg}	=	Average ponding depth above plant in soil bed (feet) = (H_{max} / 2)
H_{EDDB}	=	Average head of V_{EDDB}
h_{max}	=	Ponding depth

$I_{\#}$	=	Rainfall intensity
I_A	=	Percent impervious
I_E	=	Effective imperviousness
k	=	Coefficient of soil permeability
L	=	Annual load
L	=	Length
L_{AR}	=	Length of the aggregate reservoir
L_f	=	Filter bed length
L_{FB}	=	Length between flow boundaries
L_S	=	Filter bed length
n	=	Manning's n-coefficient
n_r	=	Number of equally sized subreservoirs
N_{TU}	=	Number of transverse collector pipes
O	=	Outflow volume in watershed inches
P	=	Annual rainfall
P_{AR}	=	Porosity of the aggregate reservoir
P_j	=	Fraction of annual rainfall events that produce runoff (usually 0.9)
PL_{MAX}	=	Maximum pavement length
P_W	=	Wetted perimeter
P_{WQ}	=	Water quality rainfall event
Q_{AVG}	=	Storm average flow rate/discharge rate
Q_{DS}	=	Peak design flow rate
Q_{WQ}	=	Water quality discharge
R	=	Annual runoff (Inflow)
$R_{\#}$	=	(Number of days)-Day wet season rainfall
R_H	=	Hydraulic radius
R_V	=	Runoff coefficient and volumetric runoff coefficient
S	=	Slope of overland flow path
S_L	=	longitudinal slope
s_p	=	Slope of the outlet pipe
s_S	=	Slope of the subsoil
S_{TU}	=	Transverse collector pipe spacing
t_f	=	Time required for V_D to filter through soil
T_I	=	Time of concentration to the most upstream inlet or entry point
$V_{\#yr}$	=	Storm (#) event volume
V_D	=	Design volume
V_{EDDB}	=	EDDB volume
V_{FB}	=	Forebay volume
V_P	=	Permanent pool volume
W_{AR}	=	Width of the aggregate reservoir
WQCV	=	Water quality volume
W_S	=	Width of swale
W_V	=	Top width of V-notch weir
θ	=	Required V-notch weir angle
V	=	Velocity

Definitions

1. Baseline Land Use Conditions. That which existed for Year 2001 for Big and Little Papillion Creeks and its tributaries (excluding West Papillion Creek) and for Year 2004 for West Papillion Creek and its tributaries.

2. Best Management Practice (BMP). A technique, measure or structural control that is used for a given set of conditions to manage the quantity and improve the quality of stormwater runoff in the most cost-effective manner.*[Source: U.S. Environmental Protection Agency (EPA)]*

3. Comprehensive Development Plans. Existing plans developed by local jurisdictions that serve as the basis for zoning and other land use regulations and ordinances. The Stormwater Management Policies are to be incorporated into the respective Comprehensive Development Plans.

4. Full Build-Out Land Use Conditions. Fully platted developable land use conditions for the combined portions of the Papillion Creek Watershed that lie in Douglas and Sarpy Counties that are assumed to occur by the Year 2040, plus the projected 2040 land uses within the Watershed in Washington County; or as may be redefined through periodic updates to the respective County comprehensive plans.

5. Green Infrastructure. The USEPA defines green infrastructure as stormwater infrastructure that uses vegetation and soil to manage rainwater where it falls. By weaving natural processes into the built environment, green infrastructure provides not only stormwater management, but also flood mitigation, air quality management, and supports sustainable communities.

6. Low-Impact Development (LID). A land development and management approach whereby stormwater runoff is managed using design techniques that promote infiltration, filtration, storage, evaporation, and temporary detention close to its source. Management of such stormwater runoff sources may include open space, rooftops, streetscapes, parking lots, sidewalks, medians, etc.

7. Peak Discharge or Peak Flow. The maximum instantaneous surface water discharge rate resulting from a design storm frequency event for a particular hydrologic and hydraulic analysis, as defined in the Omaha Regional Stormwater Design Manual. The measurement of the peak discharge shall be at the lower-most drainage outlet(s) from a new development or significant redevelopment.

8. Post-Construction Stormwater Management Plan (PCSMP). A PCSMP is a required part of the NPDES Phase II Stormwater Permits issued to many of the Omaha metropolitan area Papillion Creek Watershed Partnership (PCWP) members. Development of Stormwater Management Policies is an integral part of the PCSMP, and such policies are to be adopted by respective PCWP partners.

9. Regional Stormwater Detention Facilities. Those facilities generally serving a drainage catchment area of 500 ac. or more in size.

10. Total Maximum Daily Load (TMDL). A calculation of the maximum amount of a pollutant that a waterbody can receive and still meet water quality standards, and an allocation of that amount to the pollutant's sources. Water quality standards are set by States, Territories, and Tribes. They identify the uses for each waterbody, for example, drinking water supply, contact recreation (swimming), and aquatic life support (fishing), and the scientific criteria to support that use. A TMDL is the sum of the allowable loads of a single pollutant from all contributing point and non-

point sources. The calculation must include a margin of safety to ensure that the waterbody can be used for the purposes the State has designated. The calculation must also account for seasonal variation in water quality. The Clean Water Act, Section 303, establishes the water quality standards and TMDL programs, and for Nebraska such standards and programs are administered by the Nebraska Department of Environmental Quality.
[Source: USEPA and Nebraska Surface Water Quality Standards, Title 117].

11. Water Quality LID. A level of LID using strategies designed to provide for water quality control of the first ½ in. of stormwater runoff generated from each new development or significant redevelopment and to maintain the peak discharge rates during the 2-year storm event to baseline land use conditions, measured at every drainage (stormwater discharge) outlet from the new development or significant redevelopment.

Chapter 8 Stormwater Best Management Practices

8.1 Overview

8.1.1 Introduction

The intent of this Chapter is the proper selection, design, implementation, and maintenance of post-construction water quality Best Management Practices (BMPs) for new developments and re-development efforts. This Chapter provides information and guidance regarding the selection and design of selected BMPs. Implementation of BMPs is expected to reduce pollutants in stormwater runoff and receiving waters, improving the water quality and environment of the community.

Urban runoff carries with it a wide variety of pollutants from diverse and diffuse sources. Pollutants associated with urban runoff often occur in higher concentrations than found in runoff prior to development. In addition, urban runoff can contain pollutants that may not be present in surface runoff from undeveloped land (such as household, commercial and industrial chemicals, and petroleum products).

A high percentage of stormwater pollutant loading is associated with the runoff from smaller, frequent storms over a typical year. For the Omaha region the first 0.5 in. of runoff, also called the water quality control volume (WQCV), is used to estimate the volume produced by these smaller frequent storms which carries a high percentage of stormwater pollutant load. To reduce the concentrations and the loads of these pollutants that reach the receiving waters, a system of stormwater BMPs should be implemented to control the first 0.5 in. of runoff as part of a new development or redevelopment. BMPs are defined as measures that function to either keep pollutants from entering stormwater or remove pollutants from stormwater. Various BMPs have been implemented throughout the United States (U.S.). In general, they can be categorized as either structural or nonstructural.

8.1.2 Structural and Non-structural BMPs

Structural BMPs can be thought of as constructed facilities designed to reduce runoff and/or passively treat urban stormwater runoff before it enters the receiving waters. Non-structural BMPs consist of pollution prevention BMPs and source control BMPs. Both structural and non-structural BMPs are used for erosion control during construction and after construction (post construction). A detailed discussion of sediment and erosion control is presented in Chapter 9. This Chapter (Chapter 8) discusses BMPs appropriate for control of post-construction stormwater runoff.

The selection of the most appropriate BMPs for a given site or basin is largely dependent on whether the site is undeveloped or already developed. In areas with existing development, non-structural BMPs should be considered with structural BMPs to improve water quality when space is limited. Structural BMPs are generally more appropriate for new development and significant redevelopment, where they can be integrated into overall planning of the infrastructure and space is available. Because stormwater pollution is varied in nature and impact, no single BMP will fit all situations. BMPs must be tailored to fit the needs of particular sources and circumstances. An effective strategy for minimizing stormwater pollution loads is to use both types of BMPs, structural and nonstructural. The proper selection and implementation of both types of BMPs can provide water quality enhancement that minimizes pollutant loads being transported to the receiving waters.

8.2 BMP Selection and Implementation Guidelines

The proper selection and implementation of structural and non-structural BMPs begins early in the site design process through a comprehensive look at project objectives and performance standards.

8.2.1 Define Project Objectives and Performance Standards

Numerous objectives and performance standards affect the type, location, and size of the stormwater BMPs appropriate for the site. Project objectives are the goals or purpose for the project, while performance standards are the level of function or operation required to meet project objectives. Project objectives are divided into three categories:

1. ***Community Objectives and Performance Standards,*** defining how the project and its stormwater BMPs meet the health, safety, and welfare objectives of the City of Omaha, partially defined by the City's land use objectives through its comprehensive plans, zoning regulations, subdivision regulations, and building standards.

2. ***Environmental Objectives and Performance Standards,*** defined by pertinent stormwater regulations in Municipal Code Section 32 Article V coupled with other pertinent local, state and federal environmental regulations and goals. For Omaha, this is to provide for water quality control of the first 0.5 in. of runoff from the site and maintain a "No Adverse Impact" condition.

3. ***Financial Objectives and Performance Standards,*** typically expressed as the life cycle costs to build and maintain the development and its infrastructure coupled with the projects anticipated marketability and revenue stream.

<u>Improvement Plans</u> must document that pertinent project objectives and performance standards are satisfied. A successful project is typically one that achieves community, environmental and financial objectives and performance standards in an integrated fashion. Ideally, these objectives and performance standards would be provided upon request and/or during pre-submittal meetings. Developers, in turn, are encouraged to document their critical objectives and performance standards, facilitating a dialogue during the pre-submittal meeting oriented toward identifying and, if possible, resolving incompatible objectives before significant investment in project design is made. <u>Concept plans</u> should illustrate how the site layout addresses critical objectives and performance standards. <u>Final improvement plans</u> should be evaluated for their ability to satisfy the minimum required objectives as listed above and any additional objectives and performance standards for the site.

8.2.1.1 Community Objectives and Performance Standards

Community objectives and performance standards control the infrastructure necessary to sustain a development. Stormwater quality goals are often supported by defining ways to achieve community performance standards with design criteria that accentuate community values while minimizing health and safety concerns. Additional considerations include:

1. ***Building Density / Lot Size*** –Zoning regulations and subdivision regulations often dictate allowable building densities and lot sizes for specific land uses and, consequently, the stormwater volume, rate, and pollutant load. Impervious area reductions, stream buffer zones, and stormwater BMP sites may be accommodated by regulations in Municipal Code Section 53-11 Cluster Subdivisions that allow clustering to protect open spaces.

2. ***Traffic and Pedestrian Considerations*** – Impervious area reductions and use of permeable paving materials for stormwater control must be weighed against the necessary number of roadway lanes and their widths to meet traffic flow and parking requirements (automobiles, trucks/buses, bicycles). The streetscape may provide sites for many types of decentralized BMPs, providing that anticipated routine traffic patterns, access of emergency and construction vehicles, and public transportation vehicles are accommodated. Curb bumpouts may provide locations for BMP implementation as well as traffic calming benefits. Sidewalks and/or other pedestrian pathway designs must address both anticipated uses and their runoff contribution.

3. ***Pavement Strength*** – Pavement must be designed to provide the necessary structural support associated with the average daily traffic (ADT) volumes. Permeable pavements may be possible if projected traffic volumes and types are compatible.

4. ***American Disabilities Act (ADA) Considerations*** – Project features should accommodate persons with disabilities. Curbless designs, medians midway across busy roads (where stormwater BMPs can be located) and narrower pavement widths (reducing impervious area) are all compatible with ADA and stormwater goals.

5. ***Utilities*** – The project should consider the location and maintenance of existing and new utilities and ease of maintenance access, seeking to minimize interferences with potential BMP sites and incursions into or crossings of stream buffer areas.

6. ***Space Considerations*** –Existing open space both within and adjacent to the site should be considered for locating stormwater features and maintenance access.

7. ***Vegetation and Landscaping*** – Landscaping ordinances may require certain varieties and densities of vegetation. These ordinances should be reviewed to also identify water and/or salt-tolerant species suitable for vegetated structural BMPs.

8. ***Aesthetics*** – The design should consider the desired look and feel for the area where the project is located. Most BMP design criteria can be adjusted to accommodate aesthetic goals, and still achieve stormwater performance standards.

9. ***Maintenance*** – Maintenance activities, access, and costs should be considered during project design.

8.2.1.2 Environmental Objectives and Performance Standards

Environmental objectives and performance standards typically are framed by local, regional and state environmental regulations, and permit requirements. Many environmental objectives also achieve community objectives, enhancing the overall quality of life of residents. This section discusses how to establish environmental objectives and performance standards based on these regulations, permit requirements, and other considerations that typically affect projects in the Omaha region.

In the Omaha region, stormwater management regulations fall under the various National Pollutant Discharge Elimination System (NPDES) permits. The NPDES program regulates the quality of stormwater runoff. Post-construction stormwater management plan (PCSMP) requirements evolved as a program requirement in the Municipal Separate Storm Sewer Systems (MS4), but it applies throughout Omaha's corporate limits plus a 3 mile extra territorial jurisdiction (ETJ).

The PCSMP requirements apply to new land development and significant redevelopment that discharge to the City's MS4 or combined sewer system. New land development includes development projects in areas not previously built to urban uses (including but not limited to farmland, pasture, woodland, and green space). Significant redevelopment includes development projects in areas that are currently built to urban and suburban land uses, and are being revitalized with rehabilitation of existing structures, or demolition of existing structures and construction of new ones. These developments are required to control the WQCV.

In addition to controlling the WQCV, new development or re-development projects must maintain a "No Adverse Impact" condition. "No Adverse Impact" for the purpose of meeting the post-construction stormwater requirements is defined as no increase in the pre-project runoff rate for the 2-, 10-, and 100-year runoff rate for post project conditions. The only exception to this requirement is in cases where the project discharges *directly* to one of the existing regional detention facilities associated with the Papillion Creek Watershed (i.e., Zorinsky Lake, Glenn Cunningham Lake, or Lake Wehrspann) or to the Missouri River. Other demonstrations of a "No Adverse Impact" may be possible on a case-by-case basis.

The design, implementation, and maintenance guidelines provided in this Chapter are intended to assist developers in meeting the performance standards outlined in Municipal Code Section 32 Article V and are primarily aimed at providing control of the WQCV. Many of the BMPs discussed here can also be used to provide peak flow attenuation in order to achieve a "No Adverse Impact" condition. Chapter 2, Chapter 6 and Chapter 8 should be referenced when using BMPs for treatment of the WQCV and peak flow attenuation. BMPs not included in this document may be used in new development and redevelopment projects as long as they are preapproved by the City and are designed to control the WQCV.

Projects discharging to the combined sewer system (generally those east of 72nd Street) must control runoff such that there is no net increase in runoff from pre-development conditions as they existed in October 2002 for 2-, 10-and 100-year storm events. In addition, the City of Omaha may require stormwater detention in areas where there is not adequate downstream sewer capacity. The applicant should meet with the City of Omaha Public Works Department to verify these requirements for each individual development.

8.2.2 Selecting BMPs

Proper selection of BMPs includes considering the targeted pollutant removal to meet downstream water quality objectives, physical site design, cost, watershed objectives, and any special situations that can make BMP implementation challenging.

8.2.2.1 Target Pollutant Removal

Identifying pollutants of concern and then selecting BMPs which target those pollutants for removal is essential for meeting overall water quality goals. The Nebraska Department of Environmental Quality Section 303d List of Impaired Waterbodies should be consulted to determine if the project drains to a waterbody with documented water quality impairment. In some cases the waterbody may have an approved Total Maximum Daily Load (TMDL). The TMDL may include BMP recommendations for the treatment of stormwater runoff for the constituent of concern and should be consulted prior to selecting BMPs draining to waters with an established TMDL.

While currently not required to meet the City's post-construction stormwater performance criteria, estimating the expected annual pollutant loads from the developed area will assist in determining target pollutants. Estimating BMP pollutant removal performance helps in selecting post-construction BMPs that are most effective in removing the targeted constituents from site runoff. The intent of providing the information below is to aid in BMP selection. Estimating annual pollutant loads and anticipated pollutant removal is not required for PCSMP approval.

Estimating Annual Pollutant Loads from Developed Areas

Several computer models are equipped to simulate annual pollutant loads from urban areas and pollutant removal by common BMP types. One program is the Watershed Management Model (WMM) which was developed by Camp Dresser & McKee Inc. (CDM) as part of the Rouge River Wet Weather National Demonstration Project partially funded by a U.S. Environmental Protection Agency (USEPA) grant. The WMM software and user's manual can be downloaded at http://www.rougeriver.com/proddata/wmm.html. The Source Loading and Management Model for Windows (WinSLAMM) performs pollutant load calculations and is available from PV & Associates, LLC for a fee. The software can be downloaded at http://www.winslamm.com/winslamm_updates.html. USEPA's Storm Water Management Model (SWMM) is also capable of performing pollutant load estimates. Information on the USEPA SWMM model is available on-line at http://www.epa.gov/nrmrl/wswrd/wq/models/swmm/.

To quickly calculate by hand the expected pollutant load from an urban area, the Stormwater Center's Simple Method to Calculate Urban Stormwater Pollutant Loads can be used. The single method uses pollutant EMCs and runoff volume to estimate the mass of pollutants in runoff from a site. The Stormwater Center has summarized event mean concentrations (EMCs) of pollutants from different land uses. A summary of the Stormwater Center data is shown in Table 8-1. The model default values in Table 8-1 represent best professional judgment of the Stormwater Center, and give additional weight to studies conducted at a national level. The values do not incorporate studies in arid climates. EMCs for bacteria have also been published by the Papillion Creek Watershed Partnership (PCWP) and are shown in Table 8-2.

Table 8-1
Simple Method Model Default Value[1] EMC

Pollutant	Land Use			
	Residential	Commercial	Roadway	Industrial
Total Nitrogen (TN) (mg/l)	2.2	2	3	2.5
Total Phosphorus (TP) (mg/l)	0.4	0.2	0.5	0.4
Total Suspended Solids (TSS) (mg/l)	100	75	150	120

[1]The model default values represent best professional judgment of the Stormwater Center, and give additional weight to studies conducted at a national level. Data does not incorporate studies on arid climates.
Source: Stormwater Center Website, accessed July 2011

Table 8-2
Possible Sources and Concentrations of Fecal Coliform and E. coli in the Papillion Creek Drainage Basin

Land Use Category	Effective Percent Impervious	Fecal Coliform Bacteria (CFU/100 mL)	Equivalent E. Coli Bacteria Loading (CFU/100 mL)
Agriculture	Varies	88,400	55,700
Parks and Open Areas	5%	11,600	7,300
Rural Estate (Homes on 3 to 10 acres)	10%	17,100	10,800
Low Density Residential (Homes on 1 to 3 acres)	16%	23,700	14,900
Medium Density Residential (Homes on approx. 0.25 Acres)	38%	48,100	30,300
Churches, Schools, and Civic	50%	61,300	38,600
High Density Residential (Multi-Family Apartment Complexes)	65%	77,900	49,100
Industrial Areas	72%	85,600	53,900
Commercial and Retail Businesses	85%	100,000	63,000

CFU = Colony Forming Units
Source: Final Papillion Creek Watershed Management Plan (PCWP, April 2009)

The EMCs in Table 8-1 and Table 8-2 can be used to estimate the pollutant loads from a proposed development site. The equations used in the Simple Method are included in Appendix A along with an example calculation of estimating pollutant loads from an urban development.

Estimating Pollutant Removal Performance from BMPs

The pollutant load leaving a BMP is a function of the volume of water leaving the BMP and the effluent concentration of the pollutant. Appendix A provides a method for calculating the pollutant removal effectiveness of BMP types by comparing the pollutant load entering the BMP to the pollutant load exiting the BMP. As recommended by the International Stormwater BMP Database (ISBMPD), pollutant loads exiting the BMP are estimated using effluent concentrations and outflow volumes. **Actual pollutant removal performance for a particular BMP can only be verified using post-construction monitoring data.**

The ISBMPD publishes median influent and effluent concentrations of select pollutants for several BMP types as shown in Table 8-3. The information in Table 8-3 can be used along with estimates of outflow volume to calculate an estimated pollutant load exiting a BMP.

In some instances, the volume exiting the BMP is equal to the volume entering the BMP; however, some BMP types have been shown to reduce runoff volume through infiltration and evapotransporation. The ISBMPD published results from a study showing relative volume reduction estimates for select BMPs types (Geosyntec Consultants and Wright Water Engineers, Inc., 2011). Table 8-4 summarizes the percent volume reduction seen from grass strips and swales, bioretention, and dry detention basins. The information in Table 8-4 can be used in a planning level analysis to estimate outflow volume from the BMP as a function of inflow runoff volume. An example of this calculation is provided in Appendix A. It would not be appropriate to use the information in Table 8-4 to determine the exact outflow volume for an individual BMP. In addition, relative volume reduction may be estimated using continuous simulation modeling as discussed under Section 8.3.3. **Actual relative volume reduction can only be verified using post-construction monitoring data.**

Table 8-3
Structural BMP Median Influent and Effluent Concentrations from the ISBMPD

Constituents	Sample Location	Detention Pond (n=25)[1]	Wet Pond (n=46)[1]	Wetland Basin (n=19)[1]	Biofilter (n=57)[1]	Media Filter (n=38)[1]	Porous Pavement (n=6)[1]
Suspended Solids (mg/L)	Influent	72.65	34.13	37.76	52.15	43.27	
	Effluent	31.04	13.37	17.77	23.92	15.86	16.96
Total Cadmium (µg/L)	Influent	0.71	0.49	0.36	0.54	0.25	
	Effluent	0.47	0.27	0.24	0.30	0.19	xx
Dissolved Cadmium (µg/L)	Influent	0.24	0.19		0.25	0.16	
	Effluent	0.25	0.11	xx	0.21	0.13	xx
Total Copper (µg/L)	Influent	20.14	8.91	5.65	31.93	14.57	
	Effluent	12.10	6.36	4.23	10.66	10.25	2.78
Dissolved Copper (µg/L)	Influent	6.66	7.33		14.15	7.75	
	Effluent	7.37	4.37	xx	8.40	9.00	xx
Total Chromium (µg/L)	Influent	7.36	6.00		5.63	2.18	
	Effluent	3.18	1.44	xx	4.64	1.48	xx
Total Lead (µg/L)	Influent	25.01	14.36	4.62	19.53	11.32	
	Effluent	15.77	5.32	3.26	6.70	3.76	7.88
Dissolved Lead (µg/L)	Influent	1.25	3.40	0.50	2.25	1.44	
	Effluent	2.06	2.48	0.87	1.96	1.18	xx
Total Zinc (µg/L)	Influent	111.56	60.75	47.07	176.71	92.34	
	Effluent	60.20	29.35	30.71	39.83	37.63	16.60
Dissolved Zinc (µg/L)	Influent	26.11	47.46		58.31	69.27	
	Effluent	25.84	32.86	xx	25.40	51.25	xx
Total Phosphorus (mg/L)	Influent	0.19	0.21	0.27	0.25	0.20	
	Effluent	0.19	0.12	0.14	0.34	0.14	0.09
Dissolved Phosphorus (mg/L)	Influent	0.09	0.09	0.10	0.09	0.09	
	Effluent	0.12	0.08	0.17	0.44	0.09	xx
Total Nitrogen (mg/L)	Influent	1.25	1.64	2.12	0.94	1.31	
	Effluent	2.72	1.43	1.15	0.78	0.76	xx
Nitrate-Nitrogen (mg/L)	Influent	0.70	0.36	0.22	0.59	0.41	
	Effluent	0.58	0.23	0.13	0.60	0.82	xx
TKN (mg/L)	Influent	1.45	1.26	1.15	1.80	1.52	
	Effluent	1.89	1.09	1.05	1.51	1.55	1.23

[1] Actual number of BMPs reporting a particular constituent may be greater or less than the number reported in this table, which was based on number of studies reported in database based on BMP category.
Notes: xx- Lack of sufficient data to report median and confidence interval. Differences in median influent and effluent concentrations do not necessarily indicate that there was a statistically significant difference between influent and effluent. See Geosyntec and Wright Water Engineers, Inc. 2008a "Analysis of Treatment System Performance", for more detailed information. Table source: Geosyntec Consultants and Wright Water Engineers, Inc. 2008b.

Table 8-4
ISBMPD Percent Volume Reduction

BMP Category	25th Percentile	Median	75th Percentile	Average
Biofilter – Grass Strips	18%	34%	54%	38%
Biofilter – Grass Swales	35%	42%	65%	48%
Bioretention	45%	57%	74%	61%
Detention Basins – Surface Grass Lined	26%	33%	43%	33%

Notes: Relative Volume Reduction = Study Total Inflow Volume – Study Total Outflow Volume / Study Total Inflow Volume. Precipitation in dataset ranged from 0.08 inches to 7.0 inches for grass strips, 0.05 to 4.0 inches for swales, 0.09 to 5.3 inches for bioretention, and 0.06 to 9.3 inches. Design criteria for each BMP in the study vary. Source: Geosyntec Consultants and Wright Water Engineers, Inc. 2011

8.2.2.2 Physical Site Design Considerations

Physical site conditions may exist that make the proper design and construction of BMPs challenging. These site conditions include:

- Infiltration potential and site slope
- Size and type of proposed development

Infiltration Potential and Site Slope

The infiltration rate of the soils and the subsurface volume available for storing water determines the infiltration potential at a site. Soils with a low infiltration rate and a small amount of subsurface volume available for storing water are less applicable for the construction of BMPs that rely on infiltration to function properly. Conversely, sites with high infiltration rates reduce the time that water can be ponded at the surface, limiting the feasibility of a wet pond. BMPs which rely on infiltration are highly applicable if groundwater, clay or bedrock depth is greater than 5 ft. and the soils beneath the BMP are comprised of hydrologic soil group (HSG) A or B soils; however, infiltration of a portion of the WQCV is possible in HSG C or D soils. In some cases, stormwater infiltration BMPs may be regulated under the Safe Drinking Water Act as Class V wells under the Underground Injection Control (UIC) program. Most BMPs do not meet the Class V well definition and can be installed without regulation oversight by the UIC program. The USEPA memorandum included in Appendix B provides clarification on which stormwater infiltration BMPs have the potential to be regulated as Class V wells. Table 8-5 summarizes BMP application based on HSG.

Sites with steep slopes may prohibit the use of BMPs that require sheet flow through the system such as filter strips and swales unless such controls are terraced and/or flow across the slope. Steep slopes concentrate water quickly which increases the velocity and erosion and scour potential. Table 8-5 indicates the recommended slopes for BMP application based on the BMP categories discussed in this manual. Slopes steeper than 15 percent should be investigated by a geotechnical engineer prior to the design and construction of structural BMPs that infiltrate runoff into the subsurface.

Table 8-5
Application of BMPs Based on Infiltration Potential and Slope

BMP Category	Recommended Slope	HSG	
		A or B	C or D
Bioretention	<10%	H	H
Constructed Wetlands	0-2%	M[2]	H
Extended Dry Detention Basin	0-2%	H	L
Grassed Swales	0-2%	H	M
Filter Strips	0-6%	H	M
Eco-Roof – Green Roof	2-25%	H	H
Eco-Roof – Roof Garden	2-10%	H	H
Manufactured Systems[1]	N/A	N/A	N/A
Permeable Pavements	0-5%	H[2]	M[3]
Retention Wet Ponds	0-2%	M[4]	H
Lot-Level Rain Gardens[5]	<8%	H	L
Lot-Level Rain Barrel and Cisterns	Any	H	H
Disconnection of Impervious Areas	Any	H	H
H	High applicability		
M	Medium applicability		
L	Low applicability		

[1] N/A = Not applicable for manufactured systems because this category contains many different BMP types and cannot be generalized
[2] Especially if depth to clay layer, bedrock or groundwater is greater than 5 feet.
[3] Underdrain may be required.
[4] Clay liner may be required.
[5] Per Rodie et al., 2010. Rain Gardens, Bioswales and Xeric Gardens: A Manual for Homeowners and Small Properties in Omaha.

Size and Type of Proposed Development

The size of the drainage area and its characteristics influence the amount of runoff to each BMP making different BMPs suited to different sized drainage areas. As discussed in Section 8.2.2.1, the type of development also influences the types of pollutants that are transported by runoff from the land use. For example, industrial areas typically have higher concentrations of metals than residential areas. Table 8-6 provides guidance in selecting BMPs for a given site, based on type of development and the drainage area to the BMP. The table provides applicability guidelines for each BMP in relation to the type of development. For example, bioretention has a high applicability to development of commercial sites, based on the expected drainage area to the BMP and the water quality control that the BMP provides.

Table 8-6
BMP Application Based on Drainage and Size and Type of Development

BMP	Agricultural and Park Land	Residential Large Lot >2 acre	Residential Small Lot <2 acre	Multi-Family	Commercial	Industrial[2]	Streets/ Parking Lots	Desired Drainage Area to BMP
Lot-Level BMPs	M	H	H	H	M	M	M	< 1/8 acre
Bioretention	M	M	M	M	H	H	H	< 4 acres
Vegetated Swale	M	H	L	M	M[1]	M	M	< 5 acres
Constructed Wetland	H	S	S	S	H, S	H, S	M	Water budget > 40 acres
Filter Strips	H	H	M	M	H	H	H	< 2 acres[3]
Eco-Roof	L	L	L	M	H	H	L	< 1 acre
Extended Dry Detention	H	S	S	S	H, S	H, S	M	> 10 acres
Retention Wet Pond	H	S	S	S	H, S	H, S	M	Water budget > 40 acres
Permeable Pavement	L	L	L	H	H	M	H	< 2 acres
H	High applicability							
M	Medium applicability							
L	Low applicability							
S	Subdivision level applicability							

[1] Consider trash and floatables during selection and design.
[2] Consider potential ground water pollution risk during selection and design.
[3] Limit concentrated flow.

Specific policy regarding implementation of BMPs in relation to development, redevelopment, and public improvement projects will be defined on a case-by-case basis by City staff.

8.2.2.3 Cost Considerations

The objective of most development projects is to build a marketable project for the least possible cost. Communities and/or the successor property owners are concerned about the long-term sustainability of the project and its infrastructure, including its maintenance requirements. Sustainable stormwater management practices should address the following cost considerations:

- ***Land Easement and Acquisition Costs*** – Land that is required for public easement is land that cannot be used for building construction. Project features that achieve multiple objectives can minimize easement requirements as they allow for the development of more units that can later be sold.

- ***Construction Costs*** - Most projects have a set budget based upon the estimated market value of the developed properties and/or the future use of the site. Construction costs are considered for each aspect of the site layout plan, including the cost of installing utilities, repairing existing utilities, building and roadway construction and the BMPs for use during construction and post construction.

- ***Operations and Maintenance Costs*** – Maintenance of pavement, landscape areas, and stormwater facilities should all be considered.

- ***Marketability*** – A project must meet the demands of the consumer to be sold or rented. Marketability of a particular project plays a role in project objectives and site plan layouts as certain design features may attract more buyers and bring higher prices than others.

8.2.2.4 Selecting BMPs for Combined Sewer Areas

The City of Omaha is served by both a separate storm sewer system and a combined sewer system. The combined sewer service area encompasses approximately 43 sq. miles along the eastern edge of the City of Omaha (City of Omaha LTCP, 2009, page 2-1). In areas served by combined sewer systems, the first flush of runoff enters into the combined sewer system, mixes with wastewater, and is treated by the downstream wastewater treatment plant. If runoff and wastewater flows exceed the capacity of the combined sewer system, overflows occur which release untreated runoff and wastewater into the environment.

In compliance with federal requirements, the City has developed a plan to control overflows from its combined sewer system. This plan is known as the City of Omaha Long Term Control Plan for the Omaha Combined Sewer Overflow Control Program (LTCP). As outlined in the City's LTCP, several controls were identified to reduce the discharge of untreated water from the combined sewer system. In general, CSO control alternatives included: sewer separation, storage, and high-rate treatment (HRT). Other technologies, such as structural BMPs (referred to as Green Solutions in the LTCP), were not considered as primary control alternatives but were considered for inclusion in the LTCP after further investigation.

For projects that discharge into a combined sewer, the design of post-construction BMPs must include capture and control of the WQCV plus any storage necessary to control runoff such that there is no net increase in peak runoff from pre-development conditions as they existed in October 2002 for the 2-, 10-, and 100-year storm events (City of Omaha, 2010). Structural BMPs in a combined sewer area are most effective when they are designed to reduce runoff volume, attenuate peak flows, and divert runoff out of combined sewers if possible. The objective of reducing runoff volume is to reduce flows causing combined sewer overflows, and to treat stormwater by directing it away from the combined sewer.

In 2011, the ISBMPD published results from a study which looked at the ability for specific BMP types to reduce runoff volume. Their recommendations for selecting BMPs that provide volume reduction include (Geosyntec Consultants and Wright Water Engineers, Inc., 2011, pages 30-31):

1. Normally-dry vegetated BMPs (filter strips, vegetated swales, bioretention, and grass lined detention basins) appear to have substantial potential for volume reduction on a long-term basis. Therefore, these BMPs can be an important part of an overall strategy to manage site hydrology and control pollutant loading via volume reduction.

2. Normally-dry vegetated BMPs also tend to provide better volume reduction for smaller storms, which tend to occur more frequently than larger storms; this can lead to reduced frequency of discharges or much smaller discharge volumes.

3. Retention wet ponds and constructed wetlands do not appear to provide substantial volume reduction on average and should not be selected to achieve volume reduction objectives.

4. Variability in volumetric performance between studies indicates that design attributes and site conditions likely play key roles in performance. Therefore, when using categorical analysis results to select BMPs to maximize volume reduction, it is important to also ensure that design

features to promote volume reduction are explicitly included in design and the site characteristics are conducive to allow volume reduction. For example, where facilities will likely be lined to prevent infiltration or soils are poor, volume reduction would likely be lower on average than observed in the ISBMPD studies. Conversely, for sites with soils conducive to infiltration and design characteristics provided to promote infiltration (e.g., storage volume below the lowest outlet, etc.), volume reduction would likely be higher on average than observed in the ISBMPD studies.

8.2.2.5 Selecting BMPs for Special Situations

Each development project provides unique challenges when it comes to proper BMP selection and design; however, three special situations have been identified where BMP selection and design may be more challenging than typical development projects. These special situations are:

- Linear Projects – Utilities and Transportation
- Underground Detention
- Redevelopment and Retrofit

Linear Projects – Utilities and Transportation

Utility and transportation projects are linear projects which can prove to be especially challenging when selecting and sizing post-construction BMPs. The reason why these projects are often challenging is because the project site can extend for long distances with relatively little width available for structural BMP implementation. Figure 8-1 provides an example of a linear BMP.

Figure 8-1 Structural BMP Application for Linear Roadway Project

Regional type BMPs such as extended dry detention basins, retention wet ponds and constructed wetlands can accommodate runoff from a large drainage area. Land within the right of way for a particular project may be able to accommodate smaller surface BMPs such as swales and filter strips and bioretention cells. Vegetated BMPs, particularly those with taller vegetation types can also serve to impede access to potentially dangerous sites (North Carolina Department of Transportation [DOT] Stormwater BMP Toolbox, 2008, page 2-9). When selecting and sizing BMPs for linear construction projects, consider the existing and proposed setback requirement for utilities. Underground BMPs such as subsurface manufactured systems and underground detention may be feasible if above ground space is limited. Permeable pavement should be considered for alleys, shoulders, and parking areas.

The following resources provide additional design considerations for linear construction projects and are available on-line:

North Carolina DOT Stormwater Best Management Practices Toolbox, 2008.

California Department of Transportation (Caltrans) Storm Water Website, 2011.

Underground Detention

When available open space is limited, underground detention may be used to capture and control the WQCV and provide peak flow control for larger storm events. Sediment removal and other maintenance activities may be more difficult in underground facilities. Considerations for implementation of underground detention should include:

1. ***Depth to groundwater*** – Underground detention can be located within the groundwater table if the unit is designed to prohibit groundwater from entering the storage volume area and from floating when empty. Ideally, underground detention should be located above the groundwater table, especially if the runoff is from industrial areas with high pollutant concentrations.

2. ***Native Soil Infiltration Rate and Water Storage Capacity*** – If underground detention allows for infiltration of the water into the subsurface, then the infiltration rate and water storage capacity of the native soil should be considered to determine the amount of infiltration expected to occur. If the infiltration rate is high, the storage area may be reduced to save on construction costs as long as the entire WQCV can be shown to be captured and controlled.

3. ***Maintenance Access and Activities*** – Because underground detention facilities are hidden from sight, it is difficult to assess when maintenance is required. When designing underground facilities, access to areas where debris and sediment are expected to collect should be provided. The access should be large enough that removal of the debris and sediment can be performed. In addition, access to outlet structures must also be provided to ensure that they are not clogged and are functioning properly. Providing a pre-treatment unit such as inlet filters or vortex capture units can reduce maintenance of the underground detention area.

Two commonly used underground detention options include oversized pipes with constricted outlets and reinforced concrete structures. Underground detention can be designed to promote infiltration into the subsurface by leaving the bottom of the facility open to the native soil. If infiltration is not possible or desired, then the facility is entirely enclosed and the WQCV is allowed to exit the storage area through the designed outlet structure. The outlet structure is commonly an orifice opening sized to drain the storage area within 24 hrs. The design considerations of underground detention units is very similar to above ground extended dry detention basins as discussed in Section 8.6.3. Figure 8-2 is a schematic of an underground detention reinforced concrete structure.

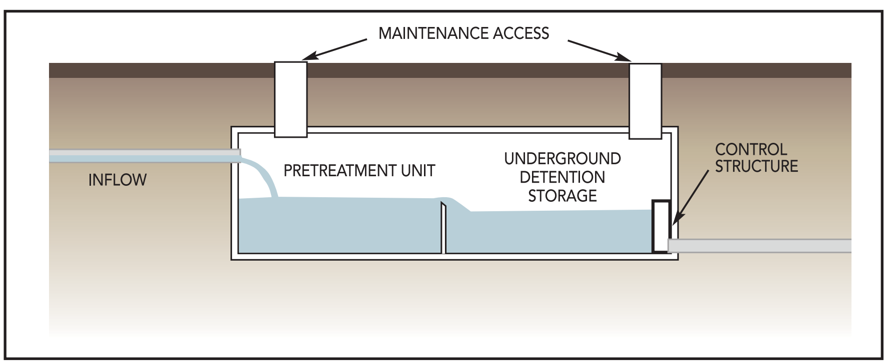


Figure 8-2 Schematic of Reinforced Concrete Structure used for Underground Detention

Redevelopment and Retrofit

Selecting BMPs for redevelopment projects may be challenging because typically open space is limited in these situations. Consider retrofitting existing stormwater features to provide water quality benefit. For example, during the initial site development swales may have been constructed to collect and transport water to area inlets. Depending upon the WQCV and the size of the existing swale, the swale may be redesigned to provide extended detention and enhanced infiltration. This can be accomplished by replacing the area inlet with an outlet structure to control the WQCV and release it over 24 hrs. Planting the swale with deep rooted vegetation will aid in reducing runoff volumes and filtering pollutants.

Existing landscaped areas can also be redesigned to provide water quality benefits. These areas may function well using soil conditioning or as a bioretention system or rain garden. This applies especially if these areas are positioned to easily collect runoff from the location where redevelopment is anticipated.

8.2.3 Guidelines for BMPs in Series (Treatment Train)

The preferred approach for water quality improvement is a combination of stormwater BMPs in series called a "treatment train." A treatment train can increase pollutant removal efficiency by providing additional treatment and volume reduction. Selection of treatment train components should be based on a combination of site characteristics, development needs, runoff sources, financial resources, and BMP characteristics (such as space requirements, design capacities, and construction and maintenance costs). (Mid America Regional Council [MARC], 2009)

A treatment train is two or more BMPs in series that capture, filter, then infiltrate or store and treat stormwater. The combination of processes provides cumulative water quality benefits. The BMPs chosen for a treatment train should be placed in series as follows:

1. Capture at source using rain barrels or cisterns.
2. Filter overland flow in to swales, filter strips, manufactured filters.
3. Infiltrate runoff with bioretention systems or rain gardens.
4. Capture and detain runoff into retention wet ponds or extended dry detention areas.

Depending on the combination of BMPs chosen, different levels of water quality benefits can be experienced. Table 8-7 presents BMP combinations for treatment trains and the associated applicability for water quality benefits.

Table 8-7
BMP Treatment Trains

	Second BMP in Series						
First BMP in Series	Infiltration Trench	Filter Strip	Vegetated Swale	Rain Garden	Bioretention	Extended Wet Detention	Extended Dry Detention Basin
Filter Strip	H	NA	L	H	H	M	M
Vegetated Swale	H	L	NA	M	H	M	L
Bioretention[1]	NA	NA	M	NA	NA	M	M
Extended Wet Detention	NA	NA	L	NA	NA	L	M
Extended Dry Detention Basin	NA	NA	L	NA	NA	L	L
H	High						
M	Medium						
L	Low						

[1] Assumes underdrain system.

8.2.4 Low-Impact Development Guidelines

The Papillion Creek Stormwater Management Policies use the following definition of Low Impact Development (LID):

> *A land development and management approach whereby stormwater runoff is managed using design techniques that promote infiltration, filtration, storage, evaporation, and temporary detention close to its source. Management of such stormwater runoff sources may include open space, rooftops, streetscapes, parking lots, sidewalks, medians, etc.*

The goal of LID is maintaining or replicating the predevelopment hydrologic regime through the use of design techniques to create a functionally equivalent hydrologic landscape. This can be accomplished using a series of site-scale controls distributed throughout the development site as close to the location where rainfall hits the landscape as possible or by applying strategies such as virgin land preservation and connection of green spaces. According to the Low Impact Development Center the LID approach includes five basic tools:

- Conservation measures.
- Impact minimization techniques such as impervious surface reduction.
- Strategic runoff timing by slowing flow using the landscape.
- Distributed BMPs to reduce and cleanse runoff close to the source.
- Pollution prevention measures to reduce the introduction of pollutants to the environment. (See Section 8.7 Lot-Level/Homeowner Non-Structural BMPs)

The first three are discussed in greater detail in this Section.

There are numerous studies that support the hydrologic and ecological benefits of LID and many that demonstrate the economic benefits of LID over traditional stormwater management techniques. Economic benefits include both lower initial capital costs and lower future operations and maintenance costs compared to traditional development. The primary applications of LID in the context of the BMP Manual are:

- Reduction of the disturbed area footprint thus reducing the area for which BMPs must be applied.
- Disconnection of impervious surfaces to control stormwater.
- Seeking opportunities to control runoff closest to the source.
- Utilizing the natural landscape and realizing ecosystem services.

Other benefits that the development community and City should consider regarding LID options are the benefits associated with programs like Leadership in Energy and Environmental Design (LEED), Leadership in Energy and Environmental Design Neighborhood Design (LEED ND), Sustainable Sites Initiative (SSI) and the National Flood Insurance Program's (NFIP) Community Rating System (CRS) to name a few.

8.2.4.1 Conservation Measures

Disturbing less area, either by avoidance or minimization of impacts, results in less runoff to treat and subsequently smaller BMPs. Many times these conserved areas can do double duty by also capturing part of the WQCV through disconnection of disturbed areas and treatment train opportunities as discussed in Section 8.2.3. Techniques to reduce the impervious footprint of the site by setting aside ecologically or hydrologically significant areas can be found in reference materials such as *Better Site Design: A Handbook for Changing the Development Rules in Your Community* (1998) produced by the Center for Watershed Protection and Conservation Design for Subdivisions by Randall Arendt. A few tools are discussed here that have been most commonly used and accepted throughout the country including, cluster and conservation subdivision designs, stream setbacks, tree protection, and protection of other sensitive sites. Sensitive areas and mechanisms for protection should all be documented per the city's Post Construction Stormwater Management Planning Guidance (2011) as presented in Section 8.4 of this manual.

Cluster and Conservation Subdivision Design

Both cluster and conservation subdivision designs identify and protect significant natural amenities. The conservation subdivision takes the protection of the resource one step further with a focused intent on also integrating the resource and providing access to the resource. The City of Omaha has a cluster design option in the Municipal Code, Chapter 53 – Subdivisions, Section 53-11, Cluster Subdivisions. The code provides a mechanism to work with open space options for subdivision developments. The code states:

> *Lots shall be permitted to be clustered or grouped to allow greater flexibility in design and development of subdivisions in order to: (1) produce innovative environments, (2) provide for more efficient use of land, (3) protect topographical features, (4) permit common open space, and (5) permit private pedestrian and vehicular access.*

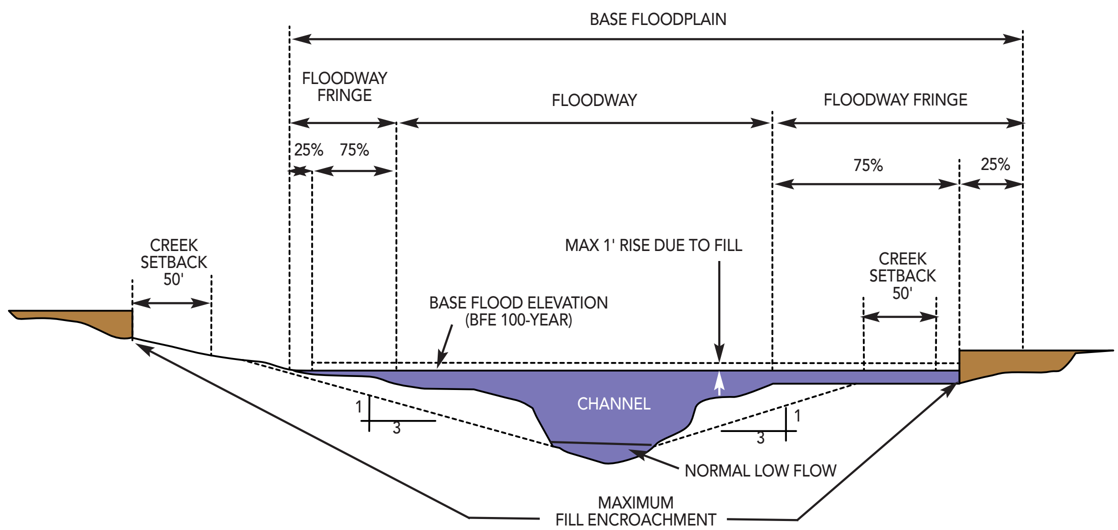
Stream Setback

The Papillion Creek Watershed Partnership (PCWP) has adopted stormwater management policies that set standards for stream (creek) setbacks. The PCWP policies are available on the web at http://www.papiopartnership.org/resources/documents/ExhibitB.pdf and in Appendix C. The policies were established with the recognition that natural areas are diminishing, and there is a need to be proactive and integrate efforts directed toward providing additional landscape and green space areas with enhanced stormwater management through restoration and conservation of stream corridors, wetlands, and other natural vegetation. The stream setback standards are intended to utilize landscape preservation, restoration, and conservation techniques to meet the multi-purpose objectives of enhanced aesthetics, quality of life, recreational and educational opportunities, pollutant reduction, and overall stormwater management. Figure 8-3 provides a schematic illustration the PCWP stream setback policy.

The PCWP stream setback policy includes:

1. Incorporating stormwater management strategies as a part of landscape preservation, restoration, and conservation efforts where technically feasible.
2. Defining natural resources for the purpose of preservation, restoration, mitigation, and/or enhancement.
3. For new development or significant redevelopment, providing a stream setback of 3:1 plus 50 ft.

along all streams as identified in the Papillion Creek Watershed Management Plan and a creek setback of 3:1 plus 20 ft. for all other watercourses.



Source: PCWP Stormwater Management Policies

Figure 8-3 PCWP Creek Setback Schematic

4. Placement of all landscape preservation features including all stormwater and LID strategies, stream setbacks, existing or mitigated wetlands, etc., identified in new or significant redevelopment into an out lot or within public right of way or otherwise approved easement.

Sensitive Sites

At a development site scale, an inventory should be conducted at the preliminary plan stage to identify any regulated areas that would require a permit such as wetlands, streams and threatened or endangered species as well as areas of opportunity either for protection or for necessary stormwater BMP consideration. Requiring documentation of these natural resources, sensitive areas or otherwise regulated areas at the preliminary planning stage would benefit both the development interest and the community in review and approval of the site plan and stormwater management plan. Here are a few considerations:

1. Identification of the soil types (and their properties) found on the project site, identified from the NRCS Soil Survey map.
2. Identification of wetland delineation in the form of a copy of National Wetland Inventory index.
3. Habitat evaluation for threatened and endangered species.
4. Location and general type of existing trees and significant vegetation and trees proposed for preservation and removal if estimated to be greater than 10-in. caliper, (prepared from aerial photo or survey).
5. Identification of streams including those that may be regulated by Section 401 and 404 of the Clean Water Act.

6. Latest (not more than two years old) aerial photograph of the site.

7. Existing contour information for the site.

A checklist is provided in the Post Construction Stormwater Management Planning Guidance (2011) regarding the submission of a Site Resource Plan.

Opportunities

Listed are existing and consideration for future opportunities to implement and incorporate conservation tools and techniques into the City's codes and development site designs.

1. Of the Omaha City Code, Article XI, Section 55-641, ED Environmental Resource District provides for one tool to apply stream setbacks. As stated in the purpose, this overlay district enable the adoption of special performance standards in combination with the site development regulations of a give base district for areas of special natural environmental significance or sensitivity. These areas include hill environments; native prairies; areas with unique soil or drainage conditions; lake, river or creek districts; forest; or other areas with unique environmental characteristics. The special purpose overlay districts also provide opportunity to include LID practice.

2. Consider managing for the range of events related to stream channel formation and integrity including the currently accounted for 2-year event but also the 10-year event.

3. Guide runoff toward stream setback area from impervious surfaces to reduce runoff volume to be captured by structural BMPs, as long as the concentration of flow is handled so as not to disturb the integrity of the buffer (e.g., level spreader).

8.2.4.2 Impact Minimization

Once the development footprint is identified there are a number of site layout considerations that can reduce impervious area and subsequently site runoff. The following are a few commonly considered opportunities to reduce site runoff at the source.

Soil Preservation

At preliminary plan stage it is important to have the soils map of the site and consideration of areas to be protected from development and construction activities. Protecting soils from disturbance and compaction retains the infiltration and storage capacity of the native site. The preliminary plan must include a preservation plan that delineates the areas to be protected and construction boundary. These areas can be used for disconnecting impervious surfaces such as rooftop gutters and/or installing stormwater facilities if proper space is available and the design criteria is met. If the soil is to be removed from the building footprint and preferably used on site, proper stockpiling should be used, so as to retain a living soil (e.g., mycorrhiza) rather than sterilizing the soil through traditional stockpiling. One such guide for proper protection, removal, storage and restoration is the Building Soil Guidelines and Resources for Implementing Soil Quality and Depth BMP T5.13 Stormwater Management Manual for Western Washington (2010).

Streets

Streets are a significant contribution to a communities overall impervious area. In many instances they are the primary conveyance system of overland runoff. Because of the limitations of working within the right-of-way, stormwater BMPs are not typically considered in treating road runoff. However, the City's *Green Streets of Omaha* sets forth a plan to improve the city's street network including opportunities to address stormwater as noted in the Introduction (City of Omaha, 2007).

> *The Omaha metropolitan region is embarking on a new stormwater management program of addressing combined sewer overflow, developing a system of regional reservoirs to manage runoff, and implementing best management practices with the development of subdivisions and major projects. Omaha's street system, a primary cause of high velocity, high volume runoff, should do its part. Tree canopies and landscaped areas can increase the permeability of street right-of-ways and delay precipitation from hitting the ground.*

Specific standards for stormwater management within linear projects are not provided in the plan. However, there is recognition of the importance of street trees and necessary rooting space for them to thrive. Specifics are found in *Green Streets of Omaha, Chapter V Installation and Maintenance Standards*. Also see Section 8.2.2.5 on Linear Projects for additional stormwater management considerations and opportunities.

In an urban setting, bump outs for traffic calming may serve as bioretention in a street or sidewalk retrofit project. Many examples are available for consideration particularly from Seattle's Green Streets Program. More information on Seattle's application of green infrastructure can be found in the USEPA's Municipal Handbook, Managing Wet Weather with Green Infrastructure: Green Streets (2008).

Consider pervious pavement or pavers in alleys for a downtown setting where there is limited space to address water quality and quantity issues. A highly recognized application of this is Chicago's Green Alleys Program. More information on Chicago's application of green infrastructure can be found in the USEPA's Municipal Handbook, *Managing Wet Weather with Green Infrastructure: Green Streets* (2008).

Curb or no curb may be the question for low-density zoned areas of the community and likewise may be preferred in a conservation subdivision context. The decision may depend on the city's ROW management policy for ditches/swales and driveway pipe maintenance and replacement programs for the proposed land use.

Parking

There are numerous accounts across the country with mall parking lots that are never full and even have ample spaces to park during the gift giving season. Most parking is driven by Institute of Transportation Engineers (ITE) standards accounting for parking needs for various land uses. Subsequently, the banking community seeks to ensure these standards are met or exceeded when loaning money to have assurance that businesses accommodate the number of customers and thus are successful at repaying those loans. With these two hurdles in mind with respect to reducing impervious surface, there are options that have been employed by other communities to address excessive parking provisions for multiple benefits including stormwater, micro-climate and air pollution reduction while accommodating safety needs of the community.

Under Article XIV- Off-Street Parking and Loading Regulations, Section 55-732 (k), General Off-Street Parking Regulations indicates no existing facility used for off-street parking on the effective date of this Chapter (March 4, 1987) shall be reduced in capacity to less than the minimum required number of spaces, or altered in design to less than the minimum standards prescribed by this Section.

Shared parking arrangements can significantly reduce impervious area. The use must be in close proximity of one another and depending on the use not have competing hrs. of operation. This may be an opportunity in the areas of transition from the urban and suburban interface. There is a provision for fewer spaces for mixed-use developments in Omaha under *Article XIV, Off-street Parking and Loading Regulations, Section 55-736 – Adjustment for Mixed Use Developments*.

Omaha's parking standards for number of spaces, stall width and depth and drive isles widths based on angle of parking are typical to those around the country. The City of Omaha does provide for compact car options as well as bonuses for bicycle parking and public transportation access as described in Omaha's municipal code Article XIV, Off-street Parking and Loading Regulations, Sections 55-737 and 55-739, respectively.

Opportunities

Listed below are existing and consideration for future opportunities to implement and incorporate impact minimization tools and techniques into the city's codes and development site designs.

1. Current street standards in Omaha reduce impervious surfaces and improve neighborhood connectivity with Chapter 53, Subdivision, Section 53-8 2(b). This Section states that cul-de-sacs shall be prohibited, except where topography or other conditions warrant their use.
2. Consideration should be given to Context Sensitive Street designs. Once such opportunity would be to consider a local residential street width of 22 ft. back of curb to back of curb with parking on one side rather than 25 ft. as stated in Section 53.8 of the municipal code.
3. The special purpose overlay districts also provide opportunity to include LID practices and conservation measures.

8.2.4.3 Landscaping

The impacts of impervious areas can be softened by landscaping. Landscaping can serve to mitigate the heat island effect, noise pollution and stormwater runoff of impervious surfaces of the built environment. There is a win/win opportunity in utilizing landscape requirements to serve multiple functions without diminishing its intended purpose for any one function.

Streetscapes

Street standards and specifications must be reviewed with the use of infiltration practices back of curb whether false curb inlets entering bioretention cells or similar surface drainage to tree planters in more urban settings. Low Impact Development Center and Green Streets are good references for consideration of these stormwater BMPs (http://www.lowimpactdevelopment.org/greenstreets/background.htm). Use of structural soils and systems designed to optimize tree rooting and health (e.g., Silva Cell) have led to new discoveries of durability within the pedestrian and amenity zone of downtowns and opportunity for healthier trees allowing for more rooting space. Optimal rooting zone for a street tree is around 1,000 cu. ft. and no deeper than 3 ft. since that is where most of the roots are located that anchor and sustain the tree. Most traditional tree pits are much less than optimal and the result is stunted and diseased trees serving as a detractor and continued replacement costs. Utilities are typically the constraint in these urban settings but if planned for, whether a new design or retrofit, can be accommodated for at a different depth of connection or location. Other benefits and considerations are documented in USEPA's *Municipal Handbook, Managing Wet Weather with Green Infrastructure: Green Streets* (2008).

The City's Green Streets of Omaha sets forth a plan to improve the city's street network including opportunities to address stormwater.

Specific standards for stormwater management within linear projects are not provided in the plan. However, there is recognition of the importance of street trees and necessary rooting space for them to thrive. Specifics are found in *Green Streets of Omaha, Chapter V Installation and Maintenance Standards*.

Parking Landscaping

Parking lot landscaping is called to perform a multitude of duties including screening, improving pavement longevity, site aesthetics and now stormwater management. A lot to ask for in limited space but it has been achieved in communities around the country. Article XIII – Landscaping and Screening of Section 55-711 defines these multiple purposes:

> *The landscaping and screening provisions are included to improve the physical appearance of the community; to improve the environmental performance of new development by contributing to the abatement of heat, glare and noise, and by promoting natural percolation of storm water and improvement of air quality; to buffer incompatible land uses from one another; and to conserve the value of property and neighborhoods within the city.*

Section 55-740(f) Landscape Requirements states the following;

> *Interior and perimeter landscaping shall be provided for all parking facilities, other than parking structures, to buffer the facility from surrounding properties and rights-of-way; reduce the environmental effects of large, hard-surfaced area; and improve the retention and absorption of storm water runoff.*

Opportunities

Listed are existing and consideration for future opportunities to implement and incorporate landscape tools and techniques into the city's codes and development site designs.

- Optimize street tree and parking lot tree standards to perform stormwater treatment functions. Omaha Section 55-928 – Green Parking Areas.

8.3 BMP Hydrology

For BMPs to function properly, the design must be based on proper hydrology which includes:

1. Capturing the appropriate amount of water (i.e. the WQCV [Section 8.3.1] or water quality discharge [Section 8.3.2]) to meet water quality objectives. For example, if a structural BMP is undersized, polluted runoff will bypass the BMP reducing the chances of meeting water quality objectives.

2. Providing the appropriate long-term hydrology for BMPs to function as designed as discussed in Section 8.3.3. For example, a wet pond that does not sustain a permanent pool volume increases maintenance activities, reduces the pollutant removal benefit from what was anticipated with the construction of a wet pond, and can create an aesthetically offensive nuisance.

3. Accounting for reductions in stormwater runoff volume when impervious areas are conveyed through pervious areas (through swales, strips, turf areas, etc.) through the concept of cascading planes as discussed in Section 8.3.4.

8.3.1 Water Quality Control Volume (WQCV)

The WQCV is the first one-half in. (0.5 in.) of stormwater runoff multiplied by the disturbed drainage area. Disturbed area is defined as the area which is subject to mechanical operations during the land development process. Undisturbed land within the drainage area may be excluded from the WQCV calculation. If the undisturbed land is temporarily fenced off preventing heavy equipment and vehicles from compacting the area or soil conditioning, as described in Section 8.6.9 is applied to a disturbed area, then the area may be subtracted from the disturbed drainage area when calculating the WQCV.

The design volume for stormwater BMPs that are sized using storage volume is equal to the WQCV except if the BMP is downstream of cascading planes as defined in Section 8.3.4. To determine the design volume for BMPs downstream of cascading planes refer to Section 8.3.4. Additional BMP volume may be necessary to meet “No Adverse Impact” requirements as described in Section 8.2.1.2 if it is used to control runoff from the 2-, 10-, or 100-year event.

8.3.2 Water Quality Discharge

For stormwater BMPs that are sized based on a flow rate (i.e. swales, filter strips, manufactured systems, etc.) the water quality discharge (Q_{WQ}) is used. The Q_{WQ} is equivalent to capturing and controlling the first one-half in. (0.5-in.) of stormwater runoff and is equal to 1.5 cfs per ac. for sites with a time of concentration equal to or less than 10 minutes. A discussion of how the Q_{WQ} was derived is provided in Appendix D. Below is an example of how the site area is used to calculate Q_{WQ}.

Example Calculation of Q_{WQ}.

Calculate the peak flow rate to use for design of a grass swale BMP that will capture and control runoff from a 0.1 ac. drainage area. The time of concentration for the site is less than 10 minutes.

The peak flow rate for the design of the grass swale BMP is equal to Q_{WQ} of 1.5 cfs per ac. multiplied by the number of ac. draining to the swale.

$$Q_{WQ} = 1.5\frac{cfs}{ac} * \ 0.1ac = 0.15\ cfs$$

The swale should be designed to convey a flow rate of 0.15 cfs.

8.3.3 Determining the BMP Water Budget for Vegetated Systems

Several structural BMPs, including constructed wetlands rely on the establishment of vegetation to increase pollutant removal and runoff volume reduction through evapotranspiration and infiltration. A water budget should be performed for these systems to increase the likelihood that plants will survive under local climatic conditions based on the parameters used in the design. The water budget for vegetated systems may change from the first year after construction when plants are not fully established to a more mature system with mature vegetation. This may mean that some facilities will require supplemental water to keep plants from dying in the first three years until the root systems have had a chance to mature. A simple water budget during the design phase can save money in the long run.

Using hydrologic modeling software and long-term precipitation records, water budgets can be estimated over a period that extends many years or for a short time period. Depending upon the software and input data available, long-term simulations may not be feasible or cost effective. At a minimum, water budgets should be performed using average annual estimates of hydrologic inputs and outputs. The designer may also choose to estimate water budgets that represent a wetter or drier than average year to gain additional insight into how the facility will operate under yearly variations in climate.

A typical water budget includes looking at all hydrologic inputs and outputs to a system. Hydrologic inputs include:

- ***Direct precipitation*** – Hourly precipitation data is available for the Omaha Region through the National Climatic Data Center at the Omaha Eppley Airfield Station. To perform an average annual water budget, select a typical year from the rainfall record with a yearly total near the average rainfall total for the entire record. Select a typical year that does not include events larger than the 5-year return interval event.
- ***Runoff from tributary areas*** – Runoff from tributary areas can be estimated using soil moisture accounting principles or other methodologies for estimating runoff using soil properties and impervious area estimates. Several computer modeling programs are equipped to perform continuous simulations including Hydrologic Engineering Centers – Hydrologic Modeling System (HEC-HMS) and USEPA's SWMM.

Hydrologic outputs include:

- ***Evapotranspiration*** – Evapotranspiration is the water lost to the atmosphere through evaporation from the soil and transpiration through plant leaves. Table 8-8 provides monthly pan evaporation totals published for the Omaha Region by the National Weather Service (NWS) (Farnsworth and Thompson, 1982). Pan evaporation data is the best indication of open water evaporation losses when multiplied by a pan factor (Chow, et al., 1988, page 88). The pan factor is typically around 0.7 and varies by season (Chow, et al., 1988, page 88). Table 8-9 shows NWS monthly pan evaporation totals that may be used to estimate evaporation losses when performing BMP water budgets. Estimates of evapotranspiration are estimated using a crop coefficient and evaporation for a reference crop. Discussions on calculating evapotranspiration are included in most hydrology textbooks.
- ***Outflow*** – Outflow from stormwater BMPs is controlled by an outlet structure. The outlet structure is designed to provide control of the WQCV or Q_{WQ}. Typical outlet structures for structural BMPs are discussed in Section 8.6. A stage-storage-discharge-relationship can be used to estimate the outflow expected from the BMP.

- ***Infiltration*** – For BMPs with high infiltration capacities, the water lost due to infiltration can be significant. Soil moisture accounting routines can be used to estimate infiltration losses based on the saturated hydraulic conductivity and porosity of the soil.

When performing an annual water budget for bioretention or rain garden BMPs, the focus should be on simulating the soil moisture to determine if adequate moisture will be available to sustain vegetation growth. For constructed wetland BMPs, the focus of the annual water budget should be to determine if the hydrology exists for wetland conditions.

Table 8-8
Monthly Means of Estimated Pan Evaporation
for the Omaha Region (Station: Omaha WSFO), in inches

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1.06	1.43	3.04	5.26	7.04	8.21	8.63	7.26	4.68	3.82	2.01	1.27	53.70

Source: Farnsworth, Richard K. and Thompson, Edwin S. 1982. National Oceanic and Atmospheric Administration (NOAA) Technical Report NWS 34 **Mean Monthly, Seasonal, and Annual Pan Evaporation for the U.S** Office of Hydrology National Weather Service Washington, D.C.

The U.S. Army Corps of Engineers (USACE) guidance on wetland hydrology suggests that a site maintain inundation or saturation of the area for at least 5 to 12 percent of the growing season, the critical consecutive day period, to maintain wetland hydrology (USACE, 1998). Additionally, this period must occur for at least 5 out of 10 years or 50 percent of the years.

The Natural Resources Conservation Service (NRCS) publishes climate summaries and the normal range for the growing season for the Omaha Region, also known as NRCS WETS data shown in Table 8-9. The normal range of the growing season is from 221 days to 164 days for the Omaha Region.

Table 8-9
Range of Growing Season for the Omaha Region

Temperature	24° F or higher	28° F or higher	32° F or higher
Probability	Growing Season Length: Beginning and Ending Dates		
	4/ 2 to 10/30	4/12 to 10/19	4/25 to 10/6
50 percent	212 days	190 days	164 days
	3/28 to 11/ 4	4/ 7 to 10/23	4/21 to 10/10
70 percent	221 days	199 days	172 days

* Percent chance of the growing season occurring between the Beginning and Ending dates.
Source: NRCS WETS Station: Omaha Eppley Field, NE6255. Creation Date: 10/21/2005

The minimum reported growing season for the Omaha Eppley Airfield station in Table 8-10 is approximately 164 days and occurs from April 25 to October 6. Saturation or inundation of the wetland for 5 to 12 percent of the growing season is recommended for wetland habitat establishment. With this growing season, the required period of inundation or saturation is between 8 and 19 days.

8.3.4 Design Volume for BMPs Downstream of Cascading Planes

Under the post-construction management guidelines, BMPs are required to capture and control the WQCV and to meet the pre-development peak flow rate for the 2-year event. When stormwater that is generated as runoff from impervious areas is conveyed through pervious areas (through swales, strips, turf areas, etc.) the runoff volume is reduced (Guo, et al., 2010). When pervious areas receive runoff from impervious areas the concept is known as cascading planes (Guo, et al., 2010). Figure 8-4 provides an illustration of the cascading planes concept. This Section provides guidance on determining the design volume for BMPs located downstream of cascading planes.

Effective impervious (I_E) can be used to represent the runoff volume reduction due to cascading planes. The Urban Drainage and Flood Control District of Denver, Colorado (UDFCD) uses the concept of effective impervious to account for runoff volumes that are reduced by using LID conveyance BMPs such as bioswales, vegetated buffers, disconnection of roof drains and other impervious areas draining to pervious areas (UDFCD, 2010). The effective impervious area concept is described in the *Incentive Index Developed to Evaluate Storm-Water Low-Impact Designs* (Guo, et al, 2010).

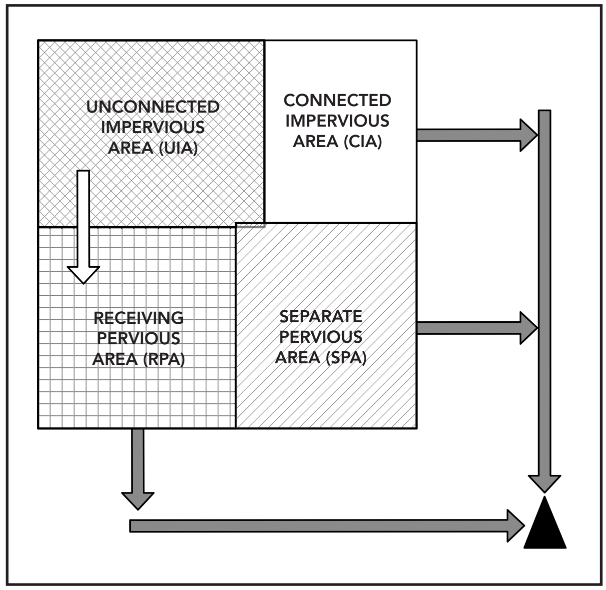


Figure 8-4 Schematic of Cascading Planes Concept. Source: *Incentive Index Developed to Evaluate Storm-Water Low-Impact Designs*. (Guo, et.al., ASCE Journal of Environmental Engineering, December 2010.)

The effective impervious area concept allows for a reduction in the design volume of the downstream structural BMP with the idea that a portion of the WQCV is "captured and controlled" within the conveyance BMP or pervious area. The portion of the WQCV captured and controlled within the receiving pervious areas can be calculated using Table 8-10.

Table 8-10 shows the depth of runoff controlled by the receiving pervious area for varying percent imperviousness of cascading planes and soil infiltration rates. An example of how Table 8-10 is used to determine the design volume for BMPs downstream of cascading plans is provided below. Additional background information on the values in Table 8-10 is provided in Appendix E.

Example Application of Cascading Planes

Consider a 10-ac. site with future impervious area of 70-percent. The ordinance requires the capture and treatment of the first 0.5 in. of runoff equating to a WQCV of 5 ac.-in. or 0.417 ac.-ft.

Using a cascading planes concept, 6 of the 7 ac. of impervious area is directed to the storm drain. The remaining 1 ac. of impervious area flows to one ac. of turf lawn on sandy-clay-loam soil with infiltration rates of 0.34 in. per hr.. The volume runoff from the 1 ac. of impervious area which flows to the pervious areas is reduced. First the percent imperviousness of the cascading planes is calculated.

$$I_A = \frac{UIA}{(UIA + RPA)} = \frac{1\ acre}{(1\ acre + 1\ acre)} = \ 0.50\ or\ 50\%$$

Where:

I_A	=	percent impervious of cascading planes
UIA	=	unconcentrated imperious area, ac
RPA	=	receiving pervious area, ac

Then, using Table 8-10, the depth of runoff controlled by cascading planes for I_A = 50 percent and f = 0.34 in.

Table 8-10
Depth of Runoff Controlled (in inches) by Cascading Planes

Percent Impervious of Cascading Planes (I_A), %	Infiltration Rate of RPA (f), in/hr[1]								
	0.12	0.16	0.26	0.34	0.43	0.83	1.04	1.92	5.85
	Soil Texture Classification								
	Clay	Sandy Clay	Clay Loam	Sandy Clay Loam	Loam	Silt Loam	Sandy Loam	Loamy Sand	Sand
1	0.049	0.064	0.100	0.127	0.154	0.255	0.295	0.404	0.497
10	0.045	0.059	0.092	0.116	0.142	0.238	0.278	0.388	0.495
20	0.040	0.053	0.082	0.105	0.129	0.219	0.257	0.368	0.491
30	0.035	0.046	0.073	0.093	0.115	0.198	0.234	0.344	0.486
40	0.030	0.040	0.063	0.081	0.100	0.175	0.209	0.316	0.476
50	0.025	0.033	0.053	0.068	0.085	0.151	0.181	0.282	0.460
60	0.025	0.027	0.043	0.056	0.069	0.125	0.151	0.243	0.434
70	0.015	0.020	0.033	0.042	0.053	0.097	0.118	0.196	0.391
80	0.010	0.014	0.022	0.029	0.036	0.067	0.082	0.142	0.319
90	0.005	0.007	0.011	0.015	0.018	0.035	0.043	0.077	0.199
100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

[1]Values for conveyance-based BMPs from UDFCD Urban Storm Drainage Criteria Manual (2010) Volume 3, page 3-17

per hr. is 0.068 in.

The design WQCV is reduced to 0.432 in. (0.5 in. – 0.068 in.) for the 2 ac. of cascading planes and the remainder of the site does not qualify for a reduction in WQCV. If a structural BMP is placed downstream of the cascading planes, then it will be sized using the 0.432 in. of runoff. If a structural BMP is placed at the downstream end of the entire site, the WQCV allowance applies only to the cascading planes portion. Therefore, the design volume for the BMP would be the area weighted total calculated as:

$$WQCV_{site} = 0.432\ inches * \frac{1\ ft}{12\ inches} * 2\ acres + 0.5\ inches * \frac{1\ ft}{12\ inches} * 8\ acres = 0.405\ acre - feet$$

For preliminary conceptual design, Table 8-10 is used to estimate the runoff depth controlled by cascading planes based on soil texture classification for the receiving pervious area (RPA). For final design, Table 8-10 is used with the measured infiltration rate of the RPA. Measurements of infiltration rate shall be recorded at a minimum of two locations within the RPA and an additional location for every 10,000 sq. ft. of RPA. The infiltration rate shall be measured using a double-ring infiltrometer using requirements outlined in ASTM D3385 or equivalent method approved by the City of Omaha.

8.4 Post-Construction Stormwater Management Plans

A Post-Construction Stormwater Management Plan (PCSMP) is required for submittal and approval by the City of Omaha Public Works Department. The paragraphs below discuss the requirements for submittal of a PCSMP, information to be included on the PCSMP application, and elements to consider during plan development.

The process for submitting a PCSMP begins with a pre-application meeting with City of Omaha Public Works Department to discuss a concept plan. After the pre-application meeting, the owner shall apply for a PCSMP application through the City of Omaha's Permix website accessible at www.omahastormwater.org. The City will assign a number to the PCSMP and the PCSMP number should be applied to the plat submittal and the final version of the PCSMP application should be submitted.

8.4.1 PCSMP Submittal Requirements

A PCSMP will be required with the submittal of (1) storm sewer construction plans for subdivisions that have an approved preliminary plat, (2) a Grading Permit Application for projects that do not require a preliminary plat and disturb 1 ac. or more of the site or (3) Building Permit Application for projects that add or replace less than 1 ac. but more than 5,000 sq. ft. of impervious surface area.

PCSMPs shall be prepared by or under the supervision of a licensed professional civil engineer registered in the State of Nebraska or other professional approved by the City of Omaha Public Works Department. The responsible professional shall be listed as the Designer on the Application and will be required to provide a seal on PCSMP sheets and calculations.

A building permit may be issued while the final PCSMP is prepared. This is handled with a letter submitted to the City requesting a Hold on the Certificate of Occupancy (CO) until such time as the final PCSMP is approved.

The PCSMP shall include the following elements:

- PCSMP Application
- PCSMP Plan Sheets
- BMP Design Information
- Drainage Study
- BMP Maintenance Requirements
- Recorded Maintenance Agreement (Maintenance Agreement will be required before Final Plat or Certificate of Occupancy is approved)

Conditional approval will be issued once the PCSMP has been reviewed and if minimum design standards are met. Final approval of the PCSMP will be issued when the BMP certification and as-built drawings are provided by the owner.

For BMP Certification, the Designer shall submit the following elements to the City of Omaha Public Works Department:

- Record Drawings of the Final PCSMP Sheets.

- BMP Certification Document - The BMP Certification document can be found at http://www.omahastormwater.org/forms. This document also provides space for indicating the inspector and inspection report holder as part of the annual monitoring requirements for the BMPs.

8.4.2 PCSMP Required Information

The PCSMP provides the reviewer with critical information about the new development that demonstrates responsibility for the site and an understanding of site conditions that will affect post-construction stormwater management. Instructions are provided on the Application.

8.4.2.1 PCSMP Plan Set

At a minimum, the PCSMP plan set must include:

1. A Site Resources Plan of the development site at scale showing existing natural and aquatic resources including, but not limited to:

    a. *Existing topography (2-ft. minimum contour interval with elevations tied to NAVD 88)*

    b. *Wetlands*

    c. *Open waterways with 50 ac. of drainage or a defined bed and bank*

    d. *Ponds or lakes*

    e. *Green space corridors*

    f. *General types of vegetation on site, excluding crops (e.g. tree canopy, turf grass, native grasses or other buffer, wetlands, etc)*

    g. *Floodplain and Floodway*

    h. *Steep slopes (greater than 17%)*

    i. *Soils types and hydrologic soils groups*

    j. *Utility lines, easements, water supply wells, and sewage treatment systems*

2. A Final Drainage and Stormwater BMP Plan of the development site at scale showing:

    a. *Existing topography (2-ft. minimum contour intervals with elevations tied to NAVD 88)*

    b. *Proposed topography (2-ft. minimum contour intervals with elevations tied to NAVD 88)*

    c. *Proposed drainage basins labeled with an identifier, runoff coefficient and drainage basin area (ac.)*

    d. *Proposed land uses/zoning in each drainage basin*

    e. *Location of proposed stormwater conveyance systems such as storm sewer, storm drains, grass channels, vegetated swales, and flow paths*

f. *Proposed areas of fill placement and limits of construction*

g. *Proposed BMPs with an identifier that matches their drainage basin*

h. *Proposed utility lines, easements, water supply wells, and sewage treatment systems*

3. Final Construction Plans:

a. *Vicinity map*

b. *Existing utilities and infrastructure*

c. *Proposed stormwater BMPs including structural components*

d. *Proposed storm sewer and stormwater conveyance systems*

e. *Other proposed infrastructure as it relates to the construction of the stormwater BMPs*

f. *Construction notes*

g. *Design water surface elevations with elevations tied to NAVD 88*

h. *Structural details of outlet structures, embankments, spillways, stilling basins, grade control structures, conveyance channels, etc.*

i. *Plan and profile sheets (if applicable)*

j. *Reference to the project geotechnical report*

Depending on the size and complexity of the project, the designer may elect to combine the components of the various plans so long as all of the components are represented and clearly identified.

8.4.2.2 BMP Design Information

Refer to Section 8.6 Structural Best Management Practices for submittal requirements for each structural BMP.

8.4.2.3 BMP Maintenance Requirements

Section 32-124 of the City of Omaha Municipal Code states, *The owners and occupants of lands on which post-construction BMPs have been installed to meet the requirements of this section shall ensure the maintenance of these BMPs and shall themselves maintain those BMPs if other persons or entities who are also obliged to maintain those BMPs (by contract or covenant, or pursuant to this section) fail to do so. BMPs shall be inspected or reviewed as appropriate at least annually, and a written record of inspection results and any maintenance work shall be maintained and available for review by the City.*

Annual review and inspection of BMPs shall be done by a professional qualified in stormwater BMP function and maintenance. Information on the Inspector that will provide annual review and inspection of BMPs and the holder of the annual inspection report shall be provided on the BMP Certification Form.

To assure compliance with the municipal code, maintenance requirements for post-construction stormwater BMPs must be documented as an exhibit to the Maintenance Agreement to ensure that the system will function properly.

The following elements are required:

- Site information,
- BMP information, and
- Description and schedule of maintenance and repair tasks for each BMP type.

Refer to Section 8.6 Structural Best Management Practices for recommended maintenance activities and frequency for each BMP.

8.4.2.4 Maintenance Agreement and Easement

Section 32-124 of the City of Omaha Municipal Code states, *the applicant or owner is required to execute an inspection and maintenance agreement, to be filed on record, binding on all subsequent owners of land served by a private stormwater management facility. Such agreements shall provide for access to the facility, at reasonable times, for inspections by the City or its authorized representative to ensure that the facility is maintained in proper working condition to meet design standards*.

Such agreements shall document the dedicated easement, the responsibilities of the owner, the Home Owner's Association or other responsible party (for Sanitary Improvement Districts), and the City of Omaha. The easement shall be large enough to include the area of the BMP and allow for maintenance access. The maintenance agreement shall be approved by the Public Works Department as part of the Final PCSMP and recorded with the Register of Deeds. A sample copy of the Maintenance Agreement can be downloaded at http://www.omahastormwater.org/forms

8.4.3 PCSMP Development

In developing a PCSMP it is important to characterize and evaluate the site. Information obtained during the site assessment enables the applicant and their consultant to assess site conditions that will contribute to an effective post-construction stormwater management plan. A complete evaluation shall include consideration of limitations and advantages of each individual site. This process will enable the selection, sizing and siting of practices that address the unique circumstances of a site.

The development of the PCSMP must be initiated in the early stages of site planning and design. However, before a stormwater management plan can be developed, defining site conditions must be completed by conducting a site assessment. The data collected during the site assessment will be used for describing site conditions, including vegetation, soils and drainage patterns. When this information is obtained, appropriate stormwater BMPs can be selected, located, sized, and designed.

The following data should be collected, to the extent practical, during the development of the PCSMP:

- **Natural Resources:** The development site's natural resources, including vegetative communities, soils and geology, and aquatic resources need to be determined to assist in stormwater management plan development and is part of the permit application. Important data includes wetlands, riparian (stream) corridors, native prairie and/or woodland. Natural resources should be assessed by trained professionals.
- **Site topography:** Topography dictates how and where water will drain from a site. On steeper sites, stormwater will runoff more rapidly, with less infiltration and greater volume.

Stormwater management requirements are substantially different than for more gently rolling or flat sites.

- **Soils:** Soil information is important for development of the stormwater management plan, and for optimal planning of the new community. Soil depth, texture (sand, silt, and clay content), and structure are important factors that will provide understanding of infiltration capacity (permeability), ability to support vegetation, and erodability. Engineering qualities and limitations of the soil are important for determining where structures can be placed, how stormwater runoff can be managed, and possible limitations for underground utilities. If hydric soils are present, it is important to understand limitations of building in these areas. Much of the information can be obtained from a U.S. Department of Agriculture (USDA) County Soil Survey, but an on-site soil assessment is recommended.

- **Aquatic Resources:** The identification of streams, ponds, and lakes as receiving waters and as an integral part of the stormwater management plan is critical. Understanding the function of these water bodies, their current condition, and potential impacts from proposed development may influence your choice of stormwater BMPs. The identification of these resources may also be necessary to comply with local, State and Federal regulations.

8.5 Lot-Level BMPs

This Section focuses on BMPs which can be applied on single-family residential lots including rain gardens, rain barrels and cisterns, and the practice of disconnecting impervious surfaces. While many of the BMPs included in this Section can be implemented on other types of land uses, the guidance provided on the following pages is aimed at the residential lot owner.

8.5.1 Rain Gardens in Residential Areas

A rain garden is a depressed area of vegetation that is designed to capture, hold, and convey excess stormwater. A rain garden is designed to accept runoff from very small areas such as roof tops, driveways, or general overland flow. Direct discharge that is concentrated, such as from downspouts or curb and gutter systems, should enter the garden through an energy dissipater device, such as a filter strip. Individual gardens aid in controlling the volume of runoff from smaller drainage areas from impacting the stormwater system. Rain gardens offer the same removal rate of pollutants (copper, lead, zinc phosphorous, etc.) as swales, prorated to reflect the smaller scale (EPA, 2006). However, to provide an effective contribution to stormwater management, rain gardens must be sufficient in number and common throughout an area (MARC, 2009).

Design Considerations	
Location Characteristics	Slope: < 8 percent
Contributing drainage area	< 0.25 acres
Design size	10 to 20% drainage area
Detention time for WQCV treatment	< 48 hours; < 24 preferred
Implementation and Maintenance Considerations	
Potential for use with other BMPs	Moderate. As a downstream infiltration BMP, can be used in treatment train.
Maintenance	High initially, lower with establishment of vegetation; Routine -Sediment/debris removal, vegetation upkeep

8.5.1.1 General Application

Rain gardens can be used to improve the quality of runoff coming from roof tops, driveways, and lawns of residential neighborhoods, small commercial areas, and parking lots (MARC, 2009). An example of a rain garden installation on a residential lot is shown in Figure 8-5. They are typically most effective for catchments less than 0.25 ac. Rain gardens work well with other BMPs if they are placed downstream of filtering BMPs such as bioswales and filter strips to remove coarser sediments and maintain sheet flow into the rain garden.

Figure 8-5 Example of Rain Garden (EPA, 2006)

Rain gardens should be placed near the source of stormwater runoff, or in a low area of the property where water collects as shown on Figure 8-6.

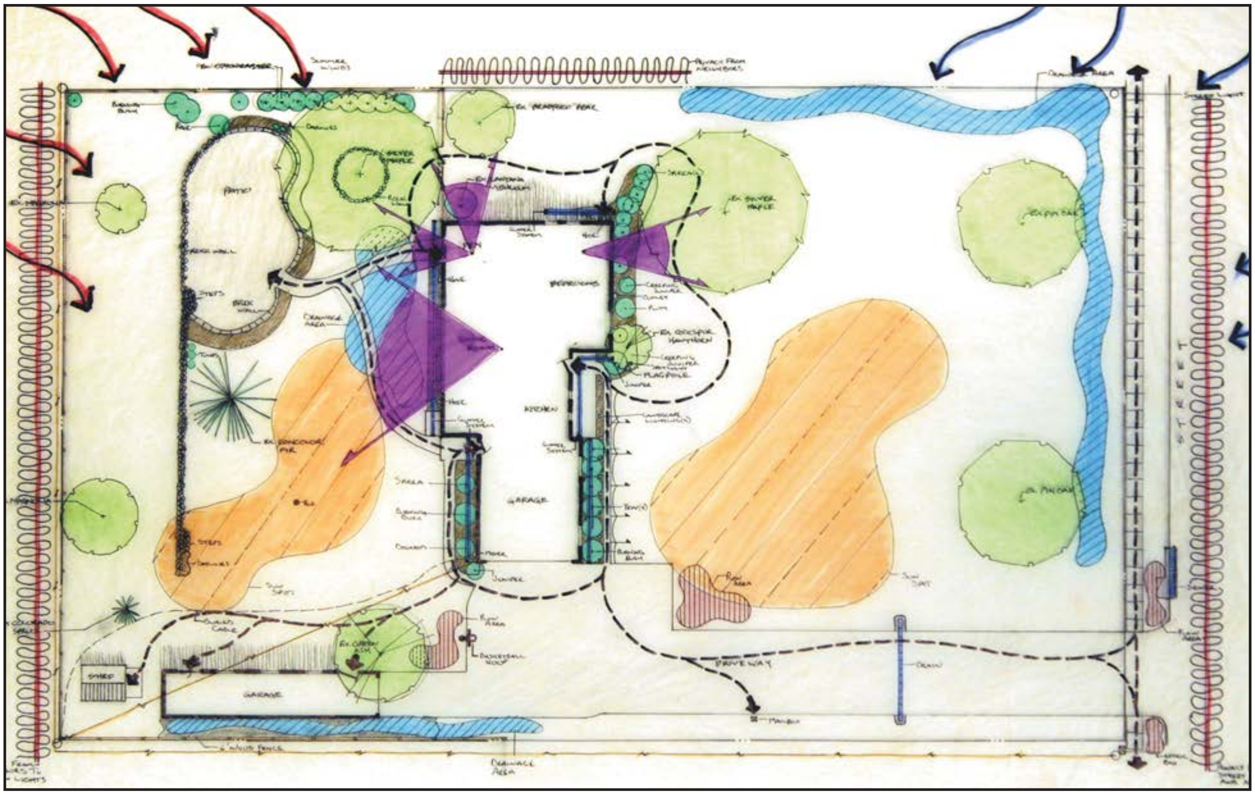


Figure 8-6 Example Placement of Rain Gardens on a Residential Lot

8.5.1.2 Design Requirements and Considerations

Rain gardens require that captured rainfall and runoff be infiltrated below the surface within 24-48 hrs. to avoid nuisance insects (MARC, 2009). Therefore, the critical design requirement is the rate at which water can infiltrate into the soil. Consider more than just the aesthetic and hydrologic benefits of a rain garden; remember that having a natural space will promote wildlife habitat and a connection with nature. Rain gardens can be planted in a variety of soils. (Rain gardens constructed in clay soils and planted with native vegetation can exhibit infiltration rates up to three times greater than those planted with turf grasses [U.S. Geological Survey (USGS), 2010].) Drainage tests can determine if the native soil needs to be amended. Regardless of amendment potential, if the underlying soil below the amendment level lacks the capability to drain appropriately, then an underdrain system or alternative location should be considered. A typical cross section of a rain garden is shown in Figure 8-7.

Site Considerations

- Rain gardens should be placed in the lowest portion of a landscaped area to allow runoff to flow to it. If locating a garden in an area that typically has ponded water (indicating poor infiltration), consider amending the existing soil as needed with compost. Till the compost to mix with the existing soil. The soil mix should be 6 in. deep in the bottom of the rain garden (Rodie et al., 2010). An organic-rich top soil will initiate plant growth and soak up excess runoff.

- The rain garden should not be placed upstream of a foundation, unless adequate design measures are taken. Avoid placing in areas where ponded water may create problems.

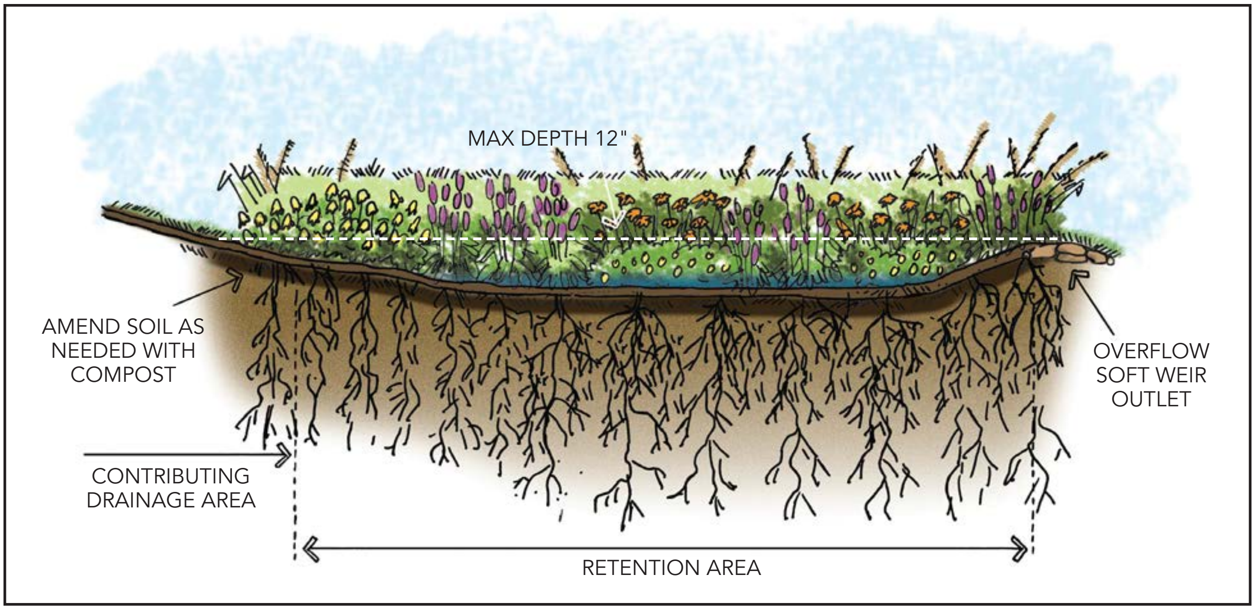


Figure 8-7 Rain Garden Cross Section

Infiltration Considerations

- The garden shall be sized to treat the amount of water flowing into it and shall have a flat bottom to ensure even infiltration into the soil across the garden.

- To determine if a soil is suitable for a rain garden, dig a hole in the ground 6 in. deep and fill it with water. Measure the depth of water after 24 hrs. The maximum ponding depth is the depth of water that will infiltrate into the soil within 48 hrs. based on the drainage test results.

Vegetation Selection Considerations

- Selected plants may include native and adapted species that are tolerant of both wet and dry cycles. Plants should be deep-rooted for drought tolerance and enhanced soil infiltration, and should be selected for appropriate aesthetic characteristics as well as environmental requirements. The most critical aesthetic consideration is typically plant height; plants which grow too tall for the scale and location of the garden appear weedy and increase maintenance, especially if they flop onto adjacent plants, sidewalks, etc. Use shorter plants in small gardens, including compact cultivars if available, and place taller plants near the middle of the garden and away from defined edges to maintain a more ordered appearance.

- Provide the soil type, anticipated maximum water depth, light conditions, and draindown time to a local nursery or landscaping specialist for planting suggestions (vegetation types, seeding rates, establishment procedures, maintenance procedures), or use the plants suggested for rain gardens by the University of Nebraska –Lincoln Extension. A basic plant list is available in Stormwater Management: Plant Selection for Rain Gardens in Nebraska (Nebguide G1759); a comprehensive list, including images, plant descriptions and selection matrices for specific garden conditions, is published in the Nebraska Bioretention and Rain Garden Plants Guide: Sustainable Selection, Placement and Management in the Northern Great Plains (EC-1261). For additional plant information and updates, reference the UNL Water Website, Property Design and Management page.

8.5.1.3 Maintenance

Maintenance activities for rain gardens include short-term and long-term maintenance tasks.

Short Term: Year 1 – Year 3

1. If possible, route stormwater away from the garden initially until vegetation becomes established, typically for a 30 to 60 day timeframe.
2. Water plants daily for the first couple of weeks depending on weather, then 2-3 times per week for the next couple of weeks. Then water as needed. Additional watering may be required biweekly during the summer months (June through August) through the first year.
3. Eliminate weeds using spot application of herbicide, or manual removal throughout the first year.
4. After significant rain (single rainfall event equaling or exceeding 0.5 in.):

    Redistribute mulch, remove trash, and inspect vegetation.

    If sediment has accumulated, remove it and replace mulch and vegetation as needed.

    Check for erosion inside and around the rain garden. Repair erosion damage if it occurs.

    Visually monitor infiltration of rain garden. If ponding lasts longer than 48 hrs., an alternative drain or modifications may be required.
5. At one year after completion, inspect vegetation. Replace dead plants and remove invasive plant species if necessary.

Long Term: Year 3 - later

1. In early spring, mow or trim vegetation to between six and eight in. above the ground. Remove accumulated debris.
2. Inspect vegetation one to two times each year and remove weeds and invasive species.
3. Trim back or remove vegetation if it becomes overgrown.

8.5.1.4 Resources

The internet provides many resources that can be referenced in the design, installation, and maintenance of a rain garden. Internet resource guidelines may differ from the City of Omaha's. In these instances, the City's criteria and guidelines govern.

10,000 Rain Gardens Initiative: www.rainkc.com/ccLib/image/pages/PDF2-66.pdf

Citizen's Guide to Protecting Wilmington's Waterways:
www.wilmingtonnc.gov/portals/0/documents/Public%20Services/Stormwater/Publications,%20Reports/cguide.pdf

Establishing Native Grasses: www.ksre.ksu.edu/library/crpsl2/MF2291.pdf

How to Build Your Own Backyard Rain Garden: www.stormwater.kytc.ky.gov

Lawns, Landscapes and Gardens. http://water.unl.edu/web/landscapes/rain-gardens

Native Vegetation Establishment: www.kansasnativeplantsociety.org, Rain Gardens: A how-to manual for homeowners: clean-water.uwex.edu/pubs/pdf/rgmanual.pdf

Rain Garden Design Brochure: http://www.marc.org/Environment/Water/raingarden.htm

Stormwater Management Rain Garden Design for Homeowners:
www.ianrpubs.unl.edu/epublic/live/g1758/build/g1758.pdf

Stormwater Management Installing Rain Gardens in Your Yard in Nebraska:
http://www.ianrpubs.unl.edu/epublic/pages/publicationD.jsp?publicationId=854

Stormwater Management Plant Selection for Rain Gardens in Nebraska.
http://www.ianrpubs.unl.edu/epublic/pages/publicationD.jsp?publicationId=852

8.5.1.5 References

USEPA. 2006. *Bioretention (Rain Gardens) BMP Fact Sheet.*
http://cfpub.epa.gov/npdes/stormwater/menuofbmps/index.cfm?action=factsheet_results&view=specific&bmp=72

Jackson Soil and Water Conservation District. http://www.jswcd.org/index.asp

MARC. 2009. Manual of Best Management Practices for Stormwater Quality - Second Edition. Section 8.1 Rain Gardens. http://kcmetro.apwa.net/chapters/kcmetro/specs/APWA_BMP_ManualAUG09.pdf.

Rodie, Steve, Hartsig, Ted and Szatko, Andy. 2010. Sustainable Landscapes - Rain Gardens, Bioswales and Xeric Gardens: A Manual for Homeowners and Small Properties in Omaha. University of Nebraska – Lincoln Extension, Water/Property Design and Management Website:
http://water.unl.edu/web/propertydesign/publications (listings by chapter)

USGS. 2010. Evaluation of Turf-Grass and Prairie-Vegetated Rain Gardens in a Clay and Sand Soil. SIR 2010-5077.2010

University of Nebraska-Lincoln Extension, Institute of Agriculture and Natural Resources. 2007. Stormwater Management, Plant Selection for Rain Gardens in Nebraska (Nebguide G1759):
http://www.ianrpubs.unl.edu/epublic/live/g1759/build/g1759.pdf

8.5.2 Rain Barrels and Cisterns for Residential Use

Rain barrels and cisterns are storage vessels used to capture rooftop runoff for reuse for landscaping and other non-potable uses. Water collected has various uses, including lawn irrigation, vegetable and flower gardening, and watering houseplants. It is critical to dewater both rain barrels and cisterns between rainfall events to accommodate future rainfall. By diverting water from storm drainage systems, rain barrels and cisterns reduce pollutants by reducing the volume of runoff entering local rivers and streams.

Design Considerations	
Location characteristics (Slope, Soil Type)	N/A
Contributing drainage area	Varies: Rooftop drainage
Design size	50-60 gallons (rain barrel) 50-5000 gallons (cistern)
Detention time for WQCV treatment	N/A
Median Effluent Concentrations	N/A
Implementation and Maintenance Considerations	
Potential for use with other BMPs	Moderate. Can be used for BMP irrigation during dry periods
Maintenance	Moderate. Keep barrel free of organic material. Mesh screens and olive oil will keep mosquitoes from breeding. Use accumulated rainfall regularly to allow adequate storage room for future rain events

8.5.2.1 Rain Barrels

A rain barrel is typically a 50-60 gallon tank to which downspouts are directed. Examples of rain barrels are shown in Figure 8-8. Roof rainwater collects in these barrels and a drainage valve and/or garden hose is used to distribute water for irrigation in between storm events. A perforated hose can also be used to distribute collected water passively.

Figure 8-8 Examples of Residential Rain Barrels

Design and Installation Requirements

- Components: 55 gallon covered plastic drum, preferably with removable lid. Two openings will need to be cut into the top for downspout discharge, and an overflow outlet. An atrium grate or downspout filter should be installed to catch debris (clean grate occasionally to prevent blockage). A spigot for controlling the release of water from the drain hole. It is recommended that the barrel have a sealed, child resistant top that can be easily removed for cleaning. Refer to the Omaha Stormwater Program for complete list of materials needed to build a rain barrel. Figure 8-9 shows the typical configuration of a rain barrel system.

- Location. Locate the barrel under downspouts where water can be easily collected for transport away from building foundations.

- Installation Guidelines. The base of the rain barrel must be level and secure. Concrete blocks or pavers can be used to achieve this. Flexible downspout adapters should be used between the top of the barrel and the end of the downspout. Overflow outlets should be routed away from foundations and to pervious areas. Refer to the Omaha Stormwater Program for complete instructions on how to build a rain barrel. Additional rain barrels will increase the quantity of water stored. Table 8-11 provides the total runoff volume generated based on a roof's sq. footage and the amount of rainfall.


Figure 8-9 Rain Barrel Diagram

Table 8-11
Total Runoff Volume Generated Based on Roof's Square Footage
Gallons of Water Produced

		Rainfall (inches)									
		0.1	0.2	0.3	0.4	0.5[1]	0.6	0.7	0.8	0.9	1
Roof Area (square feet)	100	6	12	18	24	30	36	41	47	53	59
	250	15	30	44	59	74	89	104	118	133	148
	500	30	59	89	118	148	178	207	237	266	296
	750	44	89	133	178	222	266	311	355	400	444
	1000	59	118	178	237	296	355	415	474	533	592
	1250	74	148	222	296	370	444	518	592	666	740
	1500	89	178	266	355	444	533	622	711	799	888
	1750	104	207	311	415	518	622	725	829	933	1036
	2000	118	237	355	474	592	711	829	947	1066	1184

[1] 0.5 inches equals WQCV

Where to Purchase Rain barrels

Check with local hardware store.

www.aridsolutionsinc.com/page/page/522317.htm

www.bayteccontainers.com

www.gardeners.com

www.rainbarrelsource.com

8.5.2.2 Cisterns

Cisterns are somewhat larger than rain barrels, and provide considerably more storage as well as pressurized distribution. One or more downspouts can be connected to a partially or fully buried cistern, storing water for use between rain events. Stored water is distributed by gravity (if elevated above the ground) or using an electric or hand pump (if below ground). An example of a cistern is shown in Figure 8-10.

Figure 8-10 Residential Cistern

Design and Installation Requirements

- Components: Variable size tank constructed of an impervious, water retaining material. A downspout filter should be used to collect debris. Includes electric discharge pump, secured access point, piped intake locations, and an overflow point.
- Location: Cistern can be located above or below ground. Should be located away from foundations.
- Installation Guidelines: Due to the size, complexity, and potential proximity of cisterns to foundations, a structural engineer should be consulted for design and construction.

8.5.2.3 Resources

What is a Rain Barrel?: www.epa.gov/region03/p2/what-is-rainbarrel.pdf

How to Build Your Own Rain Barrel: http://www.freerainbarrel.com/instructions.pdf

How to Install and Maintain a Rain Barrel:
http://www.smgov.net/uploadedFiles/Departments/OSE/Categories/Urban_Runoff/Install_Maintain_RainBarrel.pdf

Streams Stormwater Page and Rain Barrel Guidance:
www.lakesuperiorstreams.org/stormwater/toolkit/rainbarrels.html

Low Impact Development Sustainable School Projects:
www.lowimpactdevelopment.org/school/rainb/rbm.html

Rain Barrel Information: www.marc.org/Environment/Water/buildrainbarrel.htm

Rain Barrel Guide: www.rainbarrelguide.com

River Falls, Wisconsin: http://www.rfcity.org/eng_property.asp

Rain Barrel Information: www.townofblackmountain.org/rain_barrel.htm

Watershed Activities to Encourage Restoration (W.A.T.E.R.):
www.watershedactivities.com/projects/spring/rainbarl.html

American Rainwater Catchment Systems Association: http://www.arcsa.org/

LID Toolkit Fact Sheet: Cisterns and Rain Barrels http://www.mapc.org/resources/low-impact-dev-toolkit/cisterns-rain-barrels

Harvested Rainwater: http://rainwater.sustainablesources.com/#Components

The Online Rainwater Harvesting Community: http://www.harvesth2o.com/about.shtml

Rainwater Catchment Solutions: Clean Water Starts Before Your Tank:
http://www.smgov.net/uploadedFiles/Departments/OSE/Categories/Urban_Runoff/Clean_WaterTips.pdf

Rainwater Catchment Solutions: First-Flush Diverters:
http://www.smgov.net/uploadedFiles/Departments/OSE/Categories/Urban_Runoff/First-Flush_diverters.pdf

Rainwater Collection: http://www.nsf.org/consumer/rainwater_collection/index.asp?program=WaterTre

Manual of Rainwater Harvesting:
www.twdb.state.tx.us/publications/reports/RainwaterHarvestingManual_3rdedition.pdf

Urban Design Tools: Rain Water Cistern: www.lid-stormwater.net/raincist_construct.htm

8.5.2.4 References

City of River Falls, Wisconsin Engineering: Storm Water: http://www.rfcity.org/eng_stormwater.asp

Ersson, Ole. 2006. Rainwater Harvesting and Purification System: www.rwh.in

W.A.T.E.R. Date Unknown. Installing Your Rain Barrel:
www.watershedactivities.com/projects/spring/rainbarl.html

8.5.3 Residential Disconnection of Impervious Area

Runoff from connected impervious areas often flows directly to a stormwater collection system with no possibility for infiltration into the soil. The direct runoff from these areas is one of the greatest contributors to nonpoint source pollution and stream hydromodification. The convergence of runoff from numerous impervious drainage areas combines volumes, runoff rates, and pollutant load. By disconnecting impervious areas, runoff from rooftops, driveways, and parking lots is diverted from a stormwater management system or a curb and gutter system. Water is instead directed to a vegetated area, a bioretention area, or a holding device. Disconnecting impervious areas can potentially reduce runoff volume and filter out pollutants. Disconnection is not a standalone BMP, but is part of a BMP control strategy.

Design Considerations	
Location Characteristics	Applicable to rooftops, driveways, and parking lots
Contributing drainage area	Variable
Design size	Variable
Detention time for WQCV treatment	Not applicable
Median Effluent Concentrations	Variable
Implementation and Maintenance Considerations	
Potential for use with other BMPs	High. Can be used as the first process in a treatment train.
Maintenance	Routine -Sediment/debris removal, general up-keep

8.5.3.1 General Application

Disconnection practices can be applied in almost any area containing impervious surfaces. However, the runoff must be able to discharge to a suitable receiving area, such as a densely vegetated lawn, in order for the BMP to be effective. Figure 8-11 provides an example of a downspout that is disconnected from the stormwater system.

Figure 8-11 Example of a Downspout that is Disconnected from the Stormwater System

8.5.3.2 Design Requirements and Considerations

Disconnecting impervious areas requires little construction and few materials. Disconnection points should be directed away from buildings and the connected stormwater system, instead the runoff should be directed to grassed areas and/or other BMPs. Options include rooftop disconnection and installation of curb cuts along existing parking lots or streets.

Rooftop disconnection requires minimal modifications to downspouts to direct runoff away from collection systems and impervious areas

Curb cuts may be installed to encourage stormwater flows away from inlets

Figures 8-12 and 8-13 demonstrate typical lot diagrams for disconnecting impervious areas. Figure 8-12 is a typical lot diagram with downspouts indicated by black dots. The orange arrows show flow direction into the grassed lawn and other vegetated areas (adapted from Portland, Date Unknown). Figure 8-13 demonstrates a highly urbanized area where there are potential disconnection locations available adjacent to buildings and other impervious area.

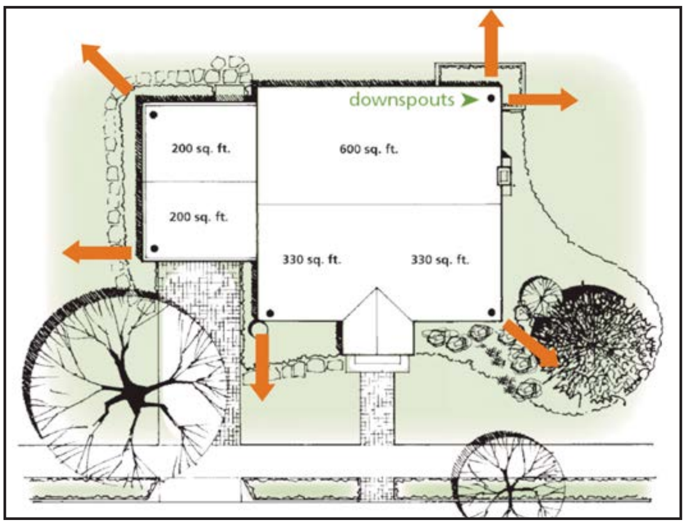


Figure 8-12 Typical Lot Diagram (Adapted From City of Portland, Date Unknown

Figure 8-13 UNMC Provides Pervious Area around Buildings for Downspout Discharge

8.5.3.3 Resources

MARC. Redirect or Disconnect Your Downspout: http://www.marc.org/Environment/Water/downspout.htm

City of Portland Environmental Services. Date Unknown. How to manage stormwater Rain Barrels: http://www.portlandonline.com/shared/cfm/image.cfm?id=182095

8.6 Structural Best Management Practices

This section describes the general application, advantages and disadvantages, design requirements and considerations, inspection and maintenance, submittal requirements, design calculations, and design examples for select structural BMPs. BMPs not included in this document may be used in new development and redevelopment projects as long as they are preapproved by the City and are designed to meet City performance standards.

8.6.1 Bioretention system

Bioretention systems use storage volume and vegetation to accept and treat stormwater runoff through infiltration into layers of plant roots and the growing medium. Bioretention systems consist of a smaller filter area surrounded by a larger area comprised of conditioned soil and vegetation. Reductions in stormwater pollutants are achieved via natural plant processes and movement through conditioned soil and filter media (e.g. growing media). Runoff volumes are also decreased by deep infiltration into the surrounding subsurface and evapotranspiration from plants.

Design Considerations	
Location characteristics (Slope, Soil Type)	Slope: < 10%[1] Soil Type: A, B, C, D
Contributing drainage area	< 4 acres[1]
Design size	1-15% drainage area
Detention time for design volume treatment	1-2 Days
Median effluent concentrations[2]	TSS = 4-6 mg/L, TP = <0.1 – 0.35 mg/L, TN = 0.6 – 2.5 mg/L, Cu = 9-16 µg/L, E coli = 58 – 90 cfu / 100 mL, Fecal coliform = 2 – 290 MPN/100 mL
Implementation and Maintenance Considerations	
Potential for use with other BMPs	Works well with upstream source controls and filter strips and swales for pretreatment
Maintenance	High initially, lower with establishment of vegetation

Note: Median effluent concentrations apply to events with measured discharge.
[1]MARC, 2009
[2] Li and Davis, 2009

8.6.1.1 General Application

Bioretention systems are used in urban areas because of the minimal land requirement and thus are usually located adjacent to highly trafficked areas. This provides opportunities for BMP public education and signage. Bioretention systems can be located in areas where they receive sheet flow from stabilized graded areas or in recessed areas that receive runoff from imperious urban infrastructure. An example of a typical application is shown in Figure 8-14. Typical applications include median strips, parking lot islands, and landscaped swales alongside roads. These areas can be designed so that runoff is either diverted directly into the bioretention system or conveyed into the bioretention system by a curb and gutter collection system (EPA, 1999; UDFCD, 2010). To maximize treatment effectiveness, the drainage area must be graded in such a way that minimizes erosive conditions as sheet flow is conveyed to the treatment area. To effectively minimize sediment loading in the treatment area, bioretention should only be used where all upstream tributary area is stabilized (EPA, 1999, UDFCD, 2010). Inundation with suspended sediment can reduce infiltration rates in the bioretention system (MARC, 2009). Manufactured filters, filter strips, forebays, and swales are commonly used as pretreatment devices to prevent sediment loads from entering bioretention systems.

Figure 8-14 Orchard Park Bioretention System in Omaha, Nebraska

8.6.1.2 Advantages and Disadvantages

Advantages	Disadvantages
High volume reduction, depending upon site conditions	Easily clogged with suspended sediment
Low Median Effluent Concentrations	Higher construction costs per impervious acre
Aesthetic and educational opportunities in high traffic areas	Cannot be used in areas with a high water table
Intercepts water near source, alleviating need for larger downstream stormwater	Cannot be used in drainage areas with slopes > 20-percent
Effective in a "treatment train" with BMPs that reduce sediment loads	May not effectively remove pollutants when first brought on-line
Minimal footprint (1 to 15-percent of drainage area)	
Function increases with time as vegetation becomes established	
May contribute to groundwater recharge	

8.6.1.3 Design Requirements and Considerations

The procedure for designing a bioretention system is outlined below. The design components are described in the order of construction starting with excavation for construction of the underdrain and continuing through bioretention soil mix, ponding area, and high flow structures. Appendix F provides an example of a complete specification for a bioretention system. A typical cross section of a bioretention system with a valve outlet is shown in Figure 8-15.

Overall Design Guidance

- Bioretention systems should not be constructed until the entire drainage area is permanently stabilized against erosion, a pre-treatment practice is implemented, or runoff is bypassed around the facility during construction. Heavy sediment loads to the bioretention system will reduce infiltration rates and require reconstruction to restore its defined performance.

- The bioretention system ponding area should be designed to capture the required design volume (V_D). The design volume is equal to the WQCV unless routing of impervious areas to

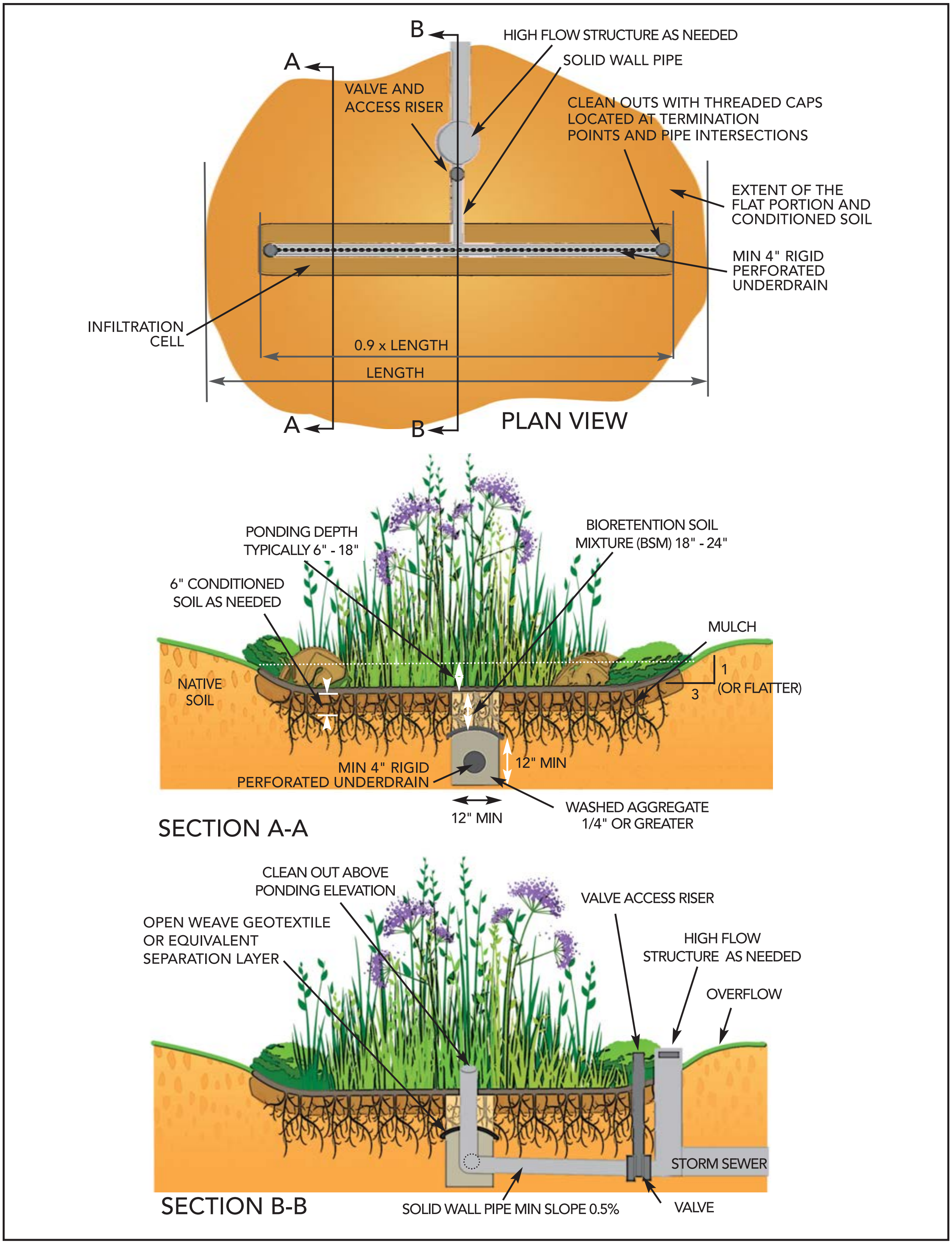


Figure 8-15 Cross Section Schematic of Bioretention System with Valve Outlet or Orifice Control

pervious areas (i.e. cascading planes) occurs within the drainage area of the bioretention system. The WQCV is based on 0.5 in. of runoff. If cascading planes are present, the design volume can be reduced because a portion of the WQCV from the impervious area is infiltrated. Section 8.3.4 provides additional guidance on calculating the design volume for drainage areas with cascading planes.

- The design volume should be able to discharge through the bioretention soil mix within 24 – 48 hrs.
- The tributary area for a bioretention system should be less than 4 ac. Multiple bioretention systems may be required for larger tributary areas (EPA, 1999).

Excavation

Excavation is required to construct the ponding area and bioretention underdrain system. The bioretention system can be partially excavated to within 6 in. of elevation of the bottom of the ponding area before final stabilization of the tributary area and utilized for erosion and sediment purposes, such as a temporary sediment basin. After stabilization is complete, all sediment should be cleared from the bioretention system, and it should be excavated to the elevation of the ponding area. Excavation of the infiltration area that will receive the bioretention soil mixture (BSM) and underdrain should be performed using a raking motion with the bucket teeth. Smearing of the soil below the underdrain and alongside the BSM infiltration area should be avoided.

The BSM and underdrain system should not be placed until the entire drainage area has been stabilized. Bioretention system side slopes should be excavated at 3:1 or flatter. Low ground-contact pressure equipment, such as excavators and backhoes, is preferred on bioretention systems to minimize disturbance to established areas around the perimeter of the infiltration area. No heavy equipment should operate within the perimeter of a bioretention system during underdrain placement, backfilling, planting, or soil conditioning of the garden.

Underdrain/Outlet

An underdrain structure allows operators to collect the runoff that filters through the system and release it to an existing storm sewer system. Key components of an underdrain/outlet for a bioretention system include:

- 4-in. or larger perforated rigid pipe that extends 90% of the longest side of the system
- All joints & connects should be properly adhered together.
- A valve should be placed at the downstream end of the underdrain and immediately upstream of discharge point or high flow structure. A valve allows the operator to regulate outflow, increase the detention time, and promote deep infiltration."
- The underdrain should be surrounded by a minimum of 4-in. of washed aggregate, ¼" or larger in size. An appropriate geotextile can be used to wrap the aggregate & underdrain, but recent field experiences have shown that the overlap in geotextile on the side & a top profile that is not flat provides for the best flow into the underdrain system. If geotextile is used only to separate the bioretention soil mix (BSM) from the underdrain aggregate, an open weave material or equivalent should be used in an upward arching manner that limits clogging of the material. Field experiences have shown that geotextile laid flat across the top of the aggregate will clog rapidly & fail to drain properly."

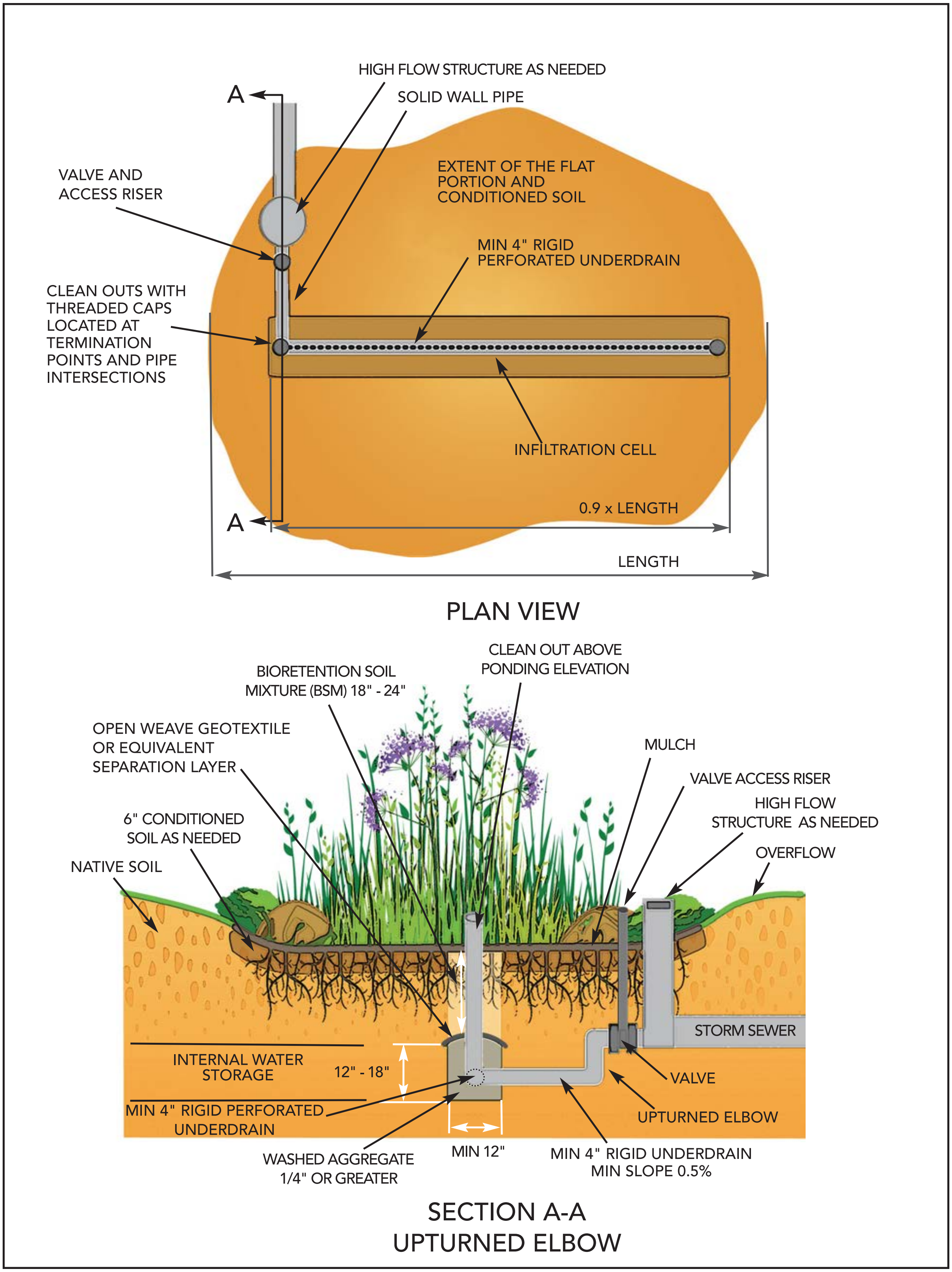


Figure 8-16 Cross Section Schematic of Bioretention System with Upturned Elbow Outlet Control

The use of a valve provides flexibility in order to optimize water quality benefits by increasing the draindown time and can also be used to manage water levels to promote vegetation growth during seasonal fluctuations in precipitation.

Valves should be selected such that the valve can be opened or closed to discharge the design volume from the system over 24 - 48hrs. Ensure proper access to the valve to allow for maintenance activities and adjusting the opening..

The upturned elbow design allows for an internal storage volume designed to provide enhanced nutrient removal. The upturned elbow control consists of a 90-degree elbow connected to the underdrain as illustrated in Figure 8-16. The upturned elbow is designed such that the outflow elevation is 18 in. below the surface elevation of the filter area.

Infiltration Cell and BSM

Bioretention systems must utilize a BSM or other appropriate infiltration media in conjunction with the underdrain. The BSM is typically comprised of a mixture of sand and compost. Specifications for both the fine sand and compost are included in Appendix F. The depth of the BSM should be between 18-48 in. depending upon the type of outlet structure. Table 8-12 below shows the maximum allowable BSM depth based on underdrain type.

Table 8-12
BSM Depth Based on Outlet Control

Underdrain Type	Depth of BSM
Orifice / Valve	18- 24 inches
Upturned Elbow (overflow set at 18 inches below top of BSM)	48 inches

High Flow and Overflow Considerations

A high flow outlet structure is necessary if a bioretention system is located in an area where adjacent property could be damaged by stormwater in the event of overtopping or within the Combined Stormwater System limits, where discharge rates are required to be less than, or equal to the existing condition. The high flow system is typically a compound system designed to convey the 10-yr event up to the 100-yr event.

An overflow path should always be designated in the event a system capacity is exceeded. The intent of the overflow path is to provide stormwater a conveyance route that avoids the potential for property damage.

Ponding Area

The ponding area of the bioretention system must be sized to capture the WQCV. The ponding depth should be minimized to reduce the hydraulic load on soils and stress on the vegetation. Ponding depths range from 6 to 18 in. for the design volume with a maximum depth of 24 in. Soil in the ponding area outside the BSM filter area may require amendments such as tiling or filling with additional organic matter, such as compost. Additional information can be found in Section 8.6.9 Soil Conditioning.

Inlet & Pretreatment

The inlet to the bioretention system is important because it is the most susceptible to damage. Damage can occur in the form of erosion and/or sediment deposition. Erosion can be mitigated through the use of energy dissipation products. The designer should provide calculations that support the selected measure for energy dissipation. Sedimentation can be managed by the use of a pretreatment facility that captures it and allows it to be easily removed. There are many forms of pretreatment that can be used including, but not limited to sump structures, forebay, or other manufactured system.

Vegetation

Native and adapted plant species improve the physical and chemical processes in soil. In the Midwest, native and adapted plants can withstand the climatic variability typical throughout the Omaha region. Guidelines for using native and adapted vegetation are included in the *Nebraska Bioretention and Rain Garden Plants Guide* published by the University of Nebraska-Lincoln Extension (Rodie and Todd, 2010).

8.6.1.4 Inspection and Maintenance

Maintenance activities for bioretention systems include short-term and long-term maintenance tasks.

Short Term: Year 1 – Year 3

1. Water young plants and seedlings a minimum of weekly for the first three months. Watering may be required biweekly during the summer months (June through August) the first year.
2. Eliminate weeds using spot application of herbicide or pulling throughout the first year.
3. After rainfall equaling or exceeding 0.5 in.
    a. Redistribute mulch, remove trash, and inspect vegetation.
    b. If sediment has accumulated, remove it and replace mulch and vegetation as needed.
    c. Check for erosion inside and around the bioretention system. Repair erosion damage if it occurs.
4. Repair or restore clogged high flow structures as needed.
5. Clean underdrain if clogged.
6. At one year after installation, inspect vegetation. Replace dead plants and remove invasive plant species.
7. At least 2 times per year, operate valve (if installed) to fully open then fully closed and reset at designed opening.

Long Term: Year 3 - later

1. In early spring, mow or trim vegetation to a height greater than 6 in. Remove accumulated debris.
2. Inspect vegetation one to two times each year and remove weeds and invasive species.
3. Trim back or remove overgrown vegetation.
4. Repair or restore clogged high flow structures as needed.
5. Clean underdrain if clogged.
6. At least 2 times per year, operate valve (if installed) to fully open then fully closed and reset at designed opening.

8.6.1.5 Submittal Requirements

For review purposes prior to construction, the following minimum submittal requirements are recommended:

- Drainage area map, including drainage area to bioretention system(s).
- Existing and proposed contour map of site (1-ft. contours recommended with elevations referenced to NAVD 88). Additional spot elevations as needed to establish design critical elevations such as top of overflow weir or pipe inverts.
- Geotechnical investigation of site (soil borings, water table location).
- Stormwater plan/profile for site.
- Bioretention system plan view, typical cross section, and profile view. Components clearly labeled with dimensions.
- Hydrologic calculations (refer to Design Example). The designer should include necessary design calculations to show that flow is unconcentrated prior to entering the bioretention system or provide for energy dissipation at the point of flow entry.
- Detail of any proposed underdrain, outlet piping, control structure, and/or overflow structures with dimensions for construction. Include appropriate design calculations (refer to Design Example).
- Vegetation plan with schedule for installation and initial maintenance. Appropriate erosion control measures should be included.
- An as-built survey of the bioretention system is recommended to confirm actual construction adheres to approved construction plans.
- Long-term inspection/maintenance plan.

8.6.1.6 Design Calculations

A short summary of the design calculations is presented below. A detailed design example is outlined in Section 8.6.1.7.

Step 1 Determine the WQCV and V_D. The WQCV is calculated by multiplying the disturbed drainage area by the runoff control volume of 0.5 in. as discussed in Section 8.3.1. The design volume is equal to the WQCV unless routing of impervious areas to pervious areas (i.e. cascading planes) occurs within the drainage area of the bioretention system. The WQCV is based on 0.5 in. of runoff. If cascading planes are present, the design volume can be reduced because a portion of the WQCV from the impervious area is infiltrated. Section 8.3.4 provides additional guidance on calculating the design volume for drainage areas with cascading planes.

Step 2 Estimate size of the ponding area. The ponding area is sized to capture the design volume. Equation 8-1 should be used to calculate an initial estimate of the size of the ponding area. The final ponding area and ponding depth should be determined using the design volume and the proposed contours.

$$A_P = \frac{V_D}{0.7 * h_{max}} \tag{8-1}$$

A_P = Ponding Area (ac.)
V_D = Design volume (ac.-ft.)

h_{max} = Maximum ponding depth above bottom of ponding area (ft.)

Determine the approximate length and width of the ponding area. If possible the length should be twice the width. When the ponding bottom area width is greater than 20 ft., it will be necessary grade the bottom of the ponding area at 0.5-percent slope toward the filter area.

Step 3 Size the bioretention system BSM filter area. The size of the BSM filter area is based on the design volume and BSM characteristics according to Equations 8-2 and 8-3.

$$A_F = \frac{V_D}{\theta_S * k * t_f} \tag{8-2}$$

Where:

A_F = Filter bed surface area (ac.)
V_D = Design volume (ac.-ft.)
Θ_S = Porosity of the BSM = approximately 0.437
t_f = Time required for V_D to filter through soil (days) = 1 day
k = BSM infiltration rate (ft. per day) = approximately 20 ft. per day based on monitoring data from Orchard Park outflow system comprised of 4-in. perforated PVC surrounded by 1-in. washed river rock.

The length of the filter area shall be a minimum of 90-percent of the length of the bottom of the ponded area.

$$L_f(ft) = 0.90 * L_P \tag{8-3}$$

Where:

L_f = Filter bed length (feet)
L_P = Ponding area length (feet)

Step 4 Size the underdrain. The underdrain pipe should be rigid with a diameter of at least 4 in. to prevent clogging. For this example, we will use the 4-in. diameter pipe. The depth of the gravel layer above the underdrain pipe should be at least 4 in. The recommended perforation diameter is 0.25 to 0.375 in. The recommended longitudinal center to center perforation spacing is 6 in. A minimum of 4 perforations per row is required.

Ensure that the slope for all underdrain pipes (G_{pipe}) is less than 0.5-percent and that one cleanout is provided at the upstream end of each pipe run and immediately upstream of the flow control device.

Step 5 Select the valve or flow control device. The valve or flow control device is sized to provide a 24-hr. draindown time of the design volume. Flow control devices are sized using an average flow rate which is calculated using Equation 8-4.

$$Q_{FC} = \frac{V_D}{24\text{ hr}} * \frac{1\text{ hr}}{3{,}600\text{ sec}} \tag{8-4}$$

Where:

Q_{FC} = Average flow through flow control device or valve (cfs)
V_D = Design volume (cu. ft.)

Conversion: 1 ac-ft = 43,560 cu. ft.

Use the average flow rate and the average depth of water above the center of the valve or flow control device to size the opening or select a valve size. Valve manufacturers publish valve coefficients to aid in valve selection. Choose a valve coefficient that falls in the middle of the valve operating range to allow for adjustment above or below the average flow. The design valve coefficient is calculated using Equation 8-5.

$$Cv = \frac{Q_{FC}}{\sqrt{\frac{h\gamma}{144}}} \tag{8-5}$$

Where:

Cv	=	Valve coefficient
Q_{FC}	=	Average flow through flow control device or valve (gpm)
h	=	Head immediately upstream of valve (ft)
γ	=	Specific weight of water (lb per ft^3) = 62.45 lb/ft^3

Conversion : 1 cfs = 448.8 gpm

In this type of low velocity service, head loss on piping less than 100-ft long is negligible and can generally be ignored.

Step 6 Identify appropriate vegetation. Consult *Nebraska Bioretention and Rain Garden Plants Guide* (Rodie and Todd, 2011) published by the University of Nebraska-Lincoln Extension office to select vegetation based on exposure, soil type, soil moisture, location, salt tolerance and desired aesthetics. Seeds should not be used to establish vegetation. Plants should be provided in 1-5 gallon pots or deep celled plugs. Consult the University of Nebraska-Lincoln Extension guide to Bioretention Gardens: A Manual for Contractors in the Omaha Region to Design and Install Bioretention Gardens Chapter 5 for additional guidance on planting requirements for bioretention systems.

8.6.1.7 Example

Design a bioretention system for a small parking lot median of a local grocery store. The median captures runoff from a 1.7 ac. site with a 0.5 ac. parking lot, 1 ac. of roof and 0.2 ac. of open space. The roof of the grocery store is directly connected to the parking lot. The parking lot is graded to drain to a filter strip constructed in sandy clay soil, which drains to the bioretention system controlled by a valve outlet.

Step 1 Determine the WQCV and V_D. The drainage area, A_T, is 1.7 ac. (1 ac. of rooftop, 0.5 ac. of parking and 0.2 ac. of filter strip).For this example, a filter strip provides pretreatment (see Section 8.6.4 for filter strip design). In addition, routing of impervious area to pervious area (cascading planes) reduces the design volume of the bioretention system because a portion of the runoff from the impervious area is infiltrated.

Table 8-10 is used to estimate the reduction in runoff volume through the filter strip using the percent imperviousness of the cascading plane and soil classification of sandy clay loam. The percent impervious of the cascading planes is equal to 1.5 ac. of impervious area divided by 1.7 ac. of cascading plan (0.2 ac. pervious area of the filter strip plus 1.5 ac. of impervious), or 88 percent. Interpolating from the values in Table 8-10 indicates that 0.0178 in. of runoff infiltrates into the filter strip. Therefore, the V_D for the bioretention system is 0.5 in. minus 0.0178 in. equals 0.48 in. The V_D is 0.48 in. over the drainage area of 1.7 ac. Thus, V_D equals (0.48 divided by 12 times 1.7) equals 0.068 ac.-ft. or 2,962 ft^3.

Step 2 Size the ponding area. The ponding area is sized to hold the design volume. Use Equation 8-1 to estimate the size of the ponding area. Ponding depths (h_{max}) for the design volume should range from 6 to 12 in. To maximize infiltration and reduce the hydraulic load on soils, we will design for a conservative 6-in.

ponding depth. The value of h_{max}, should be in ft. Thus, h_{max} equals 0.5 ft. The final ponding area should be determined by the final grading plan.

$$A_P(acres) = \frac{0.068\ ac-ft}{0.7*0.5\ feet} = 0.194\ acres\ or\ 8{,}463\ ft^2$$

Determine the approximate length and width of the ponding area. If possible the length should be twice the width. When the ponding area bottom width is greater than 20 ft., it will be necessary to grade the bottom of the ponding area at 0.5-percent slope toward the filter area. From the proposed contours the length of the ponding area measured 116 ft. and the width measured 70.5 ft.

Step 3 Size the bioretention system BSM filter area. The size of the BSM filter area is based on the design volume and BSM characteristics according to Equations 8-2 and 8-3. The recommended filter media was used for this example which has a permeability of 10 in. per hr. or 20 ft. per day.

$$A_F = \frac{V_D}{\theta_S*k*t_f} = \frac{0.068}{0.437*20*1} = 0.008\ acres\ or\ 338.9\ ft^2$$

The length of the filter area shall be a minimum of 90-percent of the length of the bottom of the ponded area.

$$L_f(ft) = 0.90 * L_P = 0.9 * 116\ ft$$

As shown in Table 8-12, the filter area BSM bed depth can be a maximum of 18-24 in. for this example because the design will include a valve outlet control. A depth of 24 in. is used in the example calculations.

Step 4 Size the underdrain. The underdrain pipe diameter should be at least 4 in. to prevent clogging. For this example, we will use the 4-in. diameter pipe. The depth of the gravel layer above the underdrain pipe should be at least 4 in.. The recommended perforation diameter is 0.375 in. The recommended longitudinal center to center perforation spacing is 6 in. A minimum of 4 perforations per row is recommended.

Ensure that the slope for all underdrain pipes (G_{pipe}) is less than 0.5-percent and that one cleanout is provided at the upstream end of each pipe run and immediately upstream of the flow control device. In this example, a valve will be placed at the end of the underdrain to allow for control of outflow.

Step 5 Select the valve or flow control device. The valve or flow control device is sized to provide a 24-hr. draindown time of the design volume.

$$Q_{FC} = \frac{2{,}962\ \text{ft}^3}{24\ \text{hr}} \times \frac{1\ \text{hr}}{3{,}600\ \text{sec}} = 0.034\ \text{cfs or } 15.3\ \text{gpm}$$

Conversion: 1 cfs = 448.8 gpm

For this example, a flow control valve will be used. The average flow rate and average depth of water above the center of the valve is used to calculate the flow coefficient of the valve (Cv).

The head immediately upstream of the valve is calculated by taking the average depth of the water above the underdrain and subtracting the head loss through the pipe length upstream of the valve. The average depth of water above the underdrain is 1.25 ft. ((2 ft. + 0.5 ft.) / 2). For this example the head loss through 20 ft. of pipe upstream of the valve is considered negligible and ignored.

h = 1.25 ft

$$Cv = \frac{15.3\ gpm}{\sqrt{\frac{1.25\times 62.4}{144}}} = 21$$

The Cv is compared to manufacturer's published Cv values to determine the appropriate valve size. Using the example manufacturer's valve flow coefficients in Table 8-13; 1-in., 1.5-in., 2-in., 3-in., and 4-in. valves can all achieve a valve flow coefficient of 21. The selected valve size is the 2-in. valve because it has the most flexibility in controlling flow and falls in the middle of the operating range. A 2-in. valve will achieve a valve flow coefficient of 21 with a rotation of 40 degrees but can also be operated at a rotation as low as 10 degrees to provide greater flow restriction, if needed.

Table 8-13
Example Flow Coefficients (Cv) for Ball Valves

Valve Size, in	Valve Rotation in Degrees								
	10	20	30	40	50	60	70	75[1]	80
1	0.04	0.5	3	5	8	12	17	24	31
1.5	0.07	3	7	12	21	29	40	55	68
2	0.09	4	10	21	34	43	65	77	85
3	1	12	29	49	69	118	160	208	220
4	4	20	43	84	128	192	278	340	419

[1] Maximum recommended controllable Cv.

Step 6 Identify appropriate vegetation. Consult *Nebraska Bioretention and Rain Garden Plants Guide* published by the University of Nebraska-Lincoln Extension office to select vegetation based on exposure, soil type, soil moisture, location, salt tolerance and desired aesthetics. Seeds should not be used to establish vegetation. Plants should be provided in 1-5 gallon pots or deep celled plugs. Consult the University of Nebraska-Lincoln Extension guide to Bioretention Gardens: A Manual for Contractors in the Omaha Region to Design and Install Bioretention Gardens Chapter 5 for additional guidance on planting requirements for bioretention areas.

8.6.1.8 References

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MARC. 2009. Manual of Best Management Practices for Stormwater Quality - Second Edition. http://kcmetro.apwa.net/chapters/kcmetro/specs/APWA_BMP_ManualAUG09.pdf.

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8.6.2 Constructed Wetland

Constructed wetlands provide capture and control of the design volume using a combination of settling and biological uptake. Flow through the wetland vegetation's roots removes nutrients and dissolved pollutants from the stormwater (California Stormwater Quality Association [CASQA], 2003). A constructed wetland has varying depths of permanent pools that supports the varying vegetation and components that comprise the wetland structure.

Constructed wetlands can take the form of very shallow retention ponds or wetland-bottomed channels. An adequate base flow is needed to encourage the growth of wetland species such as rushes, willows, cattails, and reeds. The wetland vegetation slows stormwater runoff and promotes settling and biological uptake. While constructed wetlands can be utilized for drainage areas of less than 10 ac., they tend to be more reliable and efficient when constructed on a larger scale. Water budget calculations are essential to the long-term success of a constructed wetland (Section 8.3.3).

Design Considerations	
Location characteristics (Slope, Soil Type)	Slope: Near-zero longitudinal slope Soil Type: Loamy
Contributing drainage area	Sufficient to maintain permanent shallow pool
Design size	Minimum of 3% of the drainage area; Minimum (L:W) is 3:1
Detention time for the design volume	Not applicable
Median Effluent Concentrations[1]	TSS = 17.77 mg/L, TP = 0.14 mg/L, TN =1.15 mg/L, Cu = 4.23 µg/L
Implementation and Maintenance Considerations	
Potential for use with other BMPs	Works well with upstream source controls, including filter strips and swales
Maintenance	Medium – Replacement and removal of vegetation as needed; sediment removal

[1]Geosyntec Consultants and Wright Water Engineers, Inc 2008

8.6.2.1 General Application

Constructed wetlands are most successful when the upstream drainage area can provide sufficient flow to retain the normal pool depth. Flood control measures may be instituted in conjunction with the wetland basin; however, the flood control volume should have a maximum depth of 2 ft. above the design volume depth for up to 12 hrs. (MARC, 2009). If the constructed wetland is also used for detention, extended detention should not compose more than 50 percent of the storage volume, and the maximum water surface elevation should not exceed more than 3 ft. above the permanent pool (Iowa State University [Iowa], 2009). Siting is critical to the success of constructed wetlands. Retrofitting should only be considered when drainage area and site slope requirements make this BMP an applicable option.

A constructed wetland can either be used as a stand-alone facility, or in a treatment train. In a treatment train application, a constructed wetland can be used for either pre-treatment or as a downstream application. Constructed wetlands function well when used with pre-treatment such as swales/filters, BMPs that assist in removing the sediment load which help to increase the longevity of the wetland. Possibly the most effective option is to locate the constructed wetland downstream of a swale, filter strip, or a detention facility that will remove much of the sediment load. The constructed wetland provides enhanced water quality to the receiving water body. The wetland also creates wildlife and aquatic habitats, and aesthetic onsite amenities (UDFCD, 2010).

Figure 8-17 Constructed Wetland

8.6.2.2 Advantages and Disadvantage

Advantages	Disadvantages
Relatively low maintenance costs.[2]	Efficient systems require a relatively large footprint.[1]
Can provide significant water quality improvement across many pollutants, including nutrients.[1]	Sufficient drainage area is required in order to retain the normal pool, making applications for larger drainage areas more successful.[1]
Enhancement of vegetation diversity and wildlife habitat.[2]	A near-zero longitudinal slope through the wetland is required; this BMP should not be constructed on steep unstable slopes. [1]
Protects downstream water bodies.[1]	Frequent inspection isnecessary to monitor for overgrowth of vegetation, nuisance vegetation, and animals.[1]

[1]MARC, 2009;
[2]Iowa 2009

8.6.2.3 Design Requirements and Considerations

Constructed wetlands can take on many configurations depending on the site. Figure 8-18 provides an example of a constructed wetland plan and profile view. The following paragraphs describe site considerations, vegetation considerations, and operation and maintenance considerations to take into account when designing constructed wetland

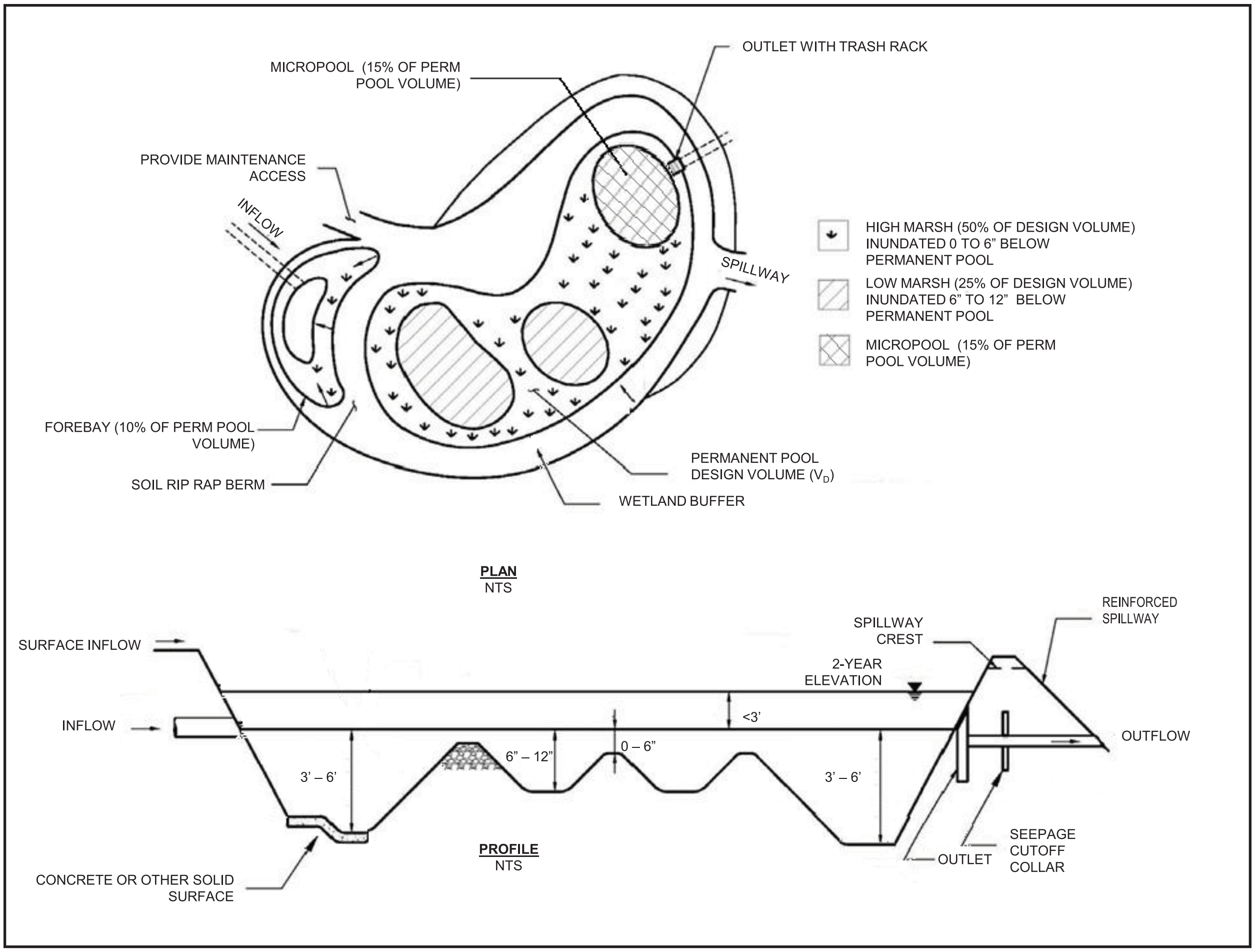

Figure 8-18 Example of a Constructed Wetland Plan and Profile View (Modified from UDFCD,2010)

Site Considerations

- A water budget analysis (Section 8.3.3) is needed to ensure that wetland hydrology will be established. The soil must be suitable for wetland vegetation. Hydric soils (soils that are normally saturated) are preferable (MARC, 2009). A near-zero longitudinal slope is required.
- The V_D for the constructed wetland is based on the WQCV. If routing of impervious area to pervious area (i.e. cascading planes) occurs within the drainage area of the wetland, the design volume of the wetland can be reduced because a portion of the WQCV from the impervious area is infiltrated. Refer to Section 8.3.4 to determine the reduced WQCV to use for sizing the wetland.
- The constructed wetland should be designed in order that the invert elevation of the outlet is set at the permanent pool elevation and sized to drain down the 2-year event within 12 hrs.
- The outlet structure shall be sized such that the peak elevation for the 10-year and 100-year design storm is less than 3 ft. above the permanent pool.
- The length to width ratio should ideally be greater than 3:1.

- A wetland soil depth of at least 4 in. should be used for surfacing shallow wetland basins.
- An effective wetland should consist of areas that have varying water depths. The different area depths should be very shallow, moderately shallow and deeper pools as described in the table below:

Wetland Water Depth Breakout	
Low Marsh (6-12” Depth)	25% of permanent pool volume
High Marsh (<6” Depth)	50% of permanent pool volume
Micropool and Forebay (3-6’ Depth)	25% of permanent pool volume

- The deeper area of the wetland should include the outlet structure so outflow from the basin is not interfered with by sediment buildup.
- A forebay, 3 to 6 ft. in depth, should be established at the wetland inflow point to capture larger sediments. Direct maintenance access to the forebay should be provided with access 15 ft. wide minimum and 5:1 slope maximum. Sediment depth markers should be provided.
- If high water velocity is a potential problem, some type of energy dissipation device should be installed.
- The designer should maximize use of pondscaping design features to create both horizontal and vertical diversity and habitat.
- Wetland bench along perimeter.
- Outlet structure with removable logs or valve to control water levels.

Vegetation Considerations

- A minimum of 3 wetland species of vegetation should be planted 2 ft. on center within the area of wetland that contains approximately 6 in. of water or less.
- Three additional wetland species (facultative wetland species) of vegetation should be planted in clumps of 5 in saturated soil outside of the frequently inundated area, with a spacing of 3 ft. on center.
- A minimum 25-ft. buffer should be established and planted with native riparian and upland vegetation (50-ft. buffer if wildlife habitat value required in design). Wetlands constructed for mitigation may have more restrictive buffer requirements.
- Surrounding slopes should be stabilized by planting in order to trap sediments and some pollutants and prevent them from entering the wetland.

Operations and Maintenance Considerations

- A written maintenance plan should be provided and adequate provision made for ongoing inspection and maintenance, with more intense monitoring activity for the first three years after construction. Wetlands constructed for mitigation may have different requirements for inspection, maintenance, and reporting.

- The wetland should be maintained to prevent loss of area of ponded water available for emergent vegetation due to sedimentation and/or accumulation of plant material.
- To minimize maintenance as much as possible, it is recommended that wetland basins be installed on stabilized watersheds and not be used for sediment control unless used in conjunction with an upstream sediment basin.
- Frequent harvesting of wetland vegetation increases nutrient removal. Removal of the plant material before winter die-off prevents nutrients from reentering the water and being transported downstream.

8.6.2.4 Inspection and Maintenance

Maintenance activities for constructed wetlands include short-term and long-term maintenance tasks.

Short Term: Year 1 – Year 3

1. Eliminate weeds and non-native species by hand throughout the first year.
2. After each rainfall equaling or exceeding 0.5 in:
   a. Remove trash and inspect vegetation.
   b. If sediment has accumulated, remove it and replace vegetation as needed. Sediment should be removed from the main pool when 10 to 15 percent of the constructed wetland normal pool is lost. (MARC, 2009)
   c. Check for erosion inside and around the constructed wetland. Repair erosion damage if it occurs.
   d. Repair or restore clogged high flow structures as needed.
3. At one year after installation, inspect vegetation. Replace dead plants and remove invasive plant species.

Long Term: Year 3 – later

1. In early spring, remove any accumulated debris.
2. Inspect vegetation one to two time each year and remove weeds and invasive species.
3. Repair or restore clogged high flow structures as needed.
4. After each rainfall equaling or exceeding 0.5 in.:
   a. Remove trash and inspect vegetation.
   b. If sediment has accumulated, remove it and replace vegetation as needed. Sediment should be removed from the main pool when 10 to 15 percent of the constructed wetland normal pool is lost. (MARC, 2009)
   c. Check for erosion inside and around the constructed wetland. Repair erosion damage if it occurs.
   d. Repair or restore clogged high flow structures as needed.

8.6.2.5 Submittal Requirements

For review purposes prior to construction, the following minimum submittal requirements are recommended:

- Drainage area map, including drainage area to the constructed wetland.
- Existing and proposed contour map of site with elevations referenced to NAVD 88 (1-ft. contours recommended). Additional spot elevations may be helpful.
- Geotechnical investigation of site (soil borings, water table location).
- Stormwater plan/profile for site.
- Constructed wetland plan view and profile view. Components clearly labeled with dimensions.
- Hydrologic calculations (refer to Design Example) and average annual water budget.
- Detail of any proposed outlet and overflow structures with dimensions for construction. Include appropriate design calculations (refer to Design Example).
- Vegetation plan with schedule for installation and initial maintenance. Appropriate erosion control measures should be included.
- An as-built survey of the constructed wetland is required to confirm actual construction adheres to approved construction plans.
- Long-term inspection/maintenance plan.

8.6.2.6 Design Calculations

A short summary of the design calculations is presented below. A detailed design example is outlined in Section 8.6.2.7.

Step 1 Determine the WQCV and wetland design volume. The WQCV is based on 0.5 in. of runoff. If routing of impervious area to pervious area (i.e. cascading planes) occurs within the drainage area of the wetland, the design volume of the wetland can be reduced because a portion of the WQCV from the impervious area is infiltrated. Refer to Section 8.3.4 to determine the reduced WQCV to use for sizing wetland.

V_D = WQCV or if cascading planes exist in the wetland drainage area, see Section 8.3.4 for calculation of V_D.

Step 2 Calculate the permanent pool design volume. The permanent pool design volume is calculated by adding 20-percent to the design volume to account for sedimentation. This total volume will be comprised of a forebay, micropool (or deepwater zone), low marsh, and high marsh.

$$V_P = V_D \times 1.2 \qquad (8\text{-}6)$$

Where:

V_P = Permanent pool volume (ac.-ft.)
V_D = Design Volume (ac.-ft.)

Step 3 Calculate the forebay volume, V_F. It is recommended that the forebay volume be 10-percent of the overall wetland permanent pool (V_P). Calculate the forebay surface area from the forebay volume and design depth. The design depth should be between 3 and 6 ft. The sides and bottom of the forebay should be paved or hardened to expedite any needed clean-out.

Step 4 Calculate the micropool, or deepwater zone, volume, V_{MP}. It is recommended that the micropool volume account for 15-percent of the overall wetland permanent pool (V_P). Calculate the micropool surface area from the micropool volume and design depth. This design depth should be between 3 and 6 ft.

Step 5 Calculate the low marsh and high marsh volumes. The remaining components of the permanent pool are the low marsh and high marsh. The low marsh should be 25-percent of the total overall constructed wetland volume, V_P. The high marsh volume comprises the remainder of the constructed wetland volume at 50-percent of the permanent pool volume.

Step 6 Determine configuration of constructed wetland. It is recommended that the flow path have a minimum length to width ratio of at least 3:1. The inlet to the wetland should distribute flows, and dissipate energy to limit erosion. The wetland should be easily accessible by maintenance vehicles.

Step 7 Select the outlet type and calculate outlet dimensions for the constructed wetland. To calculate outlet dimensions, calculate the average flow rate for the 2-year event. Determine the outlet (type and size) such that the 2-year event volume is released over a time period of 12 hrs. Procedures for calculating the 2-year event runoff volume are provided in Chapter 2: Hydrology. Outlet design must also consider facility dimensions and site constraints. Typical outlet types are also discussed in Chapter 6 Storage Facilities.

<u>All Types</u>

To calculate the average flow rate (Q_{AVG}) of the 2-year event volume over 12 hrs. use Equation 8-7.

$$Q_{AVG} = \frac{V_{2yr} * 43{,}560\frac{ft^2}{acre}}{12\ hr * 3{,}600\frac{sec}{hr}} \tag{8-7}$$

Where:

Q_{AVG} = 2-year average flow rate (cfs)
V_{2yr} = 2-year event volume (ac-ft)

Single Orifice
Orifice diameter, D_O, should be greater than 4-in. to reduce risk of clogging. If calculated D_O is less than 4 in., use a v-notch weir instead of a single orifice. To calculate the orifice diameter use Equation 8-8.

$$D_O = 2\left(\frac{Q_{AVG}}{C_O * \pi * (2 * g * H_{2yr})^{0.5}}\right)^{0.5} * \frac{12\ in}{ft} \tag{8-8}$$

Where:

D_O = Orifice diameter (in)
Q_{AVG} = 2-year average flow rate (cfs)
C_O = Orifice discharge coefficient, Where C_O = 0.66 for weir plate thickness ≤ orifice diameter, and C_O = 0.80, otherwise
g = Acceleration due to gravity (32.2 ft/s^2)
H_{2yr} = Average head of the 2-year event over orifice center; can be calculated as ½ of the 2-year depth above the outlet

V-notch Weir

Dimensions of the V-notch weir outlet include the V-notch weir angle and the top width of the V-notch opening.

$$\theta = 2 * \frac{180}{\pi} * \tan^{-1}\left(\frac{Q_{AVG}}{C_V * H_{2yr}^{5/2}}\right) \quad \text{** Note: set angles to radians on calculators and spreadsheets} \tag{8-9}$$

$$W_V = 2 * Z_{2yr} * \tan\left(\frac{\theta * \pi}{2 * 180}\right) \quad \text{** Note: set angles to radians on calculators and spreadsheets} \tag{8-10}$$

Where:

θ = Required V-notch weir angle, 20° minimum (degrees)
Q_{AVG} = 2-year average flow rate (cfs)
C_V = V-notch weir coefficient (2.5)
H_{2yr} = Average head of 2-year volume over orifice invert (ft)
W_V = Top width of V-notch weir (ft)
Z_{2yr} = Max 2-year depth above outlet (ft)

Sharp-Crested Weir

Equation 8-11 is used to calculate the length of a sharp-crested weir outlet.

$$L = \frac{Q_{AVG}}{C{H_{2yr}}^{1.5}} \tag{8-11}$$

Where:

L = Length of sharp-crested weir (feet)
Q_{AVG} = 2-year average flow rate (cfs)
C = weir coefficient (typical C =3.3)
H_{2yr} = Average head of 2-year volume over weir crest (ft)

Step 8 Water budget calculations. Perform water budget calculations for the constructed wetland using guidelines in Section 8.3.3. Determine if saturation or inundation period is at least 8 to 19 days during the growing season using an average annual water budget. Estimating water budgets for wetter than average and drier than average years may also be helpful in determining how the wetland will function during annual fluctuations in climate.

Step 9 Determine outlet protection to avoid clogging. If the chosen outlet structure discharges to a closed system, or if debris in the outlet works would be difficult to remove, determine the appropriate outlet protection to avoid clogging. Protection from clogging may include trash racks, hoods, or reversed slope pipes. Follow guidance in Chapter 6: Storage Facilities to estimate the minimum trash rack size versus outlet diameter or minimum dimensions.

8.6.2.7 Example

Design a constructed wetland to accept runoff from a 40-ac., new single-family residential development.

Step 1 Determine the WQCV and wetland V_D. The drainage area to the constructed wetland is 40 ac. Using 0.5 in. of runoff, the WQCV is calculated as:

$$WQCV = \frac{0.5\ inches}{12\ inches} * 40\ acres = 1.67\ acre - feet$$

The drainage area does not include cascading planes, therefore, the design volume V_D is equal to the WQCV.

$$V_D = WQCV = 1.67\ acre - feet$$

Step 2 Calculate the permanent pool design volume, V_P.

The permanent pool volume can be calculated using Equation 8-6, and multiplied by 1.2 to account for an additional 20 percent volume for expected sedimentation in the constructed wetland.

$$V_P = 1.67 * 1.2 = 2.00\ acre - feet$$

This total volume will be comprised of a forebay, micropool, low marsh, and high marsh.

Step 3 Calculate the forebay volume, V_F. The forebay volume, V_F, should be 10-percent of the total overall constructed wetland volume, V_P. For this project, the forebay volume is:

$$V_F = 0.1 * 2.00\ acre - feet = 0.20\ acre - feet$$

The depth of this forebay will be 3 ft., which is within the recommended 3 to 6 ft. The surface area for the forebay is 0.07 ac. or 0.20 ac.-ft. divided by 3 ft. depth. In this development, the constructed wetland is in a very accessible area near a road. This will allow easy access to the forebay and wetland for any maintenance necessary.

Step 4 Calculate the micropool, or deepwater zone, volume, V_{MP}. The micropool volume, V_{MP}, should be 15-percent of the total overall constructed wetland volume, V_P. Therefore, it is 0.30 ac.-ft. The depth of the micropool will be 5 ft., which is within the recommended 3 to 6 ft. The surface area of the micropool is 0.06 ac., based on this depth.

$$V_{MP} = 0.15 * 2.00\ acre - feet = 0.30\ acre - feet$$

Step 5 Calculate the low marsh and high marsh volumes. The remaining components of the permanent pool are the low marsh and high marsh. The low marsh should be 25-percent of the total overall constructed wetland volume, V_P, or 0.5 ac.-ft. The average depth of the low marsh will be 9 in. The surface area of the low marsh is 0.66 ac., based on this depth. The high marsh volume comprises the remainder of the constructed wetland volume and is 0.84 ac.-ft., with an average depth of 3 in. and a surface area of 3.36 ac.

The combined area of the wetland is equal to the sum of the surface area of each of the components (forebay, micropool, high marsh and low marsh).

$A_W = 0.07\ acres + 0.06\ acres + 0.66\ acres + 3.36\ acres = 4.15\ acres$

Step 6 Determine configuration of constructed wetland. The wetland will have a linear flow path with a length to width ratio of 4:1. Flows to the wetland are discharged through a small filter strip before entering the wetland forebay. This provides energy dissipation while unconcentrating the flows. The developer will establish the wetland area when 90-percent of the upstream drainage area is stabilized, limiting the impact of sedimentation on establishment of the system.

Step 7 Select the outlet type and calculate outlet dimensions for the constructed wetland. The developer would like to install a sharp-crested weir for the constructed wetland. The outlet will release the 2-year event volume over a 12-hour period or less. To size the outlet, first the average discharge should be calculated using Equation 8-7. Procedures in Chapter 2: Hydrology were used to calculate the 2-year event runoff volume as 4.8 ac.-ft.

$$Q_{AVG} = \frac{4.8\ acre{-}ft * 43{,}560\frac{ft^2}{acre}}{12\ hrs * 3{,}600\frac{sec}{hr}} = 4.86\ cfs$$

Then, the weir length should be calculated using Equation 8-11. The maximum depth of the 2-year event above the outlet is designed to be 1.16 ft.. This depth is calculated by taking the 2-year event volume and dividing it by the total wetland area (A_W). The average depth of the 2-year event can be estimated as 0.7 times the maximum depth, or 0.81 ft. Use Equation 8-11 to estimate the length of sharp-crested weir.

$$L = \frac{Q_{AVG}}{CH_{2yr}{}^{1.5}} = \frac{4.86}{3.3 * 0.81^{1.5}} = 2.0\ ft$$

Step 8 Water budget calculations. A water budget should be performed for this site to determine if the saturation or inundation period is at least 8 to 19 days during the growing season. This will increase the likelihood that the wetland vegetation installed as part of this project will survive under local climatic conditions based on the parameters for this particular design. The developer calculated the water budget for the constructed wetland using guidelines in Section 8.3.3. Estimating water budgets for wetter than average and drier than average years may also be helpful in determining how the wetland will function during annual fluctuations in climate.

Step 9 Determine outlet protection to avoid clogging. For this example, the outlet is a 2-ft. long sharp-crested weir and the outlet discharges to an existing open channel. The 2-ft. wide outlet may trap larger debris; however, it is exposed and easy to clean; therefore, no trash rack is provided.

8.6.2.8 References

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8.6.3 Extended Dry Detention Basin

Extended dry detention basins (EDDBs) are designed to detain the WQCV for 40 hrs. to allow particles and associated pollutants to settle (UDFCD, 2005). This attenuation of the WQCV reduces the peak stormwater runoff rate for all stormwater events and reduces the effective shear stress on downstream banks (Schuler et al., 2007). EDDBs have been reported to provide between 25 and 43-percent reduction in runoff volume (Geosyntec Consultants and Wright Water Engineers, Inc 2011) and modest groundwater recharge. EDDB are also better than BMP practices that retain a constant wet volume at reducing the runoff volume of smaller, more frequent storms, comparatively reducing the frequency and volume of EDDB discharges (Geosyntec Consultants and Wright Water Engineers, Inc 2011).

Unlike retention wet ponds, these facilities do not maintain a permanent pool between storm events. EDDB outlet design is relatively smaller and extends the detention time for more frequent events. EDDBs may develop wetland vegetation in the shallow pools in the bottom portions of the facilities (e.g., sediment forebays). Wetland vegetation may enhance the basin's soluble pollutant removal efficiency through biological uptake (UDFCD, 2005). The removal performance of EDDBs for soluble pollutants, such as phosphorus, nitrogen and zinc, are more consistent than retention wet ponds or wetlands, although maximum removal rates are usually lower.

Design Considerations	
Location characteristics (Slope, Soil Type)	Slope: Sites < 15% Soil type: All
Contributing drainage area	10-50 acres (75 acres absolute maximum)[1]
Design Size	1-3% Drainage area[1] Minimum (L:W) is 2:1-4:1, Depth: 2 to 5 feet
Median Effluent Concentrations[2]	TSS = 31 mg/L; TP = 0.19mg/L; TN = 2.7 mg/L; Cu = 12.1 µg/L; Fecal Coliform = 813 cfu/100 mL
Implementation and Maintenance Considerations	
Potential for use with other BMPs	Downstream of source control BMPs
Maintenance	Low – periodic sediment/debris removal, vegetation maintenance

[1] Iowa, 2009
[2] Geosyntec Consultants and Wright Water Engineers, Inc 2008

8.6.3.1 General Application

EDDBs can be used to improve stormwater runoff quality and reduce peak stormwater runoff rates. By providing extra storage above the WQCV, an EDDB can also be used for flood control purposes. Twenty-four hrs. or more of detention in an EDDB facility will remove 90-percent of the particulate pollutants (Muthukrishnan et al., 2006). Basins constructed early in the development cycle can be used as temporary sediment basins to trap sediment from construction activities within the tributary drainage area. The accumulated sediment should be removed after upstream land disturbances cease and the tributary area is stabilized. The basin should be restored to design conditions for long-term use (MARC, 2009).

EDDBs can be used to improve the quality of urban runoff coming from roads, parking lots, residential neighborhoods, commercial areas, and industrial sites given that there is adequate land space available (UDFCD, 2005). These facilities should not be used near stream corridors or stream buffer zones. EDDBs are more efficient when used in conjunction with other BMPs, such as upstream onsite source controls, downstream infiltration/filtration basins, or swales. If desired, additional volume can be provided in an EDDB for flood control benefits (UDFCD, 2005). Issues with EDDB typically arise from clogged outlets and in-field detention times that are significantly lower than design (Galli, 1992).

8.6.3.2 Advantages and Disadvantages

Advantages	Disadvantages
Simple design, construction, and maintenance	Moderate to low removal of soluble pollutants
High sediment and adsorbed pollutant removal	Potentially large land requirements
Widespread application can reduce channel degradation caused by high sediment and runoff loads	Frequent maintenance- removal of trash and debris, vegetation
Potential for use as a flood control facility	
Reduction in stormwater runoff volume	
Opportunity for passive recreational and open space facilities	

8.6.3.3 Design Requirements and Considerations

The paragraphs below provide design requirements and considerations including site requirements, basin dimensions, basin configuration, inlet design, forebay design, outlet design and considerations for vegetation selection.

Site Requirements

EDDBs are very applicable to urban development and retrofit situations due to the low hydraulic head requirements that fit easily into existing drainage system constraints (Muthukrishnan et al., 2006). Guidelines for determining the appropriate location of an EDDB are outlined as follows.

- Other infiltration BMPs should be considered in areas with high quality and/or well drained soils (EPA, 2006).
- A maintenance ramp and perimeter access must be included in the design to facilitate access to the basin for maintenance activities (CASQA, 2003).
- Public safety must be considered in EDDB design. Fences and landscaping can be used to impede access, but should not impede sheet flow into the system. Limit access to outfall pipes (CASQA, 2003).
- The EDDB bottom should be 1 to 2 ft. above the seasonal maximum groundwater table, as groundwater may surface within the basin or contribute baseflow to the basin (UDFCD, 2005).
- Design EDDBs to deter large numbers of geese from gathering in the facility by planting a buffer of trees, shrubs, and native ground cover around the facility (MARC, 2009). Geese can add to the nutrient and fecal coliform loads entering and leaving the EDDB.

Basin Dimensions

To determine the required storage volume of an EDDB, calculate the WQCV based on the drainage area and add 20 percent to the result. The basin should be sized to detain this volume over 40 hrs. The additional volume will promote silt and sediment deposition in the EDDB. This will allow a flow through velocity that is less than the settling velocity of pollutants (Muthukrishnan et al., 2006). Basin geometry is a function of the WQCV calculated and other site characteristics. General guidelines are outlined below.

- The depth of the WQCV in the EDDB should be between 2 to 5 ft.. A shallow basin with large surface area performs better than a deep basin with the same volume (Nashville, 2006).

- Side slopes should range from 20:1 to 4:1.
- Recommended flow length to width ratio is 3:1, and should be at least 2:1 (Muthukrishnan et al., 2006). The width should gradually increase from the inlet area and then retract near the outlet area to ensure adequate detention time.
- If flood control is provided within the EDDB, refer to Chapter 6 Storage Facilities on design guidance for incorporating flood control storage
- Dams are regulated by the Nebraska Department of Natural Resources, Title 458, Nebraska Administrative Code, Chapter 1-13, June 2008. All dams must meet the regulations set forth in Title 458.
- Protection for facility embankments must be provided when flood storage is included the EDDB design. Each dam should be protected with an emergency spillway unless the principal spillway is large enough to pass the peak design flow without breaching the dam (NRCS, 2000).

Basin Configuration

The inlet of the basin should be designed to minimize runoff velocities into the basin to prevent sediment re-suspension. Runoff should flow through the inlet and into a forebay. The forebay exists to reduce sedimentation prior to the main basin and reduces overall maintenance, making it more cost-effective to remove sediments and trash from the small, easily accessed forebay than the large basin. The outlet should be designed to release the captured runoff over the 40-hr. detention time.

Inlet

- Typical inlet structures include, but are not limited to, drop manholes, rundown chutes, baffle chutes, and pipe with impact basin (Muthukrishnan et al., 2006).
- All inlets should include some type of energy dissipater to reduce sediment re-suspension (MARC, 2009).

Forebay

- The forebay should be a 4 to 6 ft. deep cell delineated by a barrier and should be sized to contain at least 10 percent of the WQCV.
- The minimum length to width ratio of the forebay should be greater than 2:1 to prevent short-circuiting (Muthukrishnan et al., 2006).

Outlet

- Locate the basin outlets as far away from the basin inlets as possible to prevent water from short-circuiting the facility (Nashville, 2006).
- Outflow structures should be protected by a well screen, trash rack, grate, stone filter, or other approved device to ensure that the outlet will remain functional and not experience blockage or clogging (KC Metro APWA, 2006).
- No single outlet orifice should be less than 4 in. in diameter (smaller orifices are more susceptible to clogging). If the calculated orifice diameter necessary to achieve a 40-hr. drawdown is less than 4 in., a v-notch weir should be used instead of a single orifice outlet (MARC, 2009).

- Keep perforations larger than 1 in. when using outlets with multiple openings. Smaller orifice sizes may be used if the weir plate is placed in a riser manhole in a sump-like condition (MARC, 2009) or is protected by a well screen.

Vegetation

Native vegetation should be used to reinforce all earthen structures and be planted along the basin perimeter to prevent erosion. Utilizing vegetation at the basin inlet will also filter incoming runoff and may reduce inlet velocities. Vegetation surrounding the outlet may serve to reduce runoff impacts on downstream areas so long as it does not promote clogging of the outlet structure.

8.6.3.4 Inspection and Maintenance

Short Term: Year 1 – Year 3 (Post-Installation)

1. Water young plants and seedlings a minimum of weekly for the first three months. Watering may be required more frequently during the summer months (June through August) during the first year. Try to maintain at least a 70-percent vegetation density to ensure stability.

2. Eliminate weeds using spot application of herbicide throughout the first year.

3. Check for signs of erosion or instability and make sure that aesthetics are maintained throughout the BMP footprint

4. After rainfall equaling or exceeding 0.5 in.:

    a. Ensure that vegetation and other erosion stabilizing mechanisms are intact and check inlet/outlet structures and surrounding area for signs of erosion or instability.

    b. Inspect all inlet/outlets and repair or restore clogged flow structures as needed.

    c. Remove sediment and debris from pretreatment BMPs or forebay.

    d. Confirm drainage system functions and bank stability.

5. At one year after installation, inspect vegetation and all other supporting structure. Replace dead plants and remove invasive plant species.

6. Removed sediments should be tested for toxicants and should comply with local disposal requirements.

Long Term: Year 3 – later

1. In early spring, mow or trim vegetation to a height of no less than 6 in. Remove accumulated debris.

2. Inspect vegetation one to two times each year and remove weeds and invasive species.

3. Trim back or remove overgrown vegetation.

4. Repair or restore clogged flow structures as needed.

5. At least twice a year, check for subsidence, erosion, cracking/tree growth on the embankment, sediment accumulation around the outlet, and erosion within the basin and banks.

6. Removed sediments should be tested for toxicants and should comply with local disposal requirements.

8.6.3.5 Submittal Requirements

For review purposes prior to construction, the following minimum submittal requirements are recommended:

- Drainage area map, including drainage area to the EDDB
- Existing and proposed contour map of site with elevations referenced to NAVD 88 (1-ft. contours recommended). Additional spot elevations may be helpful.
- Geotechnical investigation of site (soil borings, water table location).
- Stormwater plan/profile for site.
- EDDB plan view and profile view. Components clearly labeled with dimensions.
- Hydrologic calculations (refer to Design Example).
- Detail of any proposed outlet and overflow structures with dimensions for construction. Include appropriate design calculations (refer to Design Example).
- Vegetation plan with schedule for installation and initial maintenance. Appropriate erosion control measures should be included.
- An as-built survey of the EDDB is recommended to confirm actual construction adheres to approved construction plans.
- Long term inspection/maintenance plan with responsible party and dedicated funding source.

8.6.3.6 Design Calculations

A short summary of the design calculations is presented below. A detailed design example is presented in Section 8.6.3.7.

Step 1 Determine the WQCV and EDDB design volume. The WQCV is based on 0.5 in. of runoff. If routing of impervious area to pervious area (i.e. cascading planes) occurs within the drainage area of the EDDB, the design volume of the EDDB can be reduced because a portion of the WQCV from the impervious area is infiltrated. Refer to Section 8.3.4 to determine the reduced WQCV to use for sizing EDDB.

V_D = WQCV or if cascading planes exist in EDDB drainage area, see Section 8.3.4 for calculation of V_D.

Step 2 Determine the EDDB volume. The EDDB volume is calculated by increasing the V_D by 20-percent to account for sedimentation.

$$V_{EDDB} = \mathrm{V_D} * 1.20 \qquad \textit{(8-12)}$$

Step 3 Size the outlet. Determine the outlet type and size such that the V_{EDDB} is detained and released over 40 hrs. Outlet design must also consider facility dimensions and site constraints. For sizing all EDDB outlets, first calculate the average discharge rate for the V_{EDDB} using Equation 8-13.

Average Discharge Rate

$$Q_{AVG} = \frac{V_{EDDB} * 43{,}560\frac{ft^2}{acre}}{40\ hrs * 3{,}600\frac{sec}{hr}} \qquad \textit{(8-13)}$$

Where:

Q_{AVG} = Average discharge rate for the V_{EDDB}(cfs)
V_{EDDB} = EDDB Volume (ac-ft)

Next the Q_{AVG} is use to calculate dimensions for a single orifice or v-notch weir outlets.

Single Orifice

$$D_O = 2\left(\frac{Q_{AVG}}{C_O * \pi * (2 * g * H_{EDDB})^{0.5}}\right)^{0.5} * \frac{12\ in}{ft} \qquad (8\text{-}14)$$

Where:

D_O = Orifice diameter (in)
Q_{AVG} = Average discharge rate for the V_{EDDB} (cfs)
C_O = Orifice discharge coefficient, Where C_O = 0.66 for weir plate thickness ≤ orifice diameter, and 0.80, otherwise
g = Acceleration due to gravity (32.2 ft/s)
H_{EDDB} = Average head of V_{EDDB} (ft)

V-Notch Weir

Dimensions of the V-notch weir outlet include the V-notch weir angle and the top width of the V-notch opening.

$$\theta = 2 * \frac{180}{\pi} * \tan^{-1}\left(\frac{Q_{AVG}}{C_V * H_{EDDB}^{5/2}}\right) \text{ ** Note: set angles to radians on calculators and spreadsheets} \qquad (8\text{-}15)$$

$$W_V = 2 * Z_{EDDB} * \tan\left(\frac{\theta * \pi}{2 * 180}\right) \text{ ** Note: set angles to radians on calculators and spreadsheets} \qquad (8\text{-}16)$$

Where:

θ = Required V-notch weir angle, 20° minimum (degrees)
Q_{AVG} = Average discharge rate for the V_{EDDB} (cfs)
C_V = V-notch weir coefficient (2.5)
H_{EDDB} = Average head of V_{EDDB} over orifice invert (ft)
W_V = Top width of V-notch weir (ft)
Z_{EDDB} = Max depth above outlet (ft)

Sharp-Crest Weir

Equation 8-17 is used to calculate the length of a sharp-crested weir outlet.

$$L = \frac{Q_{AVG}}{C H_{EDDB}^{1.5}} \qquad (8\text{-}17)$$

Where:

L = Length of sharp-crested weir (feet)
Q_{AVG} = Average discharge rate for the V_{EDDB} (cfs)
C = weir coefficient (typical C =3.3)
H_{EDDB} = Average head of V_{EDDB} over weir crest (ft)

Step 4 Determine outlet protection to avoid clogging. If the chosen outlet structure discharges to a closed system, or if debris in the outlet works would be difficult to remove, determine the appropriate outlet protection to avoid clogging. Outlet protection to avoid clogging may include trash racks, hoods, or reversed slope pipes. Follow guidance in Chapter 6 Storage Facilities to estimate the minimum trash rack size versus outlet diameter or minimum dimensions.

Step 5 Determine the forebay volume. The forebay should be sized for 10-percent of the V_{EDDB}, with a depth of 4 to 6 ft. and a minimum length to width ratio of 2:1.

Step 6 Determine basin sideslopes. Meet guidelines for basin side slopes and dam embankment side slopes. These typically range from 20:1 to no steeper than 4:1.

Step 7 Include flood control is applicable. If designing the EDDB to include flood control storage, follow guidelines in Chapter 6 Storage Facilities.

8.6.3.7 Example

Design an EDDB for a new strip mall on previously undeveloped land. The EDDB should accept runoff from a 20-ac. drainage area. The site is required to limit 2-, 10- and 100- year post-project flow rates to pre-project flow rates.

A depth-area relationship for the EDDB is derived using the proposed contours for the facility. The depth-area relationship below was used for this example.

Depth-Area Relationship for EDDB Example Calculations

Depth[1], ft	Area, acres	Volume, ac-ft
0	0.8	0
1.22	0.9	1.04
2	1.0	1.78
3	1.1	2.8
4	1.5	4.1

[1] Depth above basin bottom.

Step 1 Determine the WQCV and EDDB design volume. The drainage area to the EDDB is 20 ac. Using 0.5 in. of runoff, the WQCV is calculated as:

$$WQCV = \frac{0.5\ inches}{12\ inches} * 20\ acres = 0.83\ acre - feet$$

The drainage area does not include cascading planes, therefore, the design volume V is equal to the WQCV.

$$V_D = WQCV = 0.83\ acre - feet$$

Step 2 Determine the EDDB volume. EDDB volume is equal to 1.2 times the WQCV.

$$V_{EDDB} = 0.83\ acre - feet * 1.20 = 1.0\ acre - feet$$

Step 3 Size the outlet. The developer would like to install a v-notch outlet for this particular EDDB. To size the notch angle, use Equation 8-13 to calculate the average flow rate from the outlet.

$$Q_{AVG} = \frac{1.0\ ac{-}ft \times 43{,}560 \frac{ft^2}{acre}}{40\ hrs * 3{,}600 \frac{sec}{hr}} = 0.30\ cfs$$

Then, the v-notch angle should be calculated using Equation 8-15. The average depth of the V_{EDDB} above the outlet is 0.61 ft.

$$\theta = 2 * \frac{180}{\pi} * tan^{-1}\left(\frac{Q_{AVG}}{C_V * H_{EDDB}^{5/2}}\right) = 2 * \frac{180}{3.14} * tan^{-1}\left(\frac{0.30}{2.5 * 0.61^{5/2}}\right) = 45\ degrees$$

Step 4 Determine outlet protection to avoid clogging. For this example, the outlet is a 45-degree v-notch weir. The 45-ft. wide outlet may trap larger debris; however, it is exposed and easy to clean; therefore, no trash rack is provided.

Step 5 Determine the forebay volume. A forebay is recommended. The forebay should be 4 to 6 ft deep and sized to contain at least 10 percent of the V_{EDDB}, or 0.1 ac.-ft. The minimum length to width ratio should be greater than 2:1.

Step 6 Determine basin sideslopes. The EDDB should be located at the lower side of the development area in order to accept drainage from the development. The pond should have a minimum length to width ratio of 2:1. The overall pond depth should be between 2 and 5 ft., with side slopes no greater than 4:1.

Step 7 Include flood control if applicable. For this design example, a high-flow structure is required to control the post-project 2-, 10-, and 100-year peak flow to equal the pre-project peak flow rates. Guidelines in Chapter 2 and Chapter 6 shall be used when sizing high flow structures to control peak flows. For this example, the USACE's Hydrologic Engineering Center – Hydrologic Modeling System (HEC-HMS), a flood hydrograph routing package, was used to size the high flow structure. A HEC-HMS model was created using methods described in Chapter 2 and Chapter 6 to estimate a 2-year peak flow rate of 25.5 cfs, a 10-year peak flow rate of 49.2 cfs, and a 100-year peak flow rate of 81.5 cfs for pre-project conditions.

The depth-outflow relationship for the 45-degree v-notch weir was input into the HEC-HMS model along with the depth-area relationship for the EDDB. Sizing the high flow structure began with sizing a broad-crested weir to control the post-project 2-year 24-hr. storm event. The invert of the broad-crested weir must be located at the maximum depth of the V_{EDDB} which is 1.2 ft. above the basin bottom according to the depth-area relationship above. HEC-HMS was used to determine the combined 2-year peak outflow rate through the v-notch weir and the broad-crested weir.

The combined 2-year peak flow rate through the v-notch and broad-crested weirs were 25.3 cfs with a depth of 2.14 ft.. The 2-year peak flow was less than the pre-project 2-year peak flow rate of 25.5 cfs. Using a 4-ft. weir along with the 45-degree v-notch weir would meet the objectives of the high flow structure to control the 2-year peak flow rate, and was used in this design.

Next, the 10-year storm was routed through the EDDB to determine the 10-year peak flow through the v-notch weir and the broad-crested weir. The results of the 10-year run showed that the 10-year peak flow rate of 46.3 cfs was less than the pre-project rate of 49.2 cfs. Therefore, the 45-degree v-notch weir and 4 ft. broad-crested weir met requirements for controlling the 2-year and 10-year storms. The 10-year peak depth was 2.69 ft.

Finally, the 100-year storm was routed through the EDDB to determine the 100-year peak flow through the v-notch weir and the broad-crested weir. The results of the 100-year run showed that the peak flow rate for the 100-year event of 72.9 cfs was less than the pre-project 100-year peak flow rate of 81.5 cfs. Therefore, the 45-degree v-notch weir and 4 ft. broad-crested weir met requirements for controlling the 2-year, 10-year, and 100-year storm events. The 100-year peak depth was 3.42 ft.

8.6.3.8 References

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8.6.4 Bioswales and Filter Strips

Bioswales are open vegetated channels with dense vegetation covering the side slopes and channel bottom. They are used to capture and convey stormwater runoff at a non-erosive velocity and can be used as a substitute for traditional pipe systems to convey roadway, parking lot and other site drainage (MARC, 2009). The vegetation covering the sides and bottom of the channel provide a filtration surface and slows runoff velocities, traps particulate pollutants, and promotes infiltration.

A filter strip is an area of dense vegetation that accepts sheet flow runoff from adjacent surfaces. When situated properly between a pollution source and a water body or other BMP that receives runoff, filter strips slow runoff velocities and improve water quality by reducing sediment load and filtering pollutants absorbed by sediments. Water treatment with filter strips is most effective when sheet flow is maintained. When runoff from adjacent impervious surfaces concentrate and form a channel, the effectiveness of the filter strip is reduced (Muthukrishnan et al., 2006). If concentrated flows occur, flows must be converted back to sheet flow prior to entering a filter strip treatment area. In order to achieve this, grading and level spreaders are used to create a uniformly sloping area to distribute the runoff evenly across the filter strip (Iowa, 2009).

Design Considerations		
	Grass Swales	**Filter Strips**
Location characteristics	Slope: < 1-2%	Slope: < 1-6%
(Slope, Soil Type)	Soil Type: All	Soil Type: All
Contributing drainage area	< 1 acre	< 2 acres
Design size	Varies	Minimum Length = 15’
Residence time for Q_{WQ}	3-5 minutes	N/A
Median effluent concentrations[1]	TSS = 23.92 mg/L, TP = 0.34 mg/L, TN = 0.78 mg/L, Cu = 10.66 µg/L	TSS = 23.92 mg/L, TP = 0.34 mg/L, TN = 0.78 mg/L, Cu = 10.66 µg/L
Relative volume reduction[2]	Median 42%	Median 34%
Implementation and Maintenance Considerations (for Grass Swales and Filter Strips)		
Potential for use with other BMPs	**High -** Best when used as pretreatment for other BMPs such as bioretention	
Maintenance	**Low -** Sediment/debris removal, vegetation maintenence	

Note: Median Effluent Concentrations apply to events with measured discharge.

[1] Reported Median Effluent Concentrations for Biofilters (which includes grass swales, filter strip and wetland vegetation swale), Source Geosyntec Consultants and Wright Water Engineers, Inc 2008.

[2] Reported relative volume reduction = (Study Total Inflow Volume - Study Total Outflow Volume)/(Study Total Inflow Volume) Source: Geosyntec Consultants and Wright Water Engineers, Inc 2011

The ISBMPD reported relative volume reductions for both grassed strips and swales. Table 8-4 shows the median volume reduction for bioswales was 42-percent and volume reduction for grass strips was 34-percent (Geosyntec Consultants and Wright Water Engineers, Inc 2011).

8.6.4.1 Bioswales General Application

Bioswales are well suited for treating highway and residential road runoff and can serve as a drainage system to replace curb and gutter storm sewer systems, as shown in Figure 8-19 (CASQA, 2003). They are also commonly used for controlling parking lot and facility runoff. Bioswales are best utilized in treating areas of 1 ac. or less, and are only effective in providing water quality benefits if flow is shallow. Swales are especially effective when used in a series of stormwater BMP practices, such as conveying water to a detention pond (See treatment train discussion in Section 8.2.3).

Figure 8-19 Bioswale

8.6.4.2 Bioswales Advantages and Disadvantages

Advantages	Disadvantages
Improves water quality by filtering stormwater through dense vegetation	Provides effective water quality improvement in light to moderate runoff conditions, but during large storms is limited
Generally less expensive construction costs than underground pipes	Requires a large area for highly developed sites with large amounts of impervious area
Conveys peak discharge and slows runoff to surrounding streams and rivers	Has higher median effluent concentrations for some pollutants relative to other BMPs
Minimizes erosion when slopes are less than 4:1	Requires more maintenance than curb and gutter systems and may require irrigation to sustain vegetation during dry months.
May provide runoff volume reduction	Is not effective at reducing peak flow rates for larger storm events

8.6.4.3 Bioswales Design Requirements and Considerations

The procedure for designing a grass swale is outlined below. The design components are described in the order of construction starting with overall guidelines, site location and soils, and continuing through shape and slope design.

Overall Guidelines

The main criteria to consider in the design of a grass swale are channel capacity in relation to residence time and minimization of erosion (Iowa, 2009):

- Runoff velocity should not exceed 1 ft. per second (fps) during the peak discharge associated with the water quality design rainfall event (Q_{WQ}).
- If the grass swale is receiving concentrated flows, energy dissipation may be required and the swale may need to be held off line for a period of time for the vegetation to establish.
- The total length of the swale should provide at least 3 to 5 minutes residence time, with a minimum length of 100 ft. Smaller swale lengths may be used if swales are used as pretreatment to downstream BMPs.

Site Location and Soils

- Bioswales should be used to treat drainage areas of less than 1 ac..
- The bottom of the channel should be constructed at least three ft. above groundwater to prevent the bottom from remaining moist and prevent contamination of groundwater (Metro Council, 2001).
- In order to provide the best means for plant survival, bioswales should not be constructed in gravelly and coarse sandy soils, unless a planting medium is provided (MARC, 2009). The use of planting medium shall be limited to special applications and preapproved by the City.
- Select vegetation that can withstand relatively high-velocity flows at entrances, and both wet and dry periods (MARC, 2009, Metro Council, 2001). Vegetation should achieve a minimum 70-percent density prior to putting the swale into service.
- Soil stabilization methods such as mulch, blankets, or mats should be used prior to the establishment of vegetation (MARC, 2009).

Shape and Slope

- It is recommended that swales be designed on longitudinal slopes of 1- to 2-percent. Channel slopes greater than 4-percent should not be permitted (Iowa, 2009). Installation of check dams is recommended for slopes above 2-percent. If the natural slope is greater than 4-percent, longitudinal slope terracing can be used to reduce the longitudinal slope to meet the design standard. Figure 8-20 demonstrates longitudinal slope terracing.

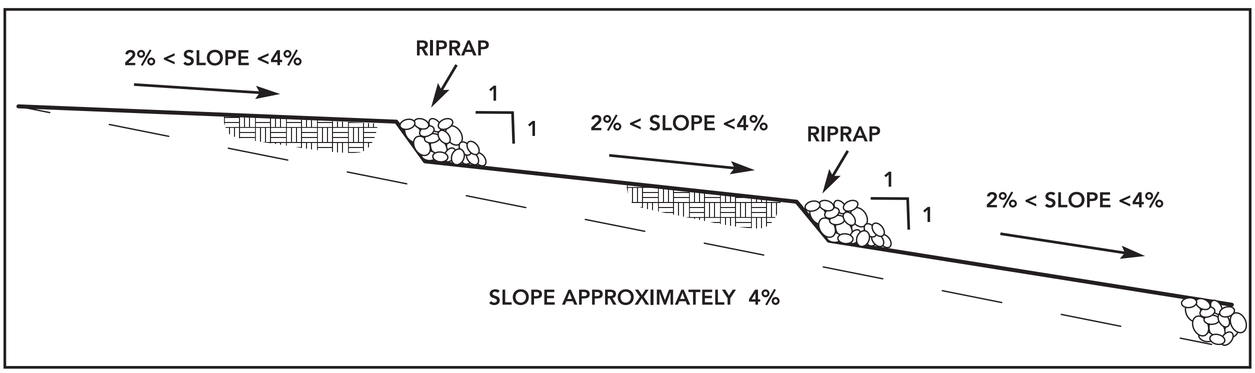

Figure 8-20 Longitudinal Slope Terracing

1. The side slopes of the channel should be as flat as possible to aid in filtration of incoming flows. A maximum slope of 3:1 is recommended; a 4:1 slope is encouraged where space permits (MARC, 2009).

2. Swales should be parabolic or trapezoidal in shape (Iowa, 2009; MARC, 2009; Metro Council, 2001). Figure 8-21 provides a cross section of a trapezoidal swale. The trapezoidal shape is the easiest to construct and is a more efficient hydraulic configuration. The criteria presented in this section assume a trapezoidal cross-section; the same de sign principles will govern parabolic cross-sections, except for the cross-sectional geometry (Iowa, 2009).

3. Size the bottom width between two and eight ft. Larger bottom widths may be used if separated by a dividing berm.

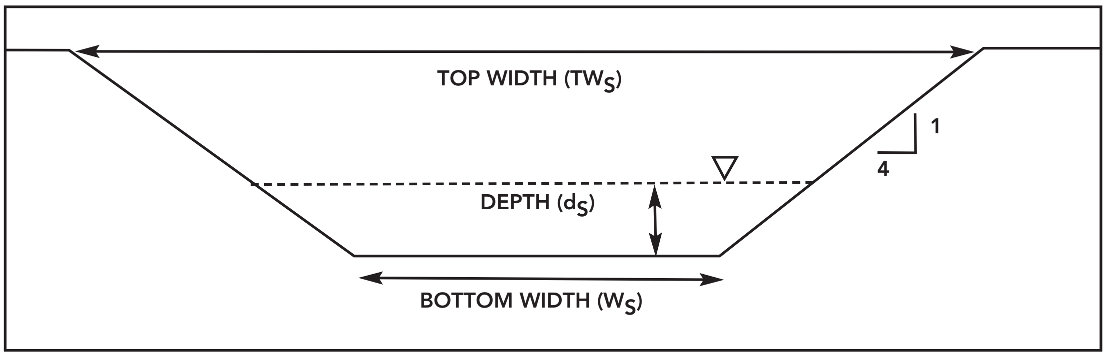

Figure 8-21 Trapezoidal Cross Section with 4:1 Side Slopes

1. Generally, swale length is a function of site drainage constraints (Iowa, 2009). The minimum longitudinal length of a grass swale should be 100 ft. to provide a 3-5 minute residence time (MARC, 2009). Swales less than 100 ft. can be used as pretreatment for downstream BMPs.

2. Identify the swale bottom width, depth, length and slope necessary to convey the water quality flow rate with a shallow ponding depth of 1 to 4 in. The depth should be half the height of the vegetation used in the swale or lower, as increased water depth would provide conveyance rather than residency time needed for the water quality improvement.

3. The Manning's roughness coefficient used to calculate width, depth and length of the swale for the water quality event should be based on sheet flow. If additional capacity is required in the swale for the conveyance of a defined design event (e.g. 10-year storm event), the Manning's roughness coefficient should be modified based on shallow concentrated flow. Table 8-14 lists typical Manning's roughness coefficients for sheet flows.

Table 8-14
Different Vegetation Typical Manning's Roughness Coefficients

Vegetation Type	Manning's Roughness Coefficient "n" for Sheet Flow
Short grass prairie	0.015
Dense grasses (weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures)	0.024
Bermuda grass	0.041

8.6.4.4 Bioswales Inspection and Maintenance

Short Term: Year 1 – Year 3

1. Water young vegetation weekly, at a minimum, for the first three months. Watering may be required biweekly during the drier summer months (June through August) the first year.
2. Eliminate weeds as soon as possible using spot application of herbicide.
3. After rainfall equaling or exceeding 0.5 in. (until second year growth is established):
   a. Remove trash and inspect vegetation.
   b. If sediment has accumulated, remove it and replace mulch and vegetation as needed.
   c. Check for erosion inside and around the grass swale. Repair erosion damage if it occurs.
   d. Repair or restore clogged inflow structures as needed.
4. Between two and three years after installation determine if water is flowing as planned. If there are drainage problems determine cause and address the issues as soon as possible.
5. Beginning one year after installation, inspect vegetation. Replace vegetation as necessary.

Long Term: Year 3 - later

1. In early spring, mow or trim vegetation to a height half the design flow depth. Remove accumulated debris.
2. Inspect vegetation one to two times each year and remove weeds.
3. Trim back or remove overgrown vegetation.
4. Repair or restore inflow structures or ditch checks as needed.
5. In fall, perform inspection annually to assess vegetation condition.

8.6.4.5 Bioswales Submittal Requirements

For review purposes prior to construction, the following minimum submittal requirements are recommended:

1. Drainage area map, including drainage area to swale.
2. Existing and proposed contour map of site (1-ft. contours recommended). Compaction requirements should be stated, if required. Additional spot elevations may be helpful.
3. Geotechnical investigation of site (soil borings, water table location).
4. Swale detail with profile and cross section.
5. Swale calculations, including Q_{WQ}, depth of Q_{WQ} in swale, and maximum velocity for Q_{WQ} (refer to Design Example). A visual representation of the cross-section of the swale to be constructed should be provided, including bottom width and side slopes.

6. Vegetation plan with schedule for installation and initial maintenance. Appropriate inlet treatments and erosion control measures should be included.

7. An as-built survey of the swale is recommended to confirm construction adheres to approved construction plans.

8. Long-term inspection/maintenance plan.

8.6.4.6 Bioswales Design Calculations

Step 1 Calculate the swale's tributary area water quality discharge flow (Q_{WQ}). Calculate the swale's tributary area Q_{WQ} for using the flow rate calculation provided in Section 8.3.2.

Step 2 Solve Manning's equation (Equation 8-18) for swale bottom width.

$$Q_{WQ} = \left[1.49 * A * R_H^{\frac{2}{3}} * S_L^{\frac{1}{2}}\right] \div n \qquad (8\text{-}18)$$

Where:

Q_{WQ} = Peak design flow rate (cfs)
A = Cross sectional area for trapezoidal cross section (ft^2) = $(d_S * w_S) + 4d_S^2$
n = Manning's n-coefficient
P_W = Wetted perimeter for trapezoidal cross-section (ft) = $w_S + 2(d_S + 4d_S^2)^{0.5}$
R_H = Hydraulic radius (ft) = A/P_W
S_L = Longitudinal slope (ft/ft)

An easy method to accomplish this step is by using a spreadsheet or solver program, for example Microsoft Excel solver (an Excel Add-In) can be used to calculate bottom width of the swale (w_S). In this example, the Manning's value, depth of water and longitudinal slope are entered as predetermined values (grey cells) based upon the design criteria in Section 8.6.4.3 and then the solver function is performed on the iterated width cell (yellow cell). When the solver function is performed Excel calculates the Area, Wetted Perimeter and Hydraulic Radius variables based on predetermined formulas.

Example Spreadsheet Setup for Calculating Swale Width Using a Trapezoidal Cross-Section with 4:1 Side Slopes

n	Depth (d_s) (ft)	Iterated Width (w_s) (ft)	Area (A) (sq ft)[1]	Wetted Perimeter (P_W) (ft)[2]	Hydraulic Radius (R_H) (ft)[3]	Longitudinal Slope (S_L) (ft/ft)	Q_{WQ}
0.24	0.17	8.24	1.48	9.61	0.15	0.020	0.375

[1] Area calculation formula is $A = (d_S * w_S) + 4 * d_S^2$
[2] Wetted Perimeter formula is $P_W = w_S + 2*(d_S^2 + (4 * d_S)^2)^{0.5}$
[3] Hydraulic Radius formula is $R = A/P_W$

Step 3 Solve for velocity. Solve for velocity ($v = Q_{WQ}/A$) using the calculated Area variable and Q_{WQ} result from Step 1. If the velocity is greater than 1 fps, the width of the channel or the longitudinal slope may need to be adjusted to obtain a velocity less than 1 fps , and therefore appropriate for shallow flow. (Iowa Stormwater Design Manual, http://www.iowadnr.gov/Portals/idnr/uploads/water/stormwater/manual/part2i.pdf)

Step 4 Calculate minimum swale length. Calculate minimum swale length for required residency time using $L_S = v * T$ where T is equal to minimum residency time between 3 and 5 minutes. If the length calculated is less than 100 ft., a minimum length of 100 ft. must be specified on construction plans unless the swale is used for pretreatment upstream of another BMP.

Note that a swale may also be designed for conveyance of a defined design storm (e.g. 10-year storm event). The calculations presented in this manual are only applicable to design of a swale for water quality improvement. Additional calculations will be necessary to size the swale for other larger events. See Chapter 5: Open Channels for details on designing swales for conveyance of larger storms.

8.6.4.7 Bioswales Example

A 4-ac. site is being developed by a church. Of the 4-ac. site, 0.25 ac. will be tributary to a proposed buffalo grass swale with a Manning's n value of 0.24 and side slopes at 4:1. Assume a time of concentration of 10 minutes to the swale. The buffalo grass is expected to be maintained at a minimum height of 6 in. Proposed longitudinal slope is 2.0-percent. The minimum residency time for the stormwater in this swale is 5 minutes.

Step 1 Calculate the water quality runoff rate Q_{WQ} using guidance in Section 8.3.2.

$$Q_{WQ} = \frac{1.5cfs}{ac} * 0.25\ ac = 0.375\ cfs$$

Step 2 Using Manning's equation to solve for the swale bottom width.

Using Microsoft Excel solver, a bottom width of 3.91 ft. was calculated using Manning's equation based on the Q_{WQ}. In order to keep the bottom width of the channel under the recommended maximum of 8 ft., the flow depth in the swale is designated at 3 in. for the water quality event. The 3-in. depth is based upon three design constraints: first that the vegetation is maintained at a minimum of 6 in.; second that the depth of the water should be half the height of the vegetation in the swale; and third that the depth of the water should not exceed 4 in. to avoid erosion issues.

Grass Swale Example: Calculating Swale Width Using a Trapezoidal Cross-Section with 4:1 Side Slopes

n	Depth (d_s) (ft)	Iterated Width (w_s) (ft)	Area (A) (sq ft)[1]	Wetted Perimeter (P_W) (ft)[2]	Hydraulic Radius (R_H) (ft)[3]	Longitudinal Slope (S_L) (ft/ft)	Q_{WQ}
0.24	0.25	3.91	1.23	5.97	0.21	0.020	0.375

In order to provide a constructible size the bottom width was rounded to 4.0 ft.

Trapezoidal (4:1) Example Problem (Rounded Width Results)

n	Depth (d_s) (ft)	Iterated Width (w_s) (ft)	Area (A) (sq ft)[1]	Wetted Perimeter (P_W) (ft)[2]	Hydraulic Radius (R_H) (ft)[3]	Longitudinal Slope (S_L) (ft/ft)	Q_{WQ}
0.24	0.25	4	1.25	6.06	0.21	0.020	0.383

Step 3 Calculate velocity for the Q_{WQ}. The velocity is calculated at 0.30 fps (v = (0.375 cfs)/(1.25 sq ft) = 0.30 fps). This is less than 1 fps, and therefore meets the recommendations for the Water Quality Storm.

Step 4 Calculate the minimum length of the swale based on residence time. The minimum length of the swale is calculated at 90 ft. (L_S= (0.30 fps)*(5 min)*(60sec/min) = 90.0 ft.). This is less than the minimum recommended length of 100 ft., therefore L_S = 100 ft.

The grass swale should be constructed with a length of 100 ft. and with a bottom width of 4.0 ft. Table 8-15 lists all of the design parameters for this grass swale.

Table 8-15
Example Grass Swale Design Parameters

Design Item	Design Parameter
Drainage Area	0.25 acre
Channel Length	100 Feet
Channel Bottom Width	4 Feet
Side Slope	4:1
Longitudinal Slope	2.0 Percent
Flow Depth	3 Inches
Maximum Velocity	0.30 fps
Vegetation	Turf Grass
Vegetation Maintenance Height	6 inches minimum

8.6.4.8 Filter Strips General Application

A filter strip can be used to improve runoff quality by filtering stormwater runoff through dense vegetation. In rural settings, filter strips are most often utilized as an agricultural BMP to filter runoff from farm fields. In urban settings, filter strips are best utilized in treating runoff from roads and highways, roof downspouts, and small parking lots (EPA, 2006b). Filter strips are frequently used as a pretreatment system for stormwater upstream of other BMPs such as an infiltration trenches or bioretention systems as shown in Figure 8-22 (Metro Council, 2001). See Section 8.2.3 for information on BMPs in treatment trains.

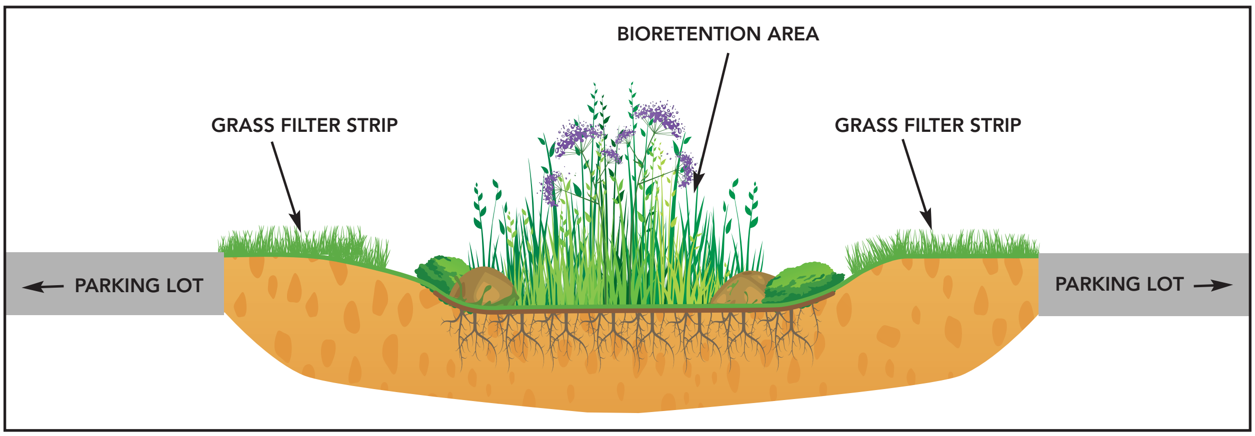

Figure 8-22 Grass Filter Strip Used for Pretreatment

8.6.4.9 Filter Strips Advantages and Disadvantages

Advantages	Disadvantages
Relatively easy and inexpensive to implement	Most effective when implemented with other BMPs (treatment train)
When implemented early in the development process it can be used as erosion and sediment control	Applications of fertilizers, herbicides, and pesticides on filter strip may be a source of pollutants in runoff
Substantial capture of sediment and pollutants that are adsorbed onto particles	Potential failure when concentrated flows with erosive velocities develop and "short circuit" the filter strip
May provide runoff volume reduction	

8.6.4.10 Filter Strips Design Requirements and Considerations

The procedure for designing filter strips is outlined below. The design components are described in the order of construction starting with general guidelines, site location and soils, slope, and shape. Figure 8-23 demonstrates a profile of a grass filter strip.

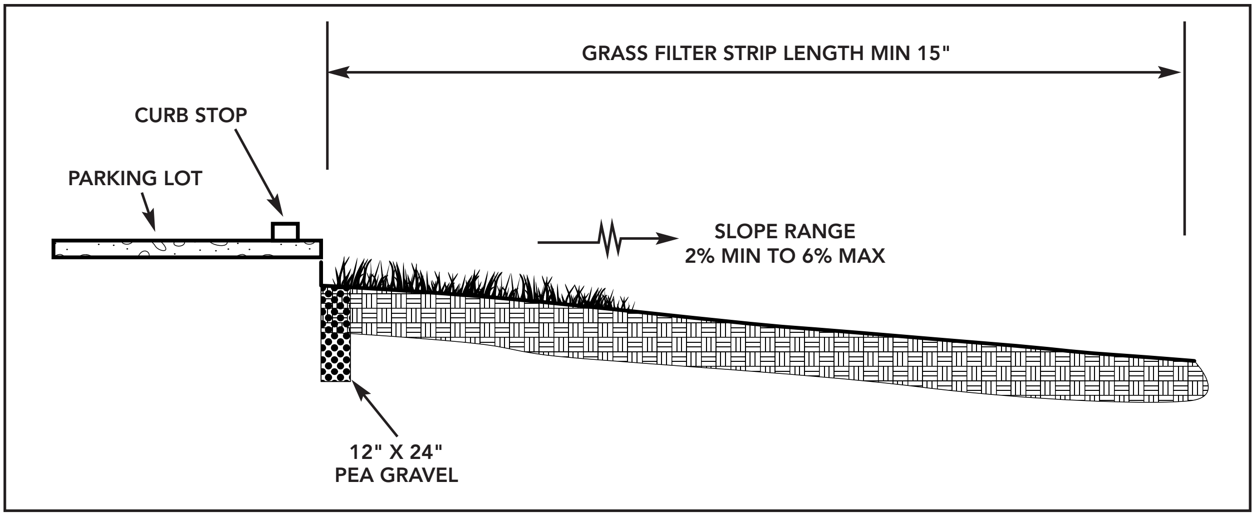

Figure 8-23 Grass Filter Strip Profile (Source Claytor and Schueler, 1996)

General Guidelines

1. Filter strips should be designed to accept sheet flow runoff from small drainage areas (1 to 2 ac. or less). Concentrated flows must be redistributed or dispersed prior to entering the filter strip (Metro Council, 2001).
2. Level spreaders must be surveyed in to avoid low spots.
3. Where applicable, vegetated filter strips should be used as a pre-treatment component for other BMPs such as bioretention areas.
4. Sheet flow runoff from paved surfaces should be limited to maximum lengths shown in Table 8-16.

Table 8-16
Maximum Pavement Length in Ft. (n=0.011) Allowable for a Given Pavement Slope

Drainage Area	10 Year Rainfall Intensity (in/hr)[1]											
Slopes (%)	3.0	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	8.5
0.5	109	93	81	73	65	59	55	50	46	44	40	38
1.0	91	78	68	61	55	50	46	42	39	37	34	32
1.5	67	58	50	45	40	37	34	31	29	27	25	24
2.0	54	47	41	36	33	30	27	25	24	22	21	19
2.5	46	39	35	31	28	25	23	21	20	19	18	17
3.0	40	34	30	27	24	22	20	19	17	16	15	14
3.5	36	31	27	24	22	20	18	17	16	15	14	13
4.0	32	28	24	22	20	18	16	15	14	13	12	12

[1] The 10-year return frequency rainfall intensity should be used for a duration equal to the time of concentration for the pavement area

5. Filter strips constructed in parking lots require special design attention to the spacing of parking blocks in order to maintain sheet flow. In these cases, the designer should specify spacing between individual parking blocks as well as spacing between parking blocks and the beginning of the filter strip. A typical parking space width ranges from 8 to 10 ft., with typical parking block widths ranging from 6 to 8 ft. Parking blocks should be spaced to allow a minimum of 2 ft. width between them. Where parking blocks are used, a minimum additional 2 ft. of surface beyond the parking block is recommended for flow to disperse prior to entering the filter strip. The additional surface required will vary based on the parking lot slope toward the filter strip.

6. Curbs and curb cuts are not recommended adjacent to a filter strip as these features tend to concentrate flow.

Site Location and Soils

1. Filter strips should be positioned at least two ft. above the water table to prevent contamination (Muthukrishnan et al., 2006).

2. Filter strips should be located in an area where they will not remain wet between storms.

3. Filter strips should not be used on soils that cannot sustain a dense grass cover with high retardance (Iowa, 2009).

4. Designers should choose grasses that can withstand relatively high flow velocities, and both wet and dry periods. Use of vegetation appropriate for the local climate is essential for plant survival.

5. Allow vegetation used in the filter strip to reach a 70-percent density of the ground cover prior to making it part of the site's stormwater management program.

Slope

1. Filter strip slopes should be designed no less than 1 percent, but not greater than 6 percent. Greater slopes would encourage the formation of concentrated flow, and lesser slopes may result in standing water. An effective flow spreader is to use a pea gravel diaphragm (small trench) at the top of the slope as demonstrated in Figure 8-23. This trench will act as a pretreatment device and as a level spreader (Iowa, 2009).

2. Both the top and the toe of the slope should be as flat as possible to encourage sheet flow and prevent erosion (Muthukrishnan et al., 2006).

Shape

3. The maximum length of pavement in the direction of flow draining to a filter strip can be determined using pavement slope and rainfall intensity for the 10-year storm. The 10-year rainfall intensity can be determined using Figure 2-2 from Chapter 2. Refer to Table 8-16 for guidelines in determining maximum pavement length.

4. Filter strip length in the direction of flow should be determined based on the slope of the filter strip and 90-percent rainfall intensity for the time of concentration for the drainage area to the filter strip. The 90-percent rainfall intensity can be determined using Figure 8-24. Refer to Table 8-17 for guidelines in determining filter strip length.

Table 8-17
Minimum Filter Strip Length (n=0.24) for a Minimum Travel Time = 3 Minutes

Slopes (%)	90-Percent Rainfall Intensity (in/hr)*																	
	1.7	1.8	1.9	2	2.1	2.2	2.3	2.4	2.5	2.6	2.7	2.8	2.9	3	3.1	3.2	3.3	3.4
0.5	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	46
1.0	42	43	45	46	48	49	51	52	54	55	57	58	59	61	62	63	65	65
1.5	51	53	55	57	59	60	62	64	66	68	69	71	73	74	76	77	79	80
2.0	59	61	63	65	68	70	72	74	76	78	80	82	84	86	87	89	91	92
2.5	66	68	71	73	76	78	80	82	85	87	89	91	94	96	98	100	102	103
3.0	72	75	77	80	83	85	88	90	93	95	98	100	102	105	107	109	112	113
3.5	78	81	84	86	89	92	95	97	100	103	105	108	111	113	116	118	120	122
4.0	83	86	89	92	95	98	101	104	107	110	113	115	118	121	123	126	126	130
4.5	88	91	95	98	101	104	107	110	113	116	119	122	125	126	131	134	136	138
5.0	93	96	100	103	107	110	113	116	120	123	126	126	132	135	138	141	144	146
5.5	97	101	104	108	112	115	119	122	125	126	132	135	138	142	145	148	151	153
6.0	101	105	109	113	117	120	124	126	131	134	138	141	144	148	151	154	157	160

* 9-percent rainfall intensity should be used with a duration equal to the time of concentration for the drainage area to the filter strip.

- The filter strip should stretch the entire width of the adjoining impervious surface where the stormwater originates (Muthukrishnan et al., 2006).
- Filter strips must be a minimum of 15 ft. in length in the direction of flow to effectively treat runoff; greater lengths will enhance treatment (Iowa, 2009).

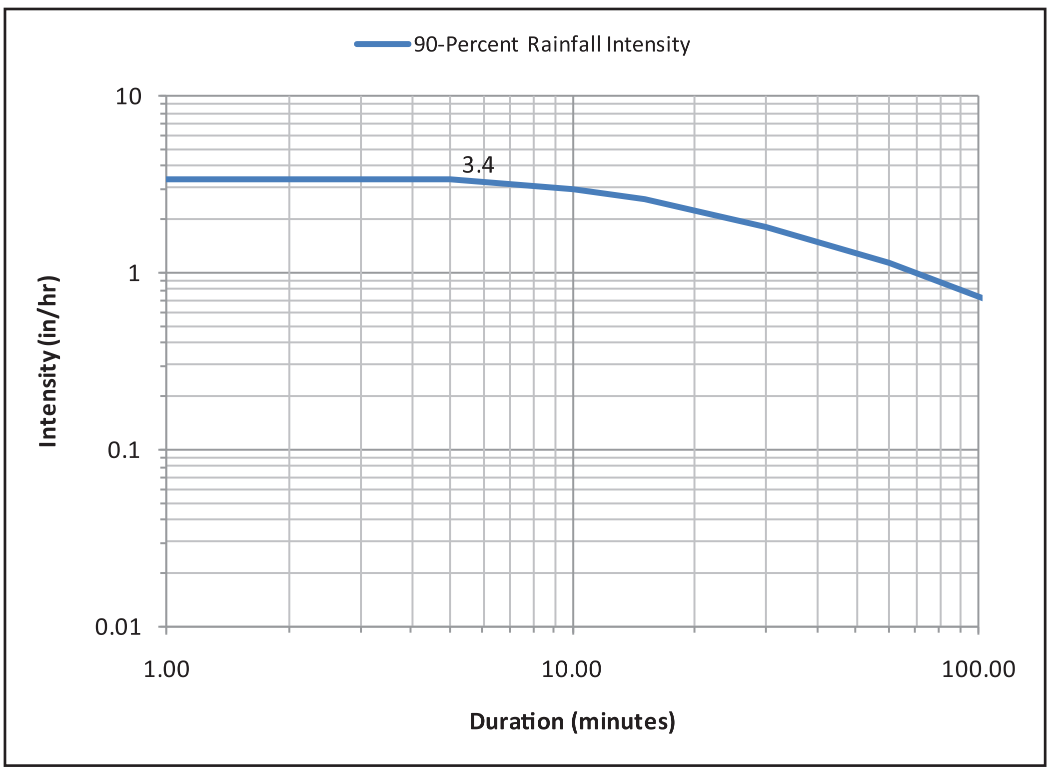

Figure 8-24 Intensity-Duration-Frequency Curve for the 90-Percent Rainfall

8.6.4.11 Filter Strips Inspection and Maintenance

Short Term: Year 1 – Year 3

1. Water young vegetation weekly, at a minimum, for the first three months. Watering may be required biweekly during the drier summer months (June through August) the first year.

2. Eliminate weeds as soon as possible using spot application of herbicide.

3. After rainfall equaling or exceeding 0.5 in. (until second year growth is established):

    a. Redistribute mulch, remove trash, and inspect vegetation.

    b. If sediment has accumulated, remove it and replace mulch and vegetation as needed.

    c. Check for erosion inside and around the filter strip. Repair erosion damage if it occurs.

    d. Repair or restore clogged flow structures as needed.

4. Between two and three years after installation determine if water is draining as planned. If there are drainage problems determine cause and address the issues as soon as possible.

5. Beginning one year after installation, inspect vegetation. Replace vegetation as necessary.

Long Term: Year 3 - later

1. In early spring, mow or trim vegetation to a height greater than 6 in. Remove accumulated debris.

2. Inspect vegetation one to two times each year and remove weeds and invasive species.

3. Trim back or remove overgrown vegetation.

4. Repair or restore clogged flow structures as needed.

5. In fall, perform inspection annually to assess vegetation condition, replace vegetation as necessary.

8.6.4.12 Filter Strip Submittal Requirements

For review purposes prior to construction, the following minimum submittal requirements are recommended:

1. Drainage area map, including drainage area to filter strip.

2. Existing and proposed contour map of site (1-ft. contours recommended). Compaction requirements should be stated, if required. Additional spot elevations may be helpful.

3. Geotechnical investigation of site (soil borings, water table location).

4. Strip detail with slope.

5. Strip design calculations, including Q_{WQ} and overland flow length of upstream drainage area.

6. Vegetation plan with schedule for installation and initial maintenance. Appropriate inlet treatments and erosion control measures should be included.

7. An as-built survey of the strip is recommended to confirm construction adheres to approved construction plans.

8. Long-term inspection/maintenance plan.

8.6.4.13 Filter Strips Design Calculations

Step 1 Calculate the time of concentration of the area draining to the filter strip using Equation 8-19.

$$T_I = \frac{1.8(1.1-C)D^{1/2}}{S^{1/3}} \qquad (8\text{-}19)$$

Where:

T_I	=	Time of concentration to the most upstream inlet or entry point (min)
C	=	Overland flow runoff coefficient for cover type
D	=	Overland flow distance parallel to slope (ft.); 100 ft. should be the maximum distance for overland flow
S	=	Slope of overland flow path (%)

Step 2 Find the 10-year rainfall intensity at the duration equal to the time of concentration using Figure 2-2 from Chapter 2.

Step 3 Use Table 8-16 to find the maximum pavement length (PL_{max}) that can drain to the filter strip, based on rainfall intensity from Step 2 and the proposed slope of the drainage area to the filter strip. Revise the proposed length and area draining to the filter strip if the proposed filter strip length does not fit within the site area while meeting all setback requirements. If the filter strip length is reduced the area draining to the filter strip needs to be reduced to a size that does not exceed the filter strip length drainage capacity.

Step 4 Find the 90-percent rainfall intensity at the duration equal to the time of concentration, using the time of concentration calculated in Step 1, using Figure 2-25.

Step 5 Use Table 8-17 to find the minimum filter strip length required based on the 90-percent rainfall intensity from Step 4 and the proposed slope of the filter strip area in the direction of flow. Compare to the site plan to verify that the filter strip length will fit within the site area while meeting all setback requirements. If the proposed length of the filter strip is less than 15 ft., increase the proposed length to 15 ft. to meet the minimum length requirement.

8.6.4.14 Filter Strips Example

A one-ac. site is being developed by a small business (C=0.50) as shown on Figure 8-25. Approximately 0.20 ac. of the parking lot with no parking blocks will be tributary to a proposed filter strip. The slope of the parking lot is proposed to be 1.0-percent, and the slope of the proposed filter strip is 2-percent. Find the length of the filter strip.

Time of Concentration (T_I)

T_I is found using Equation 8-19.

$$\frac{T_I = (1.8*(1.1-0.5)*36^{\frac{1}{2}})}{(1.0)^{\frac{1}{3}}} = 6.5 \text{ minutes}$$

Ten Year Rainfall Intensity (I_{10})

I_{10} can be found using Figure 2-2 from Chapter 2. I_{10} for a T_I of 6.5 minutes is 8 in. per hr..

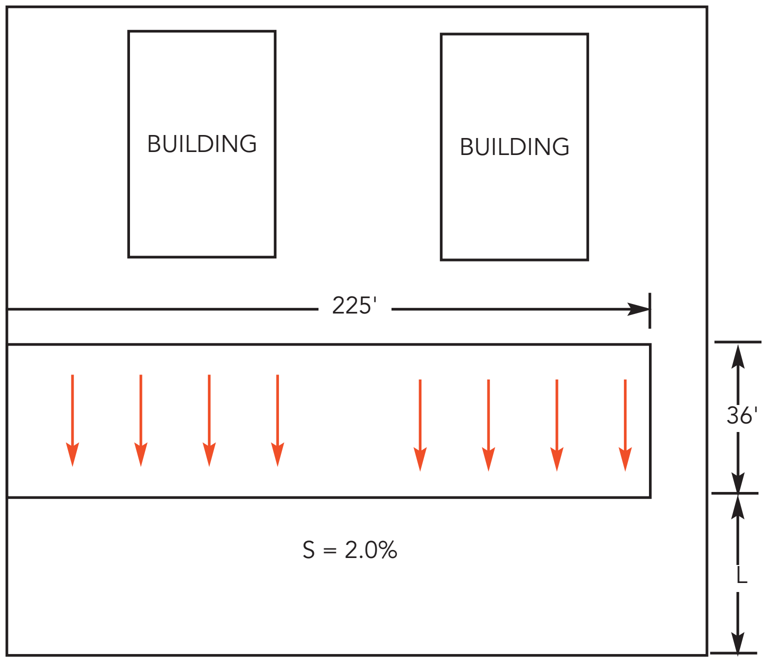

Figure 8-25 Site Plan of 1 Acre Small Business Site

Maximum Pavement Length (PL_{MAX})

PL_{MAX} can be found using Table 8-16 and finding the maximum pavement length for a drainage area slope of 1.0-percent and a rainfall intensity of 8.0 in. per hr. For this example, PL_{MAX} is 34 ft. Since proposed pavement length of 36 ft. is greater than PL_{MAX}, the site plan must be changed to incorporate at maximum pavement length of 34 ft.

Water Quality Event Intensity (I_{WQ})

I_{WQ} can be found using the curve for the 90-percent event (Figure 8-24). Using the duration of 5 minutes and the 90-percent plot, the I_{WQ} is 3.1 in. per hr.

Minimum Filter Length (FS_{MIN})

Use Table 8-16 to find the minimum length for a filter strip slope of 1-percent and a rainfall intensity of 3.1 in. per hr. For this example, FS_{MIN} is 62 ft. This length would need to be compared to the available area on the property for the filter strip. For this example the available area is 60 ft.

8.6.4.15 References

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8.6.5 Green Roof

A Green Roof consists of a series of layers that create an environment suitable for plant growth without damaging the underlying roof system. Rainfall is initially intercepted by vegetation installed on the roof, held on foliage or soaked up by plant roots. Runoff that exceeds the holding capacity of the growing medium is released from the rooftop through an underdrain system. A Green Roof can be installed on either a new or existing rooftop, provided the roof structure is able to hold the additional weight and meet local building code. Green Roofs are not new technology, with many of their known benefits only being added to in recent years. The technology provides an insulation benefit to the installed building, decreasing rooftop temperatures, mitigates urban heat island effects, and can provide stormwater quantity and quality benefits making them a valuable structural BMP alternative in urban settings where land area is limited. In addition, they may provide an aesthetic benefit helping a building to meet potential landscaping requirements, and can be designed to be used by pedestrians and/or building occupants. As the variations in Green Roof vary widely, the focus of this section will be to describe the key components of a Green Roof system. Each system will need to be designed to fit the structure and purpose for the respective building. However, there are common design elements and considerations that are instrumental to the success of each Green Roof system.

Design Considerations	
Location characteristics (Slope, Soil Type)	Slope: Extensive Systems 2% to 25% Intensive Systems 2% to 10% Soil Type: Engineered Growing Medium/ Soil
Contributing drainage area	Vegetated Rooftop Area
Design size	Vegetated Rooftop Area
Detention time for WQCV treatment	Soil matrix designed to store WQCV
Median Effluent Concentrations[3]	TN = 1.08 mg/L[1]; TP = 0.15 mg/L[1,2]
Implementation and Maintenance Considerations	
Potential for use with other BMPs	High; Green Roofs can be used as the first structural BMP in a treatment train
Maintenance	Low – Vegetation maintenance, Inspection of drainage outlets and waterproofing layer

[1] NC State University/ NC State University Bio & Ag Engineering and North Carolina Department of Environment and Natural Resources (NCDENR), 2011
[2] Water Environment Research Foundation (WERF), 2005 for Media Filters
[3] The effluent concentration from a Green Roof system is highly dependent on the characteristics of the unique soil matrix for each application combined with any contamination through atmostpheric deposition. A Green Roof only treats stormwater that falls directly on the Green Roof surface itself.

8.6.5.1 General Application

Green Roof systems are a good BMP for use in urban areas, as they can be designed and/or retrofit in the defined building footprint. Green Roofs provide several benefits for the building. Through the vegetation and engineered growing medium, a Green Roof provides an insulating mechanism. This can provide energy savings, as well as insulation from typical urban environment sounds, such as building system equipment installed on rooftops. The vegetation assists in insulating a building through the evapotranspiration process. The vegetation and growing medium also treats and reduces the volume of stormwater runoff that is generated by the roof area on which the Green Roof is installed.

Because stormwater is stored by a Green Roof and released over a period of time, stormwater flow peaks typically seen following high intensity rainfall events are reduced and delayed over a period of time (Dunnet and Kingsbury, 2004). This allows downstream drainage systems to convey “moderately increased flow” over an extended time period instead of increased peak flow over a relatively short period of time, characteristic of highly impervious areas. If a Green Roof is constructed within the drainage area of a downstream BMP, the area of the Green Roof can be subtracted from the WQCV calculation for the BMP. Because the rainfall is not infiltrated, a Green Roof acts as a filter and increases the time it takes for rainfall to reach the primary storm sewer system. In the case of a Green Roof with a slope of less than 25-percent, the result of stormwater filtering through a Green Roof is approximately a 45-percent reduction in the volume of runoff (Forschungsgesellschaft Landschaftsentwicklung Landschaftsbau [FLL], 2002).

As Green Roofs capture and filter rainfall falling immediately on the surface of the roof, their water quality benefits are limited to that immediate rainfall. Additional stormwater flow should not be routed to a Green Roof for treatment. Green Roofs have been shown as a poor BMP for nutrient storage and removal, due to the fact that filtration of water through plant material and substrate will yield added nutrient runoff (Minnesota, 2008). Concentrations of nutrients in stormwater runoff from Green Roof systems have shown in studies to be very similar to other vegetated systems that you would find in a typical landscape (EPA, 2009). However, it should be noted that even if concentrations of nutrients are typical to other vegetated systems, the loading of a pollutant is often reduced because the total stormwater runoff from a Green Roof is reduced (EPA, 2009).

Figure 8-26 Saddlebrook School, Library, and Community Center, Omaha, Nebraska (www.omahaice.org)

There are two basic types of Green Roof systems to consider for installation: Extensive Systems and Intensive Systems.

Extensive Systems

An Extensive System consists of a lightweight Green Roof system, with a shallow depth that supports a limited variety of vegetation. These systems use drought tolerant vegetation and can structurally support limited uses (such as maintenance personnel). Figure 8-26 shows an example of an Extensive System.

Figure 8-27 Gallup Campus Green Roof, Omaha, Nebraska (www.omahaice.org)

Intensive Systems

An Intensive System is a heavyweight Green Roof system, having greater soil depths which can support a wider range of plants and increased pedestrian traffic. Figure 8-27 show an example of Extensive Systems in combination with Intensive Systems in a Green Roof application.

8.6.5.2 Advantages and Disadvantages

Advantages	Disadvantages
Treat and reduce the volume of stormwater runoff	Additional roof loads may require an enhanced structural design, limiting the retrofit of existing buildings.
Conserve energy by providing additional insulation, using the evapotranspiration process of plants to cool the roof during the summer, and reducing the heat lost to wind convection during the winter	Leaks can be difficult to locate and repair. Leak detection systems can assist with this process.
Offset urban heat island effects by reducing the amount of heat typically absorbed by a conventional roof and thus lowering the ambient temperature of the roof	Conditions can be harsh for vegetation establishment.
Extend the lifespan of a conventional roof by protecting the roof surface from UV light, large temperature fluctuations, and normal wear and tear associated with exposed surface roofs	Maintenance costs can be higher than for conventional roof systems; however, roof lifecycle costs may be lower.
Improve the aesthetics of a building; provides attractive views from other buildings and/or an opportunity for pedestrian traffic	
Effective sound insulators that can reduce the impact of noise from equipment on the roof or other outside noises	
Plants on Green Roofs use carbon dioxide and produce oxygen	

8.6.5.3 Design Requirements and Considerations

This Section provides a narrative and discussion of typical design requirements and consideration for a Green Roof. Any differences in requirements between Extensive System and Intensive System design and construction are noted. This information is outlined by how it would be presented on construction drawings: Green Roof – Plan View, Green Roof – Cross Section, and Green Roof – Calculations. As the application of a Green Roof will always vary with the building, the focus of this Section is to look at the main function of key components in the Green Roof Section. Each of these key components will need to be designed to meet criteria of a specific application. It is recommended that the designer consult and design to the following minimum industry standards for all Green Roof applications:

- ANSI/SPRI RP-14 Wind Design Standard for Vegetative Roofing Systems.
- ANSI/SPRI VF-1 External Fire Design Standard for Vegetative Roofs.
- ASTM E2397-05 Standard Practice for Determination of Dead Loads and Live Loads associated with Green Roof Systems
- ASTM E2400-06 Standard Guide for Selection, Installation, and Maintenance of Plants for Green Roof Systems

Green Roof – Plan View

At a minimum, the following components should be clearly labeled on the plan view of a Green Roof design project:

1. **Structural Supports.** Placement of the heaviest components of the Green Roof should be on column heads or over beams. A Green Roof retrofit should consider additional loads imposed on the existing roof. A structural support plan should be provided in addition to the plan view of the Green Roof.

2. **Drainage.** The location of underdrains in the gravel ballast should be clearly designated, including diameter of PVC pipe to be installed and perforation requirements. All underdrains should be connected to a roof downspout or means of conveyance away from the Green Roof system. These underdrains serve as overflow points in the Green Roof system. The purpose of these underdrains is to quickly release excess runoff from larger storm events. All downspouts or means of conveyance away from the Green Roof system should be clearly labeled.

3. **Gravel Ballast.** A width of gravel, stone, or paver material along the perimeter of the Green Roof provides several functions. First, this volume of permeable material can provide additional storage capacity of the Green Roof, filtering and conveying flows in excess of the WQCV. Secondly, this width can provide protection from wind shears (Dunnet and Kingsbury, 2004). Thirdly, this ballast can serve as a separation point between roof components, and can provide a fire break point in the roof system. Having this width along a Green Roof's edge can prevent possible vegetation growth into the waterproof layer. The perimeter gravel ballast should be labeled with a width, depth, and gradation for installation. Calculations should be included on the porosity of this material. All stone used in the gravel ballast should be triple-washed.

4. **Vertical Elements.** All vertical elements sited on or penetrating the Green Roof, such as air vents and heating/air conditioning components should be clearly labeled on the plan view. It is ideal to place gravel ballast along the perimeter of all vertical elements on the Green Roof. This allows access to the element that may be required for periodic maintenance.

5. **Slope.** The slope of each area of the Green Roof should be indicated in the plan view, clearly labeled with an accompanying arrow indicating the direction of slope. Slopes exceeding 15-percent will require additional stabilization measures (horizontal strapping, laths, battens, or grids), which should be accounted for in the structural calculations (Dunnet and Kingsbury, 2004). These greater slopes can create a problem of slippage between the materials used in construction of the Green Roof.

6. **Vegetation Plan and Schedule.** The vegetation zones on the Green Roof should be clearly designated and dimensioned on the plan view. A planting plan, with list or schedule of plants and their method of installation should be included on this sheet. For more details on vegetation selection see discussion in the following paragraphs.

Green Roof – Cross Section

Each layer in the Green Roof system should be clearly labeled in cross-section view and reference manufacturer/design specifications. The layers described as follows are the minimum to be installed as part of a functioning Green Roof system. Figure 8-28 shows the typical Green Roof cross section.

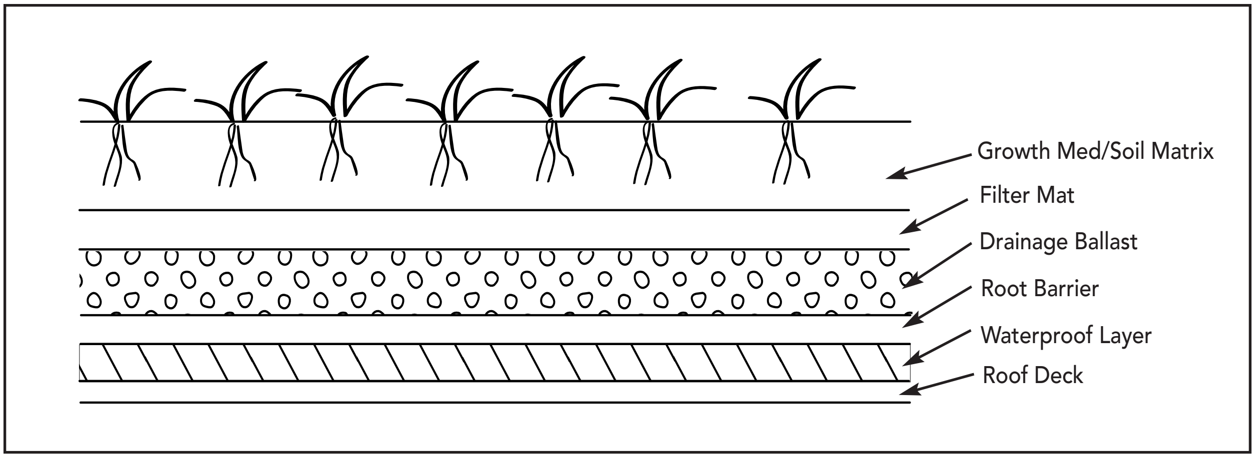

Figure 8-28 Typical Green Roof Cross Section

1. **Waterproof Layer.** An effective, durable waterproof layer is important to the longevity of a Green Roof installation. The waterproof layer protects the building's structural components from being exposed to the elements. Two common types of waterproof applications include tile/sheet single-ply roof membranes or fluid applied membranes. Either should be applied strictly according to the manufacturer's instructions. Tile/sheet membranes are installed by overlapping and sealing joints, therefore it is important to inspect the seal at each joint prior to installing the next layer of the Green Roof system to mitigate potential system leaks and opportunities for vegetation root penetration.

2. **Root Barrier/Waterproof Protection Layer.** A root barrier and/or waterproof protection layer should be installed as the next layer in the Green Roof system. This layer deters root growth into the underlying waterproof layer, in addition to providing protection to the waterproof membrane from anything installed above it during and after construction (City of Eugene, Oregon [Eugene], 2008). This layer is typically a dense material such as a PVC sheet. Sheets are overlapped and sealed together. It is critical to extend this layer beyond the actual planting area, and install around any vertical elements to create a barrier to vegetation root growth in the system. Membranes should not be impregnated with pesticides or with copper, both of which could adversely affect the water quality of any runoff from the Green Roof (Eugene, 2008). It is recommended that any investigation of this layer to assess root penetration follow ANSI/GRHC/SPRI VR-1 2011.

3. **Drainage Ballast.** The drainage ballast should be designed to convey the volume of stormwater runoff that can be absorbed by the growth medium; the purpose of this layer is not to detain stormwater. Unlike other structural BMP systems, excess runoff is conveyed as underflow in the Green Roof system; surface runoff should not occur (Dunnet and Kingsbury, 2004). It is important to remove the excess runoff in order to prevent over saturation of the vegetation. The drainage ballast can be constructed from coarse, washed granular materials, porous mats, or manufactured drainage modules, and connected to the perimeter gravel ballast.

4. **Filter Mat.** A semi-permeable fabric or mat should be installed between the drainage ballast and the growth medium/soil matrix. The purpose of this mat is to filter any small particles that could be transferred from the growth medium/soil matrix through the infiltration of runoff into the drainage ballast; it prevents the drainage ballast from becoming clogged or from potentially

blocking any drainage outlets in the Green Roof system. The filter mat should not only be installed as a horizontal layer, but also vertically at all perimeter edges of the growth medium/soil matrix layer, wrapping up around this layer.

5. **Growth Medium/Soil Matrix.** The growth medium for the Green Roof should be designed to permeate or filter the calculated WQCV. If this is not feasible, additional BMPs on the building site may be required. In addition, the depth and soil matrix of this layer must be designed to support the vegetation to be installed (either an Extensive System or an Intensive System). Soil matrixes for both must be well drained, while at the same time being able to absorb and retain rainfall volume, providing a medium for Green Roof vegetation to thrive. The filtering capacity of the growth medium and the components of the soil matrix should be designed by a landscape architect or other approved vegetation specialist.

Sizing Guidelines

All designs for new or existing Green Roofs must account for the dead and live loads associated with this type of construction in addition to the requirements of the existing building code, as adopted by the City of Omaha. Additional roof loads will require an enhanced structural design and may limit the retrofit of existing buildings. The following calculations are required as part of a Green Roof submittal:

- **Live Loads.** If the Green Roof is accessible, people load must be accounted for. Another example of live load is wind. Each live load must be itemized with appropriate supporting calculations.
- **Dead Loads.** Dead loads would include, but are not limited to, saturated weights of Green Roof materials, snow, and ice. Each dead load must be itemized with appropriate supporting calculations.
- **Building Structure Loading Capacity.** The building's structure capacity must be able to carry the calculated live loads and dead loads.

All submitted calculations must be sealed and signed by a professional structural engineer licensed in the state of Nebraska.

The growth medium or soil matrix has two critical functions: to store the WQCV and to permeate runoff in excess of the WQCV to the ballast layer with little to no ponding on top of the vegetation. As each Green Roof application is unique, the depth of growth medium/soil matrix will be unique to each project. It is recommended that a minimum depth of 2-in. be used as a guideline to detain and filter the WQCV and support vegetation.

Stormwater runoff in excess of the WQCV will need to be conveyed from the Green Roof. This involves using the drainage ballast, the gravel ballast, any overflows in the roof system, and roof downspouts. This conveyance will be unique to each Green Roof application. It is recommended that the designer submit a routing plan for stormwater runoff that includes all major elements, complete with conveyance and volume calculations.

Considerations for Vegetation Selection

A Green Roof requires very different plant material selection when compared to other vegetated structural BMPs (Minnesota, 2008). The vegetation selected must thrive in very constrained growing conditions, and account for seasonal fluctuations in temperature and moisture.

Vegetation should be installed using one of two preferred methods: vegetation mats/modular systems or plugs/potted plants.

- Vegetation mats/modular systems are pregerminated systems that achieve immediate full plant coverage. These systems provide several immediate advantages to the Green Roof installation and establishment process, including a reduction in exposed soil which reduces potential erosion and weed concerns during plant establishment. Long-term maintenance is minimal, and may require intermittent watering during very dry periods and weeding (Eugene, 2008).

- Plugs or potted plants provide can provide more options for Green Roof vegetation. A variety of plant species can be used in one Green Roof installation. Using plugs or potted plants will require initial mulching of the Green Roof, preventing erosion during vegetation establishment. Until establishing a minimum of 90-percent vegetated cover on the roof, subsequent mulching, weeding, and irrigation may be necessary.

The selection of plants to install on a Green Roof is primarily based on the depth of growing medium and the composition of the soil matrix, in addition to climate considerations (rainfall, temperature). For Green Roofs where the growing medium is shallow, xeriscape plantings are commonly used. Xeriscape plantings include a mix of sedum/succulent plant communities that are ideal for installation on a Green Roof due to their drought tolerance, growth patterns, low maintenance needs, resiliency, and fire resistance (Eugene, 2008). For richer, deeper substrates, shrubs and trees may be used. In general, plants with deep root systems typical of an infiltration type structural BMP may not be suitable for a Green Roof.

8.6.5.4 Inspection and Maintenance

Critical inspection points occur during the Green Roof installation process to ensure the integrity of the Green Roof system. In addition, key maintenance activities for Green Roof systems include both short-term and long-term tasks. The following industry standard procedures for investigation are recommended for all Green Roof applications:

- ANSI/GRHC/SPRI VR-1 2011 Procedure for Investigating Resistance to Root Penetration on Vegetative Roofs.

- ASTM E2396-05 Standard Test Method for Saturated Water Permeability of Granular Drainage Media [Falling-Head Method] for Green Roof Systems.

- ASTM E2398-05 Standard Test Method for Water Capture and Media Retention of Geocomposite Drain Layers for Green Roof Systems.

- ASTM E2399-05 Standard Test Method for Maximum Media Density for Dead Load Analysis of Green Roof Systems.

Inspection During Installation

1. Any Green Roof layer applied as sheets, mats, or rolls should be sufficiently overlapped and sealed during the installation process, as directed per the manufacturer instructions.

2. The waterproof layer should be inspected prior to the subsequent installation of Green Roof layers. Green Roof systems may include a pre-installed electronic leak detection system. If this is not included in the design of the installation, it is advised to administer a waterproofing test with an electronic leak detection system (such as vector mapping) prior to installation of subsequent Green Roof layers.

3. It is advised to complete an inspection at the completion of each layer’s installation.

4. The installation of modular products can increase installation efficiency (Minnesota, 2008).

Short Term: Installation – Year 1

1. A goal of 90-percent vegetated coverage should be achieved within 6-months for Green Roofs installed at the beginning of the growing season (Spring).

2. Temporary irrigation may be required in order to establish vegetation. A permanent irrigation system may be needed, depending on vegetation selection in the Green Roof system.

3. Monthly weeding of the Green Roof during Year 1 is recommended to deter weed seedlings and saplings from establishing.

4. All drainage outlets and/or overflows should be inspected after any rain event exceeding 0.5-in. during the first year of installation. The purpose of this inspection is to ensure the flow of excess stormwater from the roof surface. During vegetation establishment, these outlets are more susceptible to clogging.

Long Term: Year 1 – later

1. Biannual weeding of the Green Roof is recommended to deter weed seedlings and saplings from establishing.

2. Conduct annual surveys to verify that the waterproofing system remains watertight below the vegetated cover (Minnesota, 2008).

3. All drainage outlets and/or overflows should be inspected at least two times per year. The purpose of this inspection is to ensure the flow of excess stormwater from the roof surface.

4. After vegetation is well established, it is recommended to fertilize the system only as needed, and not at an interval more frequent than every other year. Fertilization at this interval allows the content of nutrient in the substrate to become exhausted, and can therefore enhance vegetation growth and appearance (Dunnet and Kingsbury, 2004). Attention should be paid to the impact any fertilization will have to the overall water quality of stormwater runoff from the Green Roof system.

8.6.5.5 Submittal Requirements

For review purposes prior to construction, the following minimum submittal requirements are recommended:

1. Green Roof dimensions and setbacks from roof lines. This plan view should also include all components outlined in Section 8.6.5.3, clearly labeled.

2. Profile view of facility, including typical cross-sections and dimensions, with all components outlined in Section 8.6.5.3, clearly labeled.

3. Specifications of all materials to be used in the Green Roof construction, including but not limited to: Growth medium/soil matrix specification, including depth; filter mat specification; drainage ballast specification; root barrier specification; waterproof layer specification.

4. Vegetation plan with schedule for installation and initial maintenance. Appropriate erosion control

measures should be included.

5. Stormwater conveyance and volume calculations.

6. Structural dead load and live load calculations. This should include verification that structure will support additional loads.

7. Long-term inspection and maintenance plan.

8.6.5.6 References

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8.6.6 Manufactured Systems

A manufactured system is generally a structural BMP whose function provides more traditional, treatment via settling and different forms of filtration. Manufactured systems are typically proprietary units; however, some may have their components designed separately by the site designer. These systems are generally designed to fit or work within an existing conveyance system component, and can be ideal for more urban areas where open space is minimal. As manufactured systems vary depending on application and the manufacturer, this Section's guidance will focus on three main categories of manufactured systems and their respective functions: filtering, storage, and separation.

A manufactured system whose primary function is filtration is typically designed to filter, at a minimum, the water quality discharge, Q_{WQ} (Section 8.3.2). Filtration systems typically remove pollutants in stormwater by passing runoff through filter cartridges or filter media.

Detention systems or manufactured systems whose primary purpose is to store stormwater runoff are designed to retain the WQCV. Pollutants settle out in these systems over a period of time.

Manufactured separation devices include two types of systems: chambered and hydrodynamic. In chambered systems, stormwater runoff passes through a series of chambers where pollutant particles settle out. Hydrodynamic systems create a vortex motion to runoff flow which drives separation of pollutant particles for removal.

Design Considerations	
Location characteristics (Slope, Soil Type)	Slope: Variable Soil Type: N/A
Contributing drainage area	< 5 acres[1]
Design size	Variable
Median Effluent Concentrations	Filter System: Variable[2,3], TP = 0.14 mg/L Detention/Storage Systems[3]: Variable, TP = 0.19 mg/L Separators: Variable[2,3], TP = 0.14 mg/L
Implementation and Maintenance Considerations	
Potential for use with other BMPs	High – Manufactured systems may be appropriate for installations in special situations (discussed in Section 8.2.2.5) Filter System or Separators: Pre-treatment in a treatment train application Detention/Storage Systems: Can be used as downstream treatment and storage in a treatment train application
Maintenance	High; Frequent removal of collected material and cleaning/changing of filter media.

[1] Metropolitan St. Louis Sewer District (MSD), 2009
[2] Removal efficiencies are very variable. The use of phosphorus as the target pollutant is recommended when using performance based water quality criteria (Virginia, 1999).
[3] Table 8-3

8.6.6.1 General Application

In general, a manufactured system can be appropriate for small drainage areas of highly impervious cover. Often, these drainage areas have high hydrocarbon or sediment loadings that need to be addressed. Care should be taken to not exceed manufacturer's recommended flow rates or volumes to a system. Manufactured systems are extremely variable in design details, design concept, and pollutant removal characteristics.

A manufactured filter system is designed to filter flow which is regulated by an inflow pipe. The filtered stormwater is then routed to a discharge point. The filters in these manufactured systems may be designed or selected to remove specific pollutants from stormwater. The filter media is typically based on the target pollutants to be removed, and can provide a pre-treatment option for stormwater discharging to another structural BMP as part of a treatment train. Typical applications of manufactured filtration systems include retrofit applications into curb inlets and catch basins. Maintenance of these systems on a schedule similar to erosion and sediment control inspections is critical to the long-term success of these BMPs. An example of a manufactured filter system is shown in Figure 8-29.

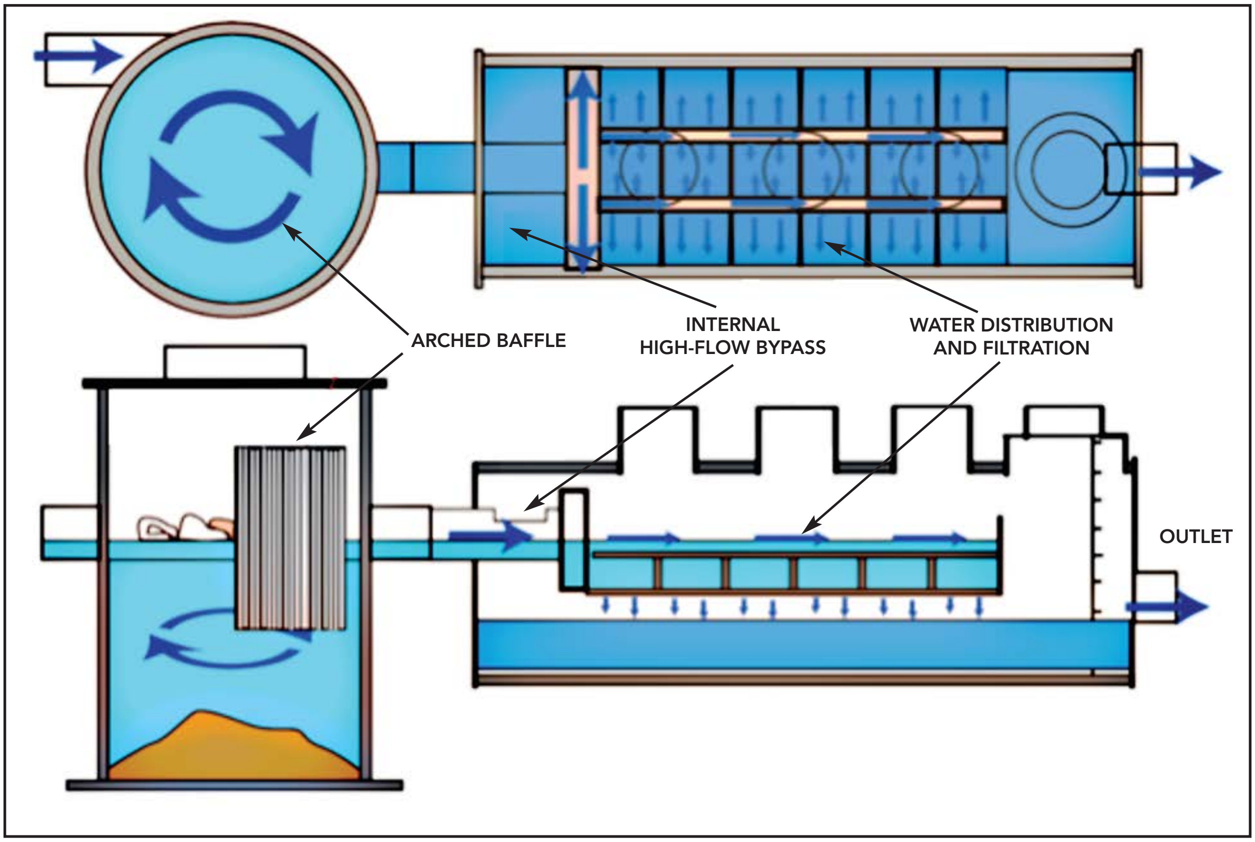

Figure 8-29 Example of Manufactured Filter System

Manufactured storage, or detention, systems can be installed below or above ground, providing the added advantage for a given space to be dual function. An example of this is providing underground detention cells below a parking lot or alley. Pollutants in stormwater runoff conveyed to these systems settle out over time, similar to the function of a typical extended detention structural BMP. Similar to extended detention structural BMP, manufactured storage systems can be used as downstream components in a treatment train if sized accordingly. Regular inspection of the outlet structure and at key points in the system susceptible to clogging is critical for the long-term functioning of this BMP. An example of a manufactured storage system is shown in Figure 8-30.

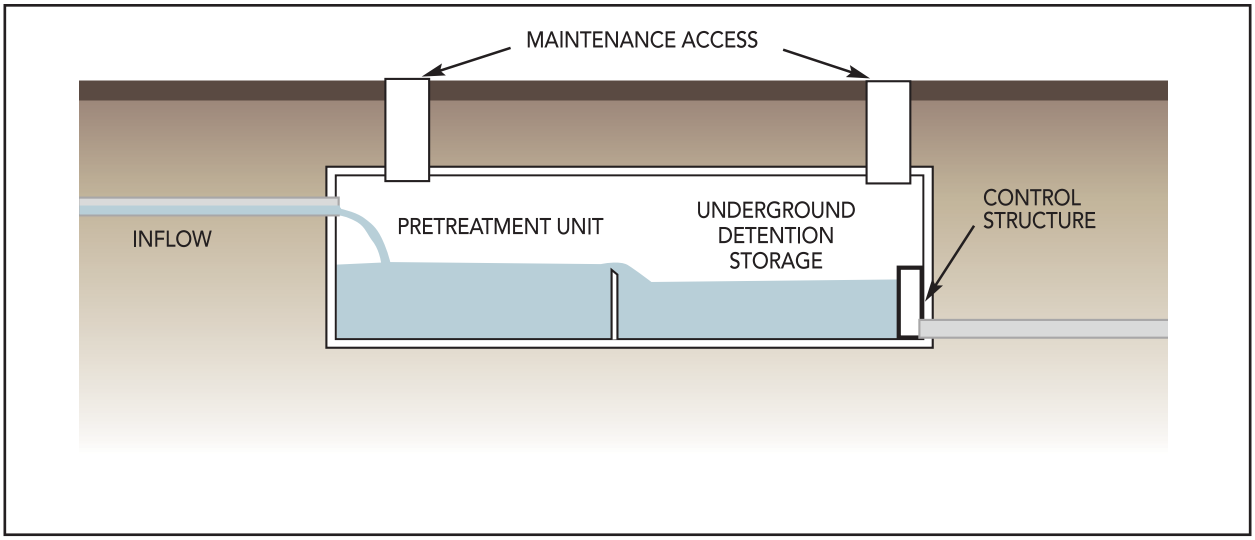

Figure 8-30 Example of Manufactured Storage System

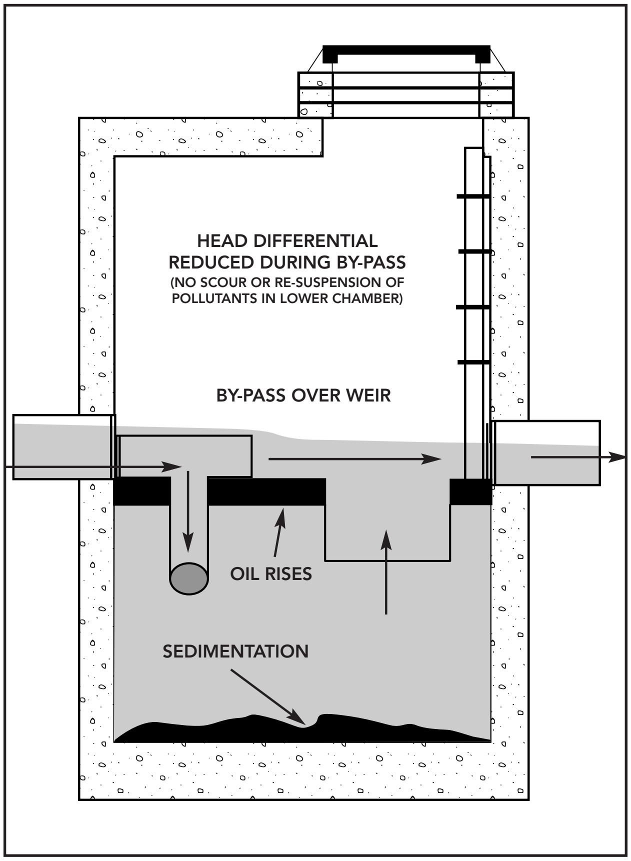

Figure 8-31 Example of Manufactured Separator System, Chamber Configuration (Adapted from Virginia Stormwater Management Handbook, 1999)

Separator systems filter flow via system hydraulics. These systems are typically used to treat stormwater using gravitational settling or circular flow to remove sediment and pollutants. They can provide pre-treatment of stormwater runoff before discharging downstream to either a typical conveyance system, or another structural BMP (as part of a treatment train). Applications of separators can be stand-alone (part of new construction) or can be used in retro-fit situations. Continual maintenance is critical for these systems to continue functioning at a high level of service. An example of a chambered system is shown in Figure 8-31 and an example of a hydrodynamic system is shown in Figure 8-32.

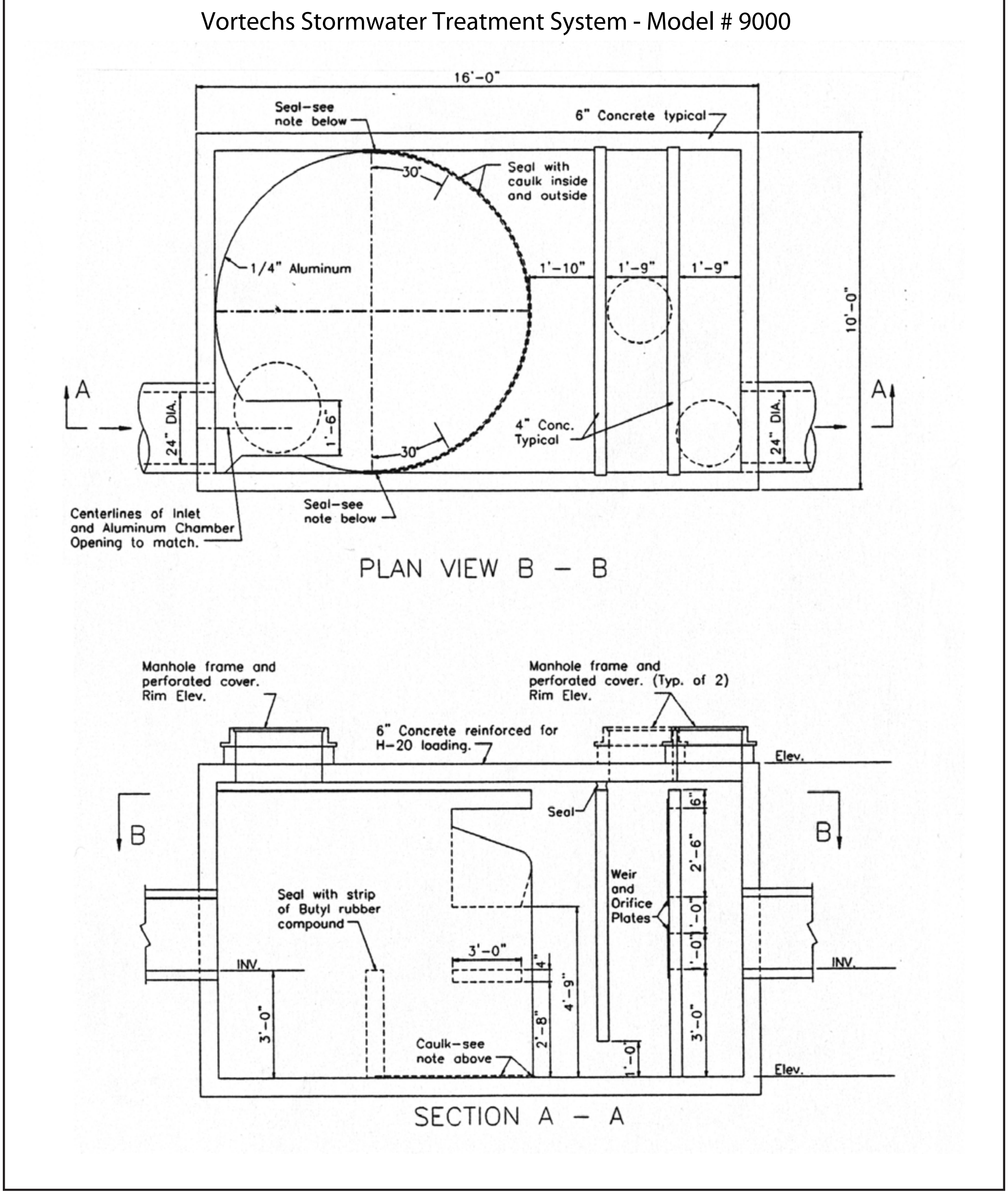

Figure 8-32 Example of Manufactured Separator System, Vortex Configuration (Adapted from Virginia Stormwater Management Handbook, 1999)

8.6.6.2 Advantages and Disadvantages

Advantages	Disadvantages
Manufactured systems can be designed to target removal of specific pollutants.	Design volumes are often limited to smaller drainage areas/runoff volume applications.
Opportunity for dual-use of a respective area (i.e. a parking lot can have an underground storage manufactured system)	Dependent on routine, scheduled maintanence for maximum system function.
Retrofit systems available.	Subsurface installations are not readily inspected, and typically lack provisions to warn of impending failure.
Typically do not require a large footprint for construction; ideal for applications in urban areas with minimal open space.	Little long-term maintenance data available to establish the life-span and long term needs of these systems.
Various options are available from manufacturers to find the “best fit” for an application.	
Quick to install; less room for error during installation	
Manufacturers provide recommended installation instructions and specifications.	
Can be cheaper than traditional technologies for stormwater treatment.	

8.6.6.3 Design Requirements and Considerations

Because each manufactured system varies in function, application, and manufacturer (or vendor), design requirements and considerations are presented in a generalized fashion. Each of these requirements and considerations, along with the appropriate design calculations, should be evaluated for the manufactured system to be installed at a given site. It is appropriate to request the design information for filter systems, detention/storage systems, and separators.

Design Flow Rate

Manufactured systems that are sized using a flow rate, including filter systems and separators, must be designed using the Q_{WQ} as discussed in Section 8.3.2. These systems provide little to no storage.

Volume

Manufactured systems that include a storage volume component must be sized to treat and/or store a volume of stormwater greater than or equal to the WQCV. Manufactured systems may be designed to reduce the peak flow rate for the 2-year event to meet pre-development peak flow rate; however, flows beyond the 2-year event peak flow rate should bypass the facility.

Pollutant Removal Characteristics

The specific pollutant removal characteristics of a manufactured system should be clearly specified. This includes listing all pollutants treated by the system with estimates of median effluent concentration.

For separators, if stormwater has the potential of containing hydrocarbons (i.e. gasoline, oil, petrochemicals) the system should be sized to contain a spill of up to 60 gallons (City of Elizabethton, Tennessee [Elizabethton], 2006). In addition, separators must actively remove floatable debris.

For any proposed installation of a manufactured system, monitoring may be required to verify the installed performance related to pollutant removal.

Overflow or Bypass

An overflow or bypass must be provided and/or specified as part of each manufactured system installation. This overflow or bypass must be designed to convey flows in excess of the manufactured system design. It is recommended that the overflow or bypass point convey excess flow at a point upstream of the manufactured system, rather than at the system itself, therefore conveying flow around the BMP. The overflow or bypass cannot convey flow in excess of the downstream system capacity. All overflow or bypass from the manufactured system should not re-suspend and release material that may already be trapped in the system.

While diversion of flows in excess of the WQCV should occur upstream of all manufactured systems, provisions should still be made at the system itself to safely overflow or bypass stormwater runoff should clogging, a blockage, or failure occur (Elizabethton, 2006).

On-line and Off-line

All filter systems or separators must be constructed off-line, meaning that runoff in excess of the WQCV must bypass the system through an upstream diversion. Storage or detention manufactured systems may be constructed on-line, and be designed for volumes in excess of the WQCV.

Tailwater Effects

For any manufactured system installation, hydraulic design should consider the effects of tailwater from downstream waterways or facilities. The effects of tailwater flooding on the system's hydraulic functionality should be considered.

Subsurface Devices

All subsurface installations of manufactured systems should consider dead and live loads that may be imposed on the structure. Sufficient and suitable access must be provided for each chamber in a manufactured system for inspection and maintenance activities. In addition, the designer should ensure that adequate clearance is available at the installation site for any operations and maintenance equipment. A structural engineering or geotechnical review may be required, depending on the installation.

Operation and Maintenance Plan

To retain a well functioning manufactured system, regular and continual inspections and maintenance is critical. Detailed operations and maintenance requirements that are critical to the manufactured system's continual functionality should be evaluated. Maintenance responsibilities should be defined prior to installation. Considerations and priority should be made for points in the manufactured system susceptible to clogging, any filter cleaning/disposal requirements, and frequency of vac-truck cleaning.

8.6.6.4 Inspection and Maintenance

Maintenance activities for manufactured systems depend on the system installed. All activities are classified as continual and should occur following rainfall or quarterly.

After rainfall equaling or exceeding 0.5 in.

1. Inspect the manufactured system. Check all filters, outlets, and overflow points in the system.

2. If sediment, debris, or other items have accumulated in the system, remove.

3. Clean filters if needed. Unclog or repair outlets and overflow points as needed.

4. Identify inspection/maintenance activities specific to the manufactured system that are critical following rainfall. Note the manufacturer's specifications for the maximum levels of pollutant accumulation allowed before removal is required.

Quarterly:

1. Inspect the manufactured system. Check all filters, outlets, and overflow points in the system.

2. If sediment, debris, or other items have accumulated in the system, remove. Clean the system with a vac-truck, as appropriate.

3. Clean filters if needed. Unclog or repair outlets and overflow points as needed.

4. Identify inspection/maintenance activities specific to the manufactured system that are critical on a biannual basis. Note the manufacturer's specifications for the maximum levels of pollutant accumulation allowed before removal is required.

5. Inspect structural components of the system for cracking, subsidence, spalling, erosion, and deterioration.

8.6.6.5 Submittal Requirements

For review purposes prior to construction, the following minimum submittal requirements are recommended.

1. Drainage area map, specifically outlining drainage area to the manufactured system.

2. Stormwater plan/profile for site and/or drainage area; stormwater routing/conveyance to manufactured system should be clearly delineated.

3. Manufactured system plan, profile, and detailed sections. Each component of the manufactured system should be clearly labeled with dimensions.

4. If retrofit installation, plan, profile, and detailed sections for the retrofit installation should be included.

5. If manufactured system is to be installed either below or above another site component (i.e. parking lot with underground manufactured storage system or building with manufactured Green Roof system), appropriate dead and live load calculations must be submitted, signed and sealed by a professional structural engineer.

6 Manufacturer's specifications for installation.

7. Manufacturer's specifications for maintenance.

8. An as-built survey of the manufactured system is recommended to confirm actual construction/installation adheres to approved construction plans.

9. A long-term inspection/maintenance plan.

8.6.6.6 References

Elizabethton. 2006. Commercial/Industrial Development Stormwater BMP Guidance: http://www.elizabethton.org/business/BMP%20Design%20Guidance%20Final.pdf

MSD. 2009. Proprietary Water Quality Products and the Metropolitan St. Louis Sewer District's Stormwater Management Program: http://www.stlmsd.com/portal/page/portal/engineering/planreview/PlanReviewInformation/ProprietaryBMPs/MSDProprietaryBMPProgramGuidance-080213rev090105.pdf

Virginia. 1999. Virginia Stormwater Management Handbook, Volumes 1 and 2, First Edition: http://www.dcr.virginia.gov/stormwater_management/documents/Chapter_3-15.pdf

8.6.7 Permeable Pavement

Permeable pavement systems are comprised of a pavement surface which allows the infiltration of water into multiple subsurface layers. Depending on the specific design of this BMP, it has the potential to capture and temporarily store stormwater runoff, and filter and infiltrate stormwater runoff into the subsoil. There are several types of pavement surfaces, including:

1. Pervious Concrete – composed of water, coarse aggregate, cement, and little to no fine aggregate. This concrete has large void spaces, allowing water to rapidly infiltrate through it. Pervious concrete applications should conform to the requirements of the American Concrete Institute (ACI) 522.1, Specification for Pervious Concrete Pavement, published by the American Concrete Institute, Farmington Hills, Michigan. The specification should be reviewed by a qualified engineer and modified for the proposed pervious concrete application, as needed.

2. Porous Asphalt – a mixture of asphalt cement, coarse aggregate, and admixtures. As with pervious concrete, little to no fine aggregate is used, producing large void spaces which allow water to rapidly infiltrate to subsurface layers.

3. Permeable Pavers – a system of interlocking blocks which are placed with spaces between them. These spaces then allow for the rapid infiltration of water. A permeable paver can be comprised of openings up to 20-percent of the overall area.

4. Porous Gravel – used in place of traditional gravel drives. This system has a greater depth of gravel than a traditional application and includes a filter material.

5. Reinforced Grass – a system of plastic or concrete pavers which have large openings intended for the placement of aggregate or turf.

Underneath each of these pavement surfaces, a crushed stone aggregate base layer. The aggregate should be clean, washed, and free of fines. This aggregate layer serves as a reservoir, and holds the WQCV until it can be fully infiltrated into the subsoil. An underdrain system is included to ensure the aggregate reservoir drains properly for storm events producing runoff volumes larger than the WQCV. Site conditions may also call for use of perimeter barriers to prevent lateral infiltration.

Design Considerations	
Location characteristics (Slope, Soil Type)[1]	Minimum Slope: 0% Maximum Slope: 5% Soil Type: A, B for maximum infiltration C, D for lower infiltration
Contributing drainage area	The contributing drainage area should never exceed 5 times the surface area of the permeable pavement.
Design size	No restrictions
Detention time for WQCV treatment[1]	48 to 72 hours
Median Effluent Concentrations[2]	TSS =17 mg/L, TP =0.09 mg/L, Cu =3 µg/L
Implementation and Maintenance Considerations	
Potential for use with other BMPs	BMPs with high sediment removal capacities may be located immediately upstream to prevent clogging of pores and decrease maintenance; may be used in combinations with other BMPs to provide full treatment of the design volume.
Maintenance	**High** - Sediment/debris removal

[1] Chicago Stormwater Ordinance Manual, 2011 and USEPA Pervious Concrete Pavement, September 2009

[2] Median effluent concentrations apply to events with measured discharge. Geosyntec Consultants and Wright Water Engineers, Inc 2008

8.6.7.1 General Application

Permeable pavement systems should be sited at locations which are low-speed, low-traffic areas, such as parks, driveways, parking stalls, and pedestrian paths. They should not be located in a place where stormwater could convey large amounts of sediment to it, as this would clog pores and reduce infiltration rates. In addition, runoff from construction activities upstream of a permeable pavement system must be carefully controlled to prevent clogging. As this BMP provides high rates of stormwater infiltration, it must not be placed where contaminants such as pesticides, fertilizers, or other soluble contaminants may be conveyed to the groundwater table.

Figure 8-33 Example of a Permeable Paver Installation

Permeable concrete installations can be sited for higher traffic areas. Such sites require a greater thickness for the concrete pavement layer than installations in low traffic areas. An engineer should specify the thickness of the concrete on a site-by-site basis. If a permeable pavement system is sited next to an existing or a proposed structure, including buildings or other infrastructure, the impact of the increased infiltration on the structure's foundation must be considered.

Figure 8-33 shows an example of a permeable paver installation.

8.6.7.2 Advantages and Disadvantages

Advantages	Disadvantages
Has the ability to provide a large amount of volume reduction, depending on specific design and site conditions	Easily clogged with suspended sediment. As with all filtering systems, maintenance is key to performance.
Provides benefit to water quality	Higher construction and maintenance costs compared to traditional pavement However, some of this cost is offset with the elimination of traditional drainage infrastructure.
Has dual use as road/path infrastructure	Is ineffective when used in areas with a high water table
Suitable for cold-climate applications	Special consideration is required during design phase to ensure infiltration does not affect surrounding structures, if applicable
Reduces impervious area in a watershed	
Contributes to groundwater recharge	
Reduced maintenance in winter due to less frequent occurrences of melting and refreezing.	

8.6.7.3 Design Requirements and Considerations

Overall Design Guidance

1. Permeable pavement should not be constructed until the entire drainage area is permanently stabilized against erosion or a pre-treatment practice is implemented, nor should any activities be completed which could cause large sediment loads to be conveyed to the permeable pavement, such as staging of landscape mulch or soil on or near the pavement. Heavy sediment loads to the pavement will reduce infiltration rates and require additional maintenance to restore the infiltration rate to design levels.

2. Permeable pavement should not be sited where infiltration into the soil at the bottom of the aggregate reservoir could cause damage to surrounding structures due to expansive soils and/or bedrock. The aggregate reservoir drains primarily through infiltration, and would not function properly if not allowed to do so.

3. Coordination and communication between all parties involved, including, but not limited to, the City, the engineer, and the installer, is important in construction of a permeable pavement system. Prior to the construction phase, a meeting between all involved parties should be conducted to communicate the design process and procedures and to establish lines of communication.

4. The aggregate reservoir should be designed to capture at a minimum the required V_D. The design volume is equal to the WQCV unless routing of impervious areas to pervious areas (i.e. cascading planes) occurs within the drainage area of the permeable pavement system. The WQCV is based on 0.5 in. of runoff. If cascading planes are present, the design volume can be reduced because a portion of the WQCV from the impervious area is infiltrated. Refer to Section 8.3 to determine the design volume to use for sizing the permeable pavement system.

5. The design volume should be drained from the system within 48 hrs.

6. Subsurface investigations and the design of the subsurface system should be completed by a qualified engineer.

7. A qualified engineer with experience in the design of both conventional and permeable pavements should complete the pavement design when it will carry vehicular traffic. The engineer should ensure that the subgrade is properly designed, and should complete inspections both during and after construction to ensure that the system is constructed properly and can support traffic loads.

8. For permeable asphalt and concrete applications, a test panel of the pavement should be provided by the installer prior to construction using the proposed design mix and the specification. This panel should have had 30 days to cure.

9. After construction, the infiltration rate of the permeable pavement installation should be tested using the process outlined in ASTM C1701 for pervious concrete and ASTM D5093 for permeable pavers and the results formally reported to the City.

10. The most current ACI 522.1 specification shall be utilized for each installation.

11. Special consideration should be given to ensure that settlement of the pavement is accounted for. An increase in the pavement elevation of up to one-quarter in. may be required to account for the effects of settlement.

Excavation and Subsurface Investigations

1. Excavation is required to construct the aggregate reservoir and/or the underdrain system.

2. A desktop review of available geologic and geotechnical information should be completed prior to any field tests to determine the suitability of a proposed site for a permeable pavement application.

3. If the desktop analysis does not reveal any issues preventing permeable pavement from being constructed on a proposed site, a geotechnical engineer should scope and perform a subsurface study. Table 8-18 outlines the subsurface investigations which should be considered. The geotechnical engineer may modify the recommendations in Table 8-18 or require additional testing depending on site conditions.

Table 8-18

Subsurface Investigations Guideline for Permeable Pavement Applications

Type of Investigation	Goals	Recommendations[1]
Exploratory Borings/Pits	Characterize subsurface conditions Develop subgrade preparation requirements	At least one boring for every 40,000 square feet of permeable pavement, with a minimum of two borings for each installation Extend boring/pit at least 5 feet below proposed bottom of base Extend boring/pit to at least 20 feet below proposed bottom of base when expansive soils or bedrock could be encountered Temporary monitoring wells may be considered for placement in the borings/pits at sites with shallow groundwater which is encountered or believed to exist
Laboratory Tests	Characterize the subgrade Evaluate infiltration rates Assess subgrade for supporting traffic loads	The following tests may be considered: Moisture content (ASTM D2216) Dry Density (ASTM D2936) Atterburg Limits (ASTM D4318) Gradation (ASTM D6913) Swell Consolidation (ASTM D4546) Subsoil infiltration rate (ASTM D3385) Subgrade Support Hydraulic Conductivity

[1] *UDFCD,* 2010

Aggregate Reservoir

The aggregate reservoir will vary in depth and should be composed of American Association of State Highway and Transportation Officials (AASHTO) No. 57 coarse aggregate with all fractured faces. Stone should be clean with no small particles to clog soils. Additional depth may be used to provide additional storage greater than the WQCV, if site conditions allow. Testing of the aggregate should be completed using the Los Angeles Abrasion test as specified by ASTM C131-06. The results of the LA Abrasion test should be submitted to the City for approval prior to construction of the permeable pavement installation. A porosity of 40-percent or less should be used to calculate storage capacity. The bottom of the aggregate layer should be 2 ft. above the normal groundwater table. Disturbance of the subgrade soil by construction activities should be avoided. A nonwoven geotextile fabric should be placed at the bottom and sides of all aggregate reservoirs, unless a perimeter barrier is required to prevent lateral exfiltration. If placement of pavement does not occur immediately following aggregate installation, the aggregate reservoir should be protected by erosion and sediment control to ensure that sediment is not carried to the reservoir as a result of other construction activities.

The ability of a permeable pavement system to capture water varies depending on if it is installed on a flat or a sloped subgrade. A flat installation, shown in Figure 8-34, is preferred as it has a simpler design and maximizes the volume of water the aggregate reservoir can store per unit area of permeable pavement. By comparison, sloped installations require large partitions or flow barriers. As shown in Figures 8-35 and 8-36, these flow barriers are required to fully utilize the aggregate reservoir for storage and filtration of the WQCV. The ability of the pavement to infiltrate the WQCV is reduced as the velocity of water on the pavement surface increases and potentially exceeds the infiltration rate of the pavement. Because of this, sloped installations with longitudinal slopes greater than 1 percent should be designed for only rain falling directly on the pavement surface. No contributing drainage area should be allowed to drain to the pavement surface for such installations. Flat installations with 1 percent or less longitudinal slopes can receive stormwater flow from adjoining drainage areas in combination with the pavement surface.

A qualified engineer should evaluate the aggregate reservoir design to ensure that it has structural integrity for the traffic loadings required at the permeable pavement site.

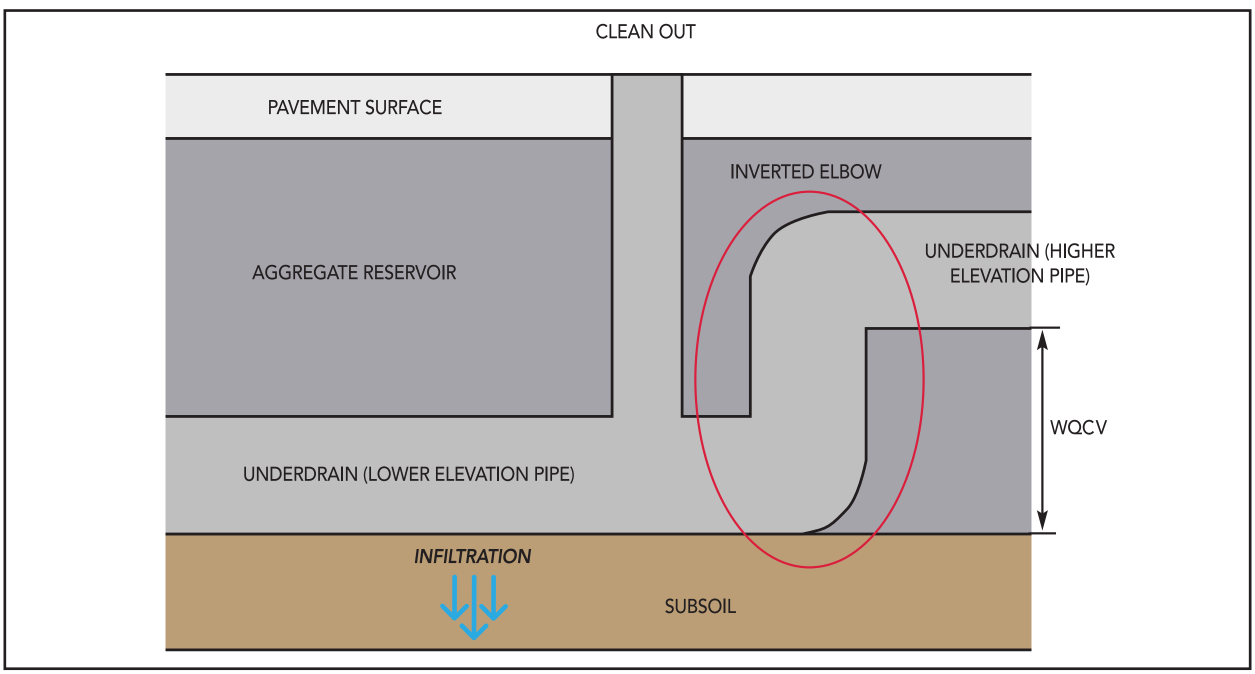

Figure 8-34 Profile View of a Flat Permeable Pavement System

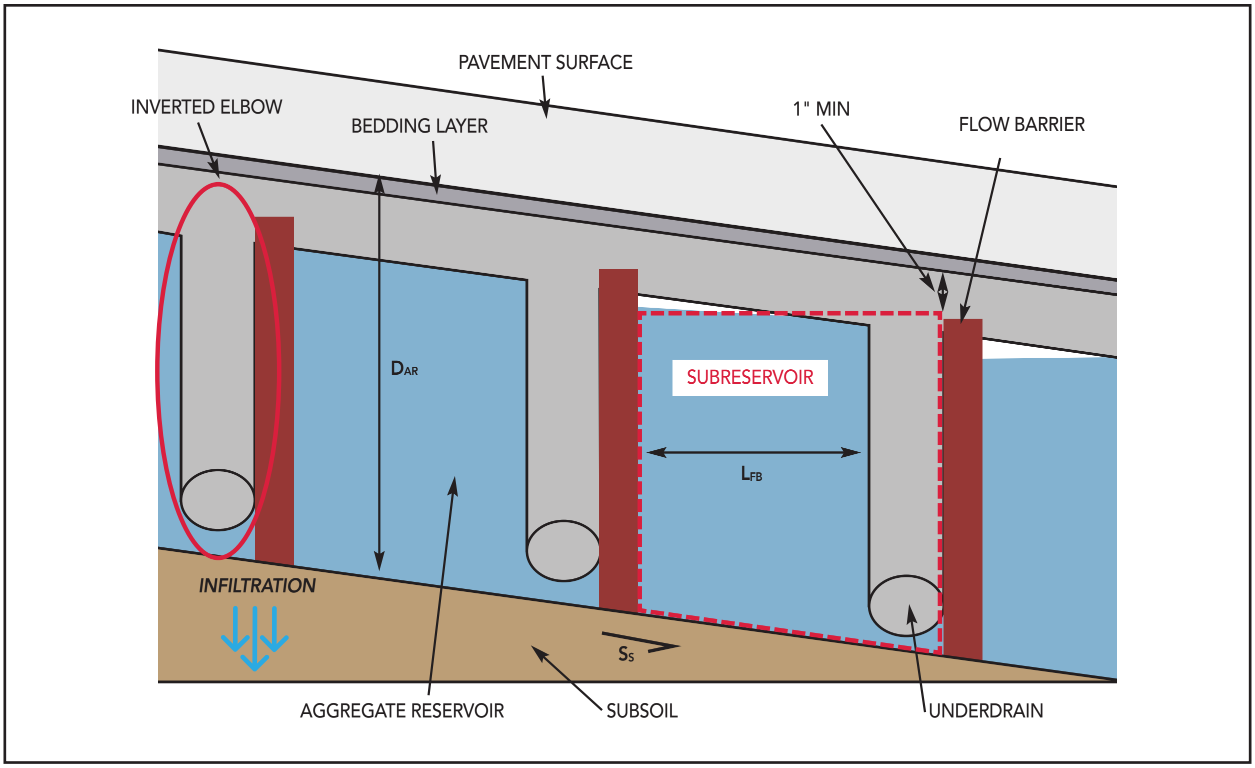

Figure 8-35 Profile View of a Sloped Permeable Pavement System

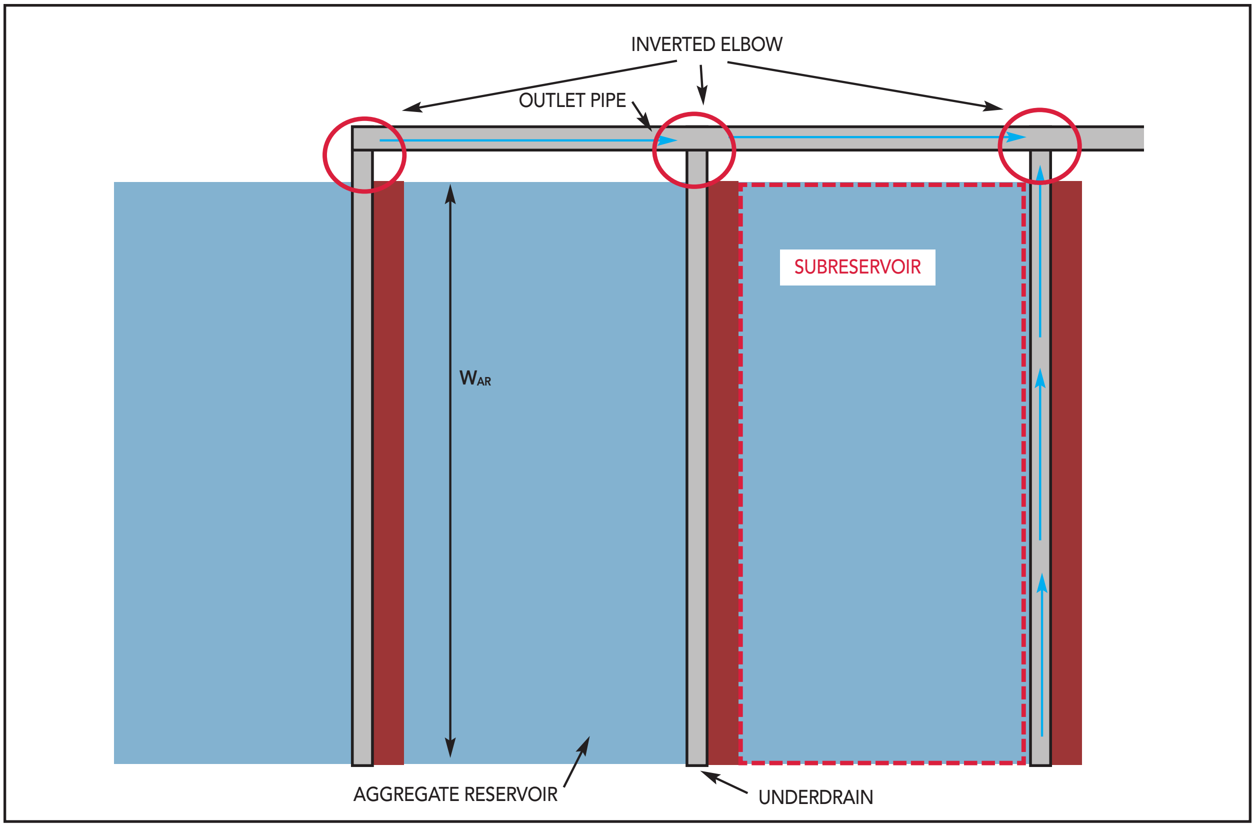

Figure 8-36 Plan View of a Sloped Permeable Pavement System

Perimeter Barriers and Flow Barriers

A perimeter barrier is required for sites where the geotechnical engineer has deemed lateral infiltration a risk to surrounding structures due to expansive soils and/or bedrock. An example of such a scenario would be when the permeable pavement installation is surrounded by conventional pavement. The perimeter barrier should completely surround and contain the permeable pavement system. It should extend from below the aggregate layer to the edge of the installation.

Flow barriers should be provided for sloped permeable pavement installations, as shown in Figure 8-36 These barriers should be buried into the subsoil and extend up into the aggregate base. At least 1-in. of clearance between the top of the barrier and the bottom of the permeable pavement must be maintained. These barriers must be impermeable to keep water from flowing to the lowest point in the aggregate reservoir. If a proprietary system is used for either perimeter or flow barriers, such as an impermeable geomembrane liner, the manufacturer's instructions and specifications for its installation should be followed.

Underdrain System and Outlet Pipe

An underdrain system should be included in the permeable pavement system to ensure that the inflow of stormwater volume is greater than the WQCV or design volumes are conveyed from the aggregate reservoir. The underdrain should run the entire length of the installation so that the invert of the pipe rests on the subgrade soil. Immediately before the pipe leaves the downstream end of the installation, an inverted elbow should be installed, as shown in Figure 8-34. The inverted elbow is a vertical portion of pipe which connects the pipe at the bottom of the aggregate reservoir to a pipe at a higher elevation. The higher elevation pipe should be placed so that its invert allows the WQCV to be stored in the aggregate base, as shown in Figure 8-34.

The underdrain should be made of slotted Polyvinyl Chloride (PVC) pipe meeting the dimensions given below in Table 8-19. The inverted elbow and higher elevation pipe should be made of normal PVC pipe with no slots. The designer should consider requirements to limit peak flow rates to predevelopment conditions (Chapter 2) when sizing the underdrain.

Table 8-19
Slotted Pipe Dimensions

Pipe Diameter	Length Between Slots	Maximum Slot Width	Slot Centers	Open Area Per Foot of Pipe
4 inches	1-1/16 inches	0.032 inches	0.413 inches	1.90 square inches
6 inches	1-3/8 inches	0.032 inches	0.516 inches	1.98 square inches

Note: Variations in these values are expected from available pipe diameters; however, the maximum slot width should not exceed 0.032 inches, and the ability of the pipe to completely drain the reservoir within 48 to 72 hours should be evaluated if large deviations in open area per foot of pipe are encountered.

For sloped permeable pavement installations, an underdrain pipe should be placed in each subreservoir which runs parallel to the flow barrier along the base of the barrier. These should connect to an outlet pipe which spans the entire length of the permeable pavement, as shown in Figure 8-35. For a sloped installation, an inverted elbow is required for each subreservoir, as shown in Figures 8-35 and 8-36. The higher invert outlet pipe has no restrictions and can be constructed using conventional methods with any material pipe normally used for stormwater drainage, so long as it has capacity to drain the aggregate reservoir according to the calculations described in Section 8.6.7.6. The outlet pipe should be constructed using conventional methods, and no material layer is required to surround it.

A cleanout should be placed near the bend of the inverted elbow on the lower elevation pipe in both sloped and flat installations, as shown in Figure 8-34. This will allow for maintenance of the lower elevation pipe.

High Flow Conveyance

High flow conveyance must be provided to divert stormwater away from permeable pavement during large storm events. This conveyance can be located upstream of the pavement or be a structure which conveys overflow from the aggregate reservoir, as with the underdrain system and outlet pipe. It can direct flows to downstream BMPs as part of a treatment train, or discharge to a stormwater conveyance system. The high flow conveyance system should be sized such that all criteria for maximum allowable street encroachment by stormwater, as discussed in Chapter 3: Storm Drainage System, is maintained.

Installation of Concrete Section around Perimeter

Where permeable pavement is installed, a concrete section is required to be installed around the perimeter of the pavement to ensure the pavements are held tightly together under repeated traffic loading. Exceptions can be made in non-vehicular applications subject to City approval. Use of a normal curb section is acceptable in most cases. The bottom of the concrete section should extend to at least the bottom depth of the pavement.

8.6.7.4 Inspection and Maintenance

Maintenance activities for permeable pavements are vital and should be performed at the frequencies indicated below.

As needed

1. Maintain pre-treatment measures (vegetated strips, swales, mechanical devices) to prevent sediment and soil from being carried to the permeable pavement.
2. Monitor to determine if the aggregate reservoir drains properly after typical events via infiltration and underdrain.
3. Remove vegetation growing in the permeable pavement.
4. Ensure surface is free of sediment and clean surface if clogging is observed.

Biannual

1. Clean entire surface by broom, blower, rotary brush, or sweeping.
2. Repair and/or replace joint aggregate after cleaning (for permeable paver applications).
3. Inspect and clean outlet structures.
4. Inspect surface for deterioration or settling.

Every Five Years

Vacuum the entire surface and replace the joint aggregate. For pervious concrete, powerwashing can be completed before vacuuming to loosen sediment. Powerwashing is not recommended for permeable paver systems and vacuuming is the only recommended maintenance procedure. Alternatively, vacuuming could be completed after a rainfall in lieu of powerwashing.

Winter Conditions

Permeable pavements (except for porous gravel) can be snowplowed. Mechanical removal of ice and snow is preferred. Sand should not be placed on permeable pavement, as it can cause clogging. In addition, deicers should be used on a limited basis, as they will flow through the pavement and into groundwater.

8.6.7.5 Submittal Requirements

For review purposes prior to construction, the following minimum submittal requirements are recommended:

1. Drainage area map, including drainage area to the permeable pavement system.
2. Existing contour map with elevations referenced to NAVD 88 and proposed grading layout.
3. Results of geotechnical investigation of site.
4. Plan view with aerial photography of the drainage area with any long term sources of sediment identified, flow paths from these sources to the permeable pavement system, and any pretreatment used to mitigate sediment.
5. Stormwater plan/profile for site.
6. Permeable pavement system plan view and profile view with all components clearly labeled with dimensions.
7. All design calculations (refer to Design Example). If pretreatment is used, all design calculations for the device or devices should be submitted.
8. Detail of proposed underdrain, outlet, and/or high flow conveyance structures with dimensions for construction. Include appropriate design calculations (refer to Design Example).
9. A stormwater control plan which identifies appropriate erosion control measures should be included.
10. Documentation of the infiltration rate of the permeable pavement as determined by ASTM C1701.
11. An as-built survey of the permeable pavement system is recommended to confirm actual construction adheres to approved construction plans.
12. Long-term inspection/maintenance plan.

8.6.7.6 Design Calculations

A short summary of the design calculations is presented below. A detailed design example is outlined in Section 8.6.7.7.

Step 1 Conduct a subsurface investigation to characterize subgrade conditions. The testing should be completed by a geotechnical engineer. If the results of the investigation indicate that an impermeable barrier and/or a perimeter barrier is necessary, the type and material of the barrier(s) should be chosen and the barriers designed.

Step 2 Calculate the WQCV and the V_D. The WQCV is based on 0.5 in. of runoff. Refer to Section 8.3.1 for guidance in calculating the WQCV. If routing of impervious area to pervious area (i.e. cascading planes) occurs within the drainage area of the permeable pavement, the design volume of the permeable pavement can be reduced because a portion of the WQCV from the impervious area is infiltrated. Refer to Section 8.3.4 to determine the reduced WQCV to use the V_D for cascading planes.

V_D = WQCV or if cascading planes exist in permeable pavement drainage area, see Section 8.3.4 for calculation of V_D.

Step 3 Size the depth of the aggregate reservoir. The depth of the aggregate reservoir is based on the design volume (V_D) according to Equation 8-20 for a flat installation and Equations 8-21, 8-22, and 8-23 for sloped installations. Equation 8-20 also accounts for 1-in. of clearance between the top of the flow barrier and the bottom of the permeable pavement surface.

$$D_{AR} = \left(\frac{V_D/_{0.9}}{W_{AR} * L_{AR}} + \frac{1}{12}\right) \tag{8-20}$$

Where:

D_{AR} = Depth of the aggregate reservoir for flat installations (ft.)
V_D = Design volume (cu. ft.)
W_{AR} = Width of the aggregate reservoir (ft.)
L_{AR} = Length of the aggregate reservoir (ft.)

For a sloped permeable pavement system where the slope of the subsoil is approximately equal to the slope of the permeable pavement surface, subreservoirs with equal storage volume are created. The length between flow barriers and total number of the subreservoirs created by the flow barriers must be determined before the depth of the aggregate reservoir can be determined. Equation 8-21 is used to find the length between flow barriers.

$$L_{FB} < \frac{1.5 * V_D}{s_S * A_{pp} * p} \tag{8-21}$$

Where:

L_{FB} = Length between flow boundaries (ft.)
V_D = Design volume (cu. ft.)
s_S = Slope of the subsoil (ft./ft.)
A_{pp} = Area of the permeable pavement (sq. ft.)
p = Porosity of the aggregate reservoir, less than or equal to 0.4

Equation 8-22 is then used to determine the number of subreservoirs. When the number of equally sized subreservoirs is not an integer, this signifies that a subreservoir with a smaller volume than the others will be created.

$$n_r = \frac{L_{AR}}{L_{FB}+W_{FB}} \tag{8-22}$$

Where:

n_r = Number of equally sized subreservoirs
L_{AR} = Length of the aggregate reservoir (ft.)
L_{FB} = Length between flow barriers (ft.)
W_{FB} = Width of the flow boundaries, measured in the direction of flow (ft.)

After calculation of the length between flow barriers and the resulting number of subreservoirs, Equation 8-23 is used to determine the depth of the aggregate reservoir. This equation allows for 1-in. of clearance between the top of the flow barrier and the bottom of the permeable pavement surface.

$$D_{AR} = \left[\frac{V_D}{0.9 * L_{FB} * W_{AR} * \mathrm{P}_{AR} * n_r} + 0.5 * s_S * L_{FB} + \frac{1}{12}\right] \tag{8-23}$$

Where:

D_{AR} = Depth of the aggregate reservoir for sloped installations (ft.)
V_D = Design volume (cu. ft.)
L_{FB} = Length between flow boundaries (ft.)
W_{AR} = Width of the aggregate reservoir (ft.)
P_{AR} = Porosity of the aggregate reservoir, less than or equal to 0.4
n_r = Number of equally sized subreservoirs
s_s = Slope of the subsoil (ft./ft.)

If a permeable pavement system is designed in which the soil slope does not match the slope of the permeable pavement, the calculation of the depth of aggregate required to store the design volume is not straightforward. This is because the heights of the flow barriers will vary, creating subreservoirs with different storage volumes. Should such an installation be proposed, a depth of aggregate reservoir should be assumed and calculations completed as described in Step 4 for this situation.

If a permeable pavement system is proposed which has a permeable pavement slope that is adverse to the subsoil slope, the height of the flow barriers must allow for 1 in. of freeboard between the bottom of the pavement and the maximum water surface in the subreservoir.

Step 4 Calculate volume reduction (if conditions allow). Further volume reduction can be achieved by increasing the depth of the aggregate reservoir. Once the desired aggregate depth has been determined, find the new V_D based on this depth. Equation 8-24 shows the design volume calculation for a flat installation, and Equation 8-25 shows the calculation for a sloped installation.

$$V_D = 0.9 * \left[\left(D_{AR} - \frac{1}{12}\right) * L_{AR} * W_{AR} * \mathrm{p}\right] \tag{8-24}$$

Where:

D_{AR} = Depth of the aggregate reservoir for flat installations (ft.)
V_D = Design volume, flat installation (cu. ft.)
W_{AR} = Width of the aggregate reservoir (ft.)
L_{AR} = Length of the aggregate reservoir (ft.)
p = Porosity of the aggregate reservoir, less than or equal to 0.4

$$V_D = 0.9 * \left[0.5 * s_S * L_{FB} + (D_{AR} - s_s * L_{FB}) - \frac{1}{12}\right] * L_{FB} * W_{AR} * p * n_r \tag{8-25}$$

Where:

V_D = Design volume, sloped installations (cu. ft.)
n_r = Number of equally sized subreservoirs
D_{AR} = Depth of the aggregate reservoir for sloped installations (ft.)
W_{AR} = Width of the aggregate reservoir (ft.)
L_{FB} = Length between flow boundaries, ensure that Equation 8-21 is satisfied for the resulting design volume (ft.)
s_s = Slope of the subsoil (ft./ft.)
p = Porosity of the aggregate reservoir, less than or equal to 0.4

As previously described, where the soil slope does not match the pavement slope, calculation of the storage volume of the proposed aggregate reservoir is not straightforward. To calculate the storage volume, Equation 8-25 should be modified by dropping the term n_r, and then it should be used to calculate the storage volume of each subreservoir. The design volume is then the sum of the volumes stored in all subreservoirs. The result is then compared to the WQCV to see if adequate storage volume is provided by the assumed depth of aggregate reservoir. If there is not, then a larger depth should be assumed and the calculations completed again.

Step 5 Size the underdrain and outlet pipe. Size the underdrain to drain the aggregate reservoir within 48 to 72 hrs. and design the connection to existing stormwater infrastructure. Equation 8-27 gives the pipe size for a flat installation which has one outlet pipe. Equation 8-27 is also used to size outlet pipes for sloped installation subreservoirs, where V_S is the volume stored in each subreservoir, and not the total design volume. Calculation of V_S is given in Equation 8-26. The pipe diameter should be rounded up to the standard 4-in. or 6-in. PVC pipe sizes listed in Table 8-19. If one 6-in. pipe is not large enough, multiple outlet pipes should be used.

For a sloped installation, Equation 8-26 is used to find the volume stored in each subreservoir when they are all equally sized.

$$V_s = \frac{V_D}{n_r} \qquad \textit{(8-26)}$$

Where:

V_S = Volume stored in each subreservoir (cu. ft.)
n_r = Number of equally sized subreservoirs

For sloped installations where n_r is not an integer, a subreservoir with a volume smaller than the others will be created. To determine the volume stored in this smaller subreservoir, Equation 8-26 should first be used to determine the volume stored in the equally sized subreservoirs. The volume of the smaller reservoir is then calculated as the V_S calculated in Equation 8-26 multiplied by the difference between n_r calculated in Equation 8-22 and n_r rounded down to the nearest integer.

Equation 8-27 is used to size the outlet pipes. For the diameter of an outlet pipe from a flat installation or the main outlet pipe from a sloped installation, the volume used to size the pipe is the design volume, V_D. For the subreservoir outlet pipes in a sloped installation, the volume used is the volume stored in each subreservoir, V_S. Equation 8-27 will be used twice in a sloped installation to size the subreservoir outlet pipes and the main outlet pipe from the subreservoir.

$$D_p = 12 * \left(3.47 * 10^{-4} \frac{V*n}{\sqrt{s_p}}\right)^{3/8} \qquad \textit{(8-27)}$$

Where:

D_P = Diameter of the outlet pipe (in.)
V = Design volume (V_D) for flat installation to size main outlet pipe receiving flow from subreservoir outlets; or, Volume stored in each subreservoir (V_S) for a sloped installation subreservoir outlet pipe (cu. ft.)
n = Manning's n value for pipe material, 0.009 – 0.011 for PVC
s_p = Slope of the outlet pipe, minimum 0.005 (ft./ft.)

For a sloped installation, the outlet pipe from a subreservoir smaller than the equally sized subreservoirs should be the same size as is required in the equally sized subreservoirs.

Step 6 Size overflow conveyance. Size overflow conveyance to pass large flows up to the 100-year event and to maintain the peak discharge rates during the two-year storm event to existing conditions. If the outlet pipe is to be used for this purpose, recalculate the required diameter of the outlet pipe using other means.

8.6.7.7 Example

Design a permeable pavement system BMP that will be used as a pedestrian path. The permeable pavement system will be 100-ft. long with a 6-ft. width and will intercept flow from 1,050-sq. ft. of drainage area. The drainage area is highly urban with 85-percent impervious area. The pavement will have a longitudinal slope of 3 percent, as prior approval from the City was obtained, and the subsoil slope will match the pavement slope.

Step 1 Conduct a subsurface investigation to characterize subgrade conditions. Testing was completed by a geotechnical engineer. The results of the investigation indicate that an impermeable barrier and/or a perimeter barrier is necessary, the type and material of the barrier(s) should be chosen and the barriers designed to prevent lateral movement of water.

Step 2 Calculate the WQCV and Design Volume (V_D). The drainage area, A_T, is 1,050 sq. ft., or 0.024 ac. The WQCV is based on 0.5 in. of runoff. No routing of impervious area to pervious area (i.e. cascading planes) occurs within the drainage area of the permeable pavement. Refer to Section 8.3.1 for guidance in calculating the WQCV.

$$V_D = \frac{0.5\ in}{12\ in/ft} * 1{,}050\ ft^2 = 43.75\ ft^3$$

Pretreatment

Runoff from the highly urbanized drainage area is likely to carry high sediment loads. Thus, a pretreatment device is strongly recommended. Vegetated filter strips and vegetated swales work to reduce the velocity of runoff and promote settling of suspended sediments. In situations where area is limited, utilize underground manufactured devices to detain and slow runoff (MARC, 2009). For this example, it is assumed that the pretreatment is implemented properly and that all runoff volume is translated completely to the permeable pavement system.

Step 3 Size the depth of the aggregate reservoir. Because this system will be constructed on a sloped surface, flow barriers will be required. The length between flow barriers must be calculated first. Equation 8-21 is used to calculate the maximum length between flow barriers. A porosity of 0.3 is assumed for the aggregate reservoir.

$$L_{FB} < \frac{1.5 * 43.75 ft^3}{0.03\frac{ft}{ft} * 600 ft^2 * 0.3} = 12.15\ ft$$

For this example, it is assumed that a length between flow barriers of 10 ft. is ideal due to constructability constraints and will be used. Because this is less than the calculated 12.15 ft., it satisfies the condition required by Equation 8-21.

Next, the number of subreservoirs is calculated using Equation 8-22. For this example, the permeable pavement length of 100 ft. and a flow barrier with a width of 0.5 ft. is assumed.

$$n_r = \frac{100 ft}{10 ft + 0.5 ft} = 9.5$$

To calculate the depth of the aggregate reservoir, Equation 8-23 is used.

$$D_{AR} = \left[\frac{43.75 ft^3}{0.9 * 10 ft * 6 ft * 0.3 * 9.5} + (0.5 * 0.03\frac{ft}{ft} * 10 ft) + \frac{1}{12}\right] = 0.52 ft = 6.2\ in$$

The minimum allowable depth for an aggregate reservoir is 12 in., so a D_{AR} of 12 in. is used.

Step 4 Calculate volume reduction. For this example, volume reduction is attained because the D_{AR} of 12 in. is higher than the 6.2 in. depth required to store the WQCV. Because this is a sloped installation, the volume of water the aggregate reservoir stores, is the total volume captured in each subreservoir. To calculate the total volume of storage capacity in the entire aggregate reservoir, Equation 8-25 is used.

$$V_D = 0.9 * \left[0.5 * 0.03\frac{\text{ft}}{\text{ft}} * 10\text{ ft} + \left(1 - 0.03\frac{\text{ft}}{\text{ft}} * 10\text{ ft}\right) - \frac{1}{12}\right] * 10\text{ ft} * 6\text{ ft} * 0.3 * 9.5 = 118\text{ ft}^3$$

Step 5 Size the underdrain and outlet pipe. For this example, the underdrains for the subreservoirs must be sized, as well as the outlet pipe taking flow from each of the subreservoir underdrains. Equation 8-27 is used to find the diameters for both types of pipes. For the subreservoir outlet pipes, the volume in each subreservoir is used in the equation, as calculated using Equation 8-26.

$$\text{V}_\text{s} = \frac{118\text{ft}^3}{9.5} = 12.4\text{ ft}^3$$

This value is then used in Equation 8-27 to find the size of the subreservoir outlet pipes. The minimum allowed slope for the pipes is 0.5%.

$$\text{D}_\text{p} = 12\left(3.47 * 10^{-4}\frac{12.4\text{ ft}^3 * 0.01}{\sqrt{0.005\frac{\text{ft}}{\text{ft}}}}\right)^{3/8} = 0.75\text{ in}$$

The calculated pipe diameter of 0.75 in. is rounded up to 4-in., in accordance with Table 8-19. The sizing of the outlet pipe uses the total design volume and the longitudinal slope of 3% in Equation 8-27:

$$\text{D}_\text{p} = 12\left(3.47 * 10^{-4}\frac{12.4\text{ ft}^3 * 9.5 * 0.01}{\sqrt{0.03\frac{\text{ft}}{\text{ft}}}}\right)^{3/8} = 1.2\text{ in}$$

The calculated pipe diameter of 1.2 in. is rounded up to 4 in.

Step 6 Size overflow conveyance. Overflow conveyance is sized to pass flows up to the 100-year event using control structures upstream of the permeable pavement installation. In addition, peak discharge rates are maintained during the two-year storm event to match existing conditions.

8.6.7.8 References

City of Chicago, Illinois. 2012. *Stormwater Management Ordinance Manual*: http://www.cityofchicago.org/dam/city/depts/water/general/Engineering/SewerConstStormReq/2012StormManual.pdf

Geosyntec Consultant and Wright Water Engineers, Inc. 2008. Overview of Performance by BMP Category and Common Pollutant Type: ISBMPD (1999-2008): http://www.bmpdatabase.org/Docs/Performance%20Summary%20Cut%20Sheet%20June%202008.pdf

MARC. 2009. Manual of Best Management Practices for Stormwater Quality, Section Edition. http://kcmetro.apwa.net/chapters/kcmetro/specs/APWA_BMP_ManualAUG09.pdf.

City of Omaha. 2006. Omaha Regional Stormwater Design Manual: http://www.ci.omaha.ne.us/pw/for-contractors-a-consultants

UDFCD. 2010. Urban Storm Drainage Criteria Manual, Best Management Practices Vol. 3. Denver, Colorado: http://www.udfcd.org/downloads/down_critmanual.htm (listings by volume)

8.6.8 Retention Wet Ponds

The primary components of a retention wet pond include the permanent pool, the littoral bench surrounding the permanent pool, and the live storage volume above the permanent pool. The live volume is sized such that the WQCV displaces a portion of the permanent pool and is released within 12 hrs. of a storm event. The hydraulic residence time of the permanent pool is typically two weeks or more. The primary pollutant removal mechanism is settling as stormwater runoff resides in the permanent pool, but pollutant uptake, particularly of nutrients, also occurs to some degree through biological and chemical activity in the pond (CASQA, 2003). Wet ponds are a highly utilized stormwater BMP practice due to community acceptance and amenity value (Schuler et al., 2007). Retention wet ponds should be used for drainage areas of 10 ac. or more and are typically not used in retrofits due to their typically large area requirement.

Design Considerations	
Location characteristics (Slope, Soil Type)	Slope: Sites < 15% Soil type C, D or a liner may be used
Contributing drainage area	Greater than 10 acres
Design Size	1-3% Drainage area[1] Minimum (L:W) is 3:1
Median Effluent Concentrations[2]	TSS = 13 mg/L, TP = 0.12 mg/L, TN = 1.43 mg/L, Cu = 6.4 µg/L, Fecal Coliform = 133 cfu / 100 mL
Implementation and Maintenance Considerations	
Potential for use with other BMPs	Downstream of source control BMPs, or swales and filter strips
Maintenance	Low – periodic sediment/debris removal, vegetation maintenance

[1] Department of Environmental Protection – Bureau of Watershed Management (DEP), 2006
[2] Geosyntec Consultants and Wright Water Engineers, Inc 2008

8.6.8.1 General Application

Retention wet ponds can be used to improve stormwater runoff quality from roads, parking lots, residential neighborhoods, commercial areas, and industrial sites. A retention wet pond is more applicable to treat larger tributary areas than other BMPs, and can be utilized as a second BMP in a treatment train. Retention wet ponds may be used for a smaller site if the drainage area is sufficient for sustaining a permanent pool. Wet ponds may also be incorporated into an extended storage or a detention pond design for flow control (Metro Council, 2001) and work well in conjunction with other BMPs such as upstream source controls.

Under the proper conditions, retention wet ponds can satisfy multiple objectives, including water quality improvement, erosion protection, creation of wildlife and aquatic habitats, and recreational and aesthetic provision (UDFCD, 2005). Wet ponds are generally ineffective at reducing runoff volumes by themselves (Metro Council 2001) but can be used to reduce runoff flow rates if additional flood control volume is provided above the permanent pool (UDFCD, 2005). Wet ponds also provide some volume reduction through evaporation, typically less than 5-percent (Strecker et al., 2004 as referenced by Schuler et al., 2007).

8.6.8.2 Advantages and Disadvantages

Advantages	Disadvantages
Cost-effective for large drainage areas	Large land area requirement
Removal of both solid and soluble pollutants (Metropolitan Council, 2001)	Inadequate baseflow could result in high salts, nutrients and algae in effluent
Highly effective at nutrient removal	Large storm events could cause low dissolved oxygen or sediment re-suspension
Minimal risk of groundwater contamination	Attract waterfowl, which may increase downstream nutrient loading and bacteria
Sediment removal is less frequent than other BMPs (Metropolitan Council, 2001)	Minimal impact on runoff volume reduction
May provide recreation, wildlife habitat, aesthetics and/or open space	Not suitable for small drainage areas or most retrofits
	Thermal pollution may occur

8.6.8.3 Design Requirements and Considerations

Previous studies have shown that 90-percent of pollutant removal in a wet pond occurs between rainfall events (MD DEQ, 1986 as cited in Metro Council, 2001), and modeling results indicate that two-thirds of the sediment, nutrients, and trace metal loads are removed within the first 24 hrs. after a storm event (Metro Council, 2001). Correct permanent pool storage volume, live storage volume, basin configuration, and outlet sizing are thus very important.

The procedure for designing a retention wet pond is outlined in the following sub-sections. The design components are described in the order of construction starting with excavation for construction of the permanent pool storage area, pretreatment forebay, and inlet/outlet structures.

Overall Design Guidance

- Retention wet ponds should not be constructed until the entire drainage area is permanently stabilized against erosion and sedimentation, or a pre-treatment practice is implemented. Heavy sediment loads to the pond will reduce effectiveness and require premature dredging of the pond to restore its performance.

- To maintain baseflow to the permanent pool in between rainfall events, the minimum drainage area to the detention wet pond should be at least 10 ac. Other environmental conditions such as average ET rates and soil infiltration rates should be considered. High ET and infiltration rates are undesirable for a detention wet pond. Infiltration should be prevented in more conductive soils with a liner to sustain a permanent pool.

Basin Configuration

- To encourage settling/sedimentation, designers should maximize the horizontal and vertical flowpath between the inlet and outlet and avoid “dead zones” in the basin design (Metro Council, 2001). A minimum length to width ratio of 3:1 is recommended. Other ways to avoid short-circuiting include a wedge-shaped pond with inflows on the shallow end (DEP, 2006).

Permanent Pool Storage Area & Live Volume (WQCV)

- Retention wet ponds should be designed such that the live volume (WQCV) is released over 12 hrs. (EPA, 2006).

- The pond depth is an important design factor as it controls sediment deposition. The optimum pond depth should range from 2 to 3 ft. minimum, up to 12 ft. maximum, with an average depth of 4 to 8 ft. Shallow wet ponds tend to have more effective solids removal than their deeper counterparts. However, in pools less than 2 ft. deep, wind will likely re-suspend particles (Metro Council, 2001). If the permanent pool is designed to support fish, sufficient permanent pool depth should be maintained.

- Side slopes of the retention wet pond should be no greater than 4:1. Embankment side slopes may be 3:1 with site-specific approval.
- Design of the permanent pool volume should allow for 14 days hydraulic residence time to allow for particulate settling and nutrient uptake. A longer hydraulic residence time will encourage better settling and sedimentation. This is accomplished by sizing the pool using regional precipitation data and characteristics of the tributary area. These considerations are illustrated in the design example at the end of this section.

Inlet

1. Typical inlet structures include, but are not limited to, drop manholes, rundown chutes, baffle chutes, and pipes with impact basins (Muthukrishnan et al., 2006).
2. All inlets should include some type of energy dissipater to reduce sediment resuspension (MARC, 2009).

Forebay/Pretreatment

1. Wet pond design should include pretreatment to capture sediment. For ponds greater than 4,000 cu. ft. (Metro Council, 2001), a forebay is recommended. For smaller ponds, the design should include a filtration BMP, such as swale or filter strip.
2. The forebay should be a 4 to 6 ft. deep cell delineated by a barrier and should be sized to contain at least 10 percent of the design volume (MARC, 2009).
3. The minimum length to width ratio of the forebay should be greater than 2:1 to prevent short-circuiting (Muthukrishnan et al., 2006).

Littoral Bench

1. The littoral bench slopes should be no steeper than 2:1 (Metro Council, 2001).
2. The littoral bench should extend inward at least 10 ft. from the perimeter of the permanent pool and should be between 6 in. to 12 in. below the permanent pool surface (CASQA, 2003; UDFCD, 2005).
3. The slope of the littoral bench should not exceed 6:1. The bench should be planted with native wetland vegetation to promote biological uptake of nutrients and dissolved pollutants and reduce the formation of algal mats. To maximize biological uptake but prevent plants from encroaching on the open water surface, the vegetated littoral bench should comprise 25 percent to 50 percent of the permanent pool surface area (Nashville Metropolitan-Davidson County [Nashville], 2006).

Outlet Structure

1. Outlet control devices should be designed to prevent clogging, allow maintenance and provide temperature benefits. This can be achieved with a reverse sloped outlet where the invert of the outlet is at the permanent pool elevation, but the water enters the outlet 2 to 7 ft. below the normal water surface.

2. Outlet devices are generally multistage structures with pipes, orifices, or weirs for flow control. Orifices, if used, should be at least 4 in. in diameter and should be protected from clogging by using a trash rack, well screen or other method (DEP, 2006).

3. Outlet devices should be installed in the embankment for accessibility. If possible, outlet devices should enable the normal water surface to be varied. This allows the water level to be adjusted (if necessary) seasonally, as the wet pond accumulates sediment over time, if desired grades are not achieved, or for mosquito control (DEP, 2006).

4. An emergency drain to completely drain the permanent pool for maintenance within 24 hrs. should be incorporated into the design (DEP, 2006).

Siting Considerations

1. Do not locate on fill sites or on/near steep slopes. Depending on soils, bottom modifications can include compaction, incorporating clay into the soil or an artificial liner (Nashville, 2006)

2. The design water surface depth should be a minimum of 20 ft. away from property lines and building structures or per agency specification. A greater distance may be necessary when the retention facility may compromise foundations or slope stability (KC Metro APWA, 2006)

3. For public safety considerations, fences and landscaping should be used to impede access to the facility. The facility should be contoured to eliminate drop-offs or other hazards.

8.6.8.4 Inspection and Maintenance

Maintenance activities for retention wet pond include short-term and long-term maintenance tasks.

Short Term: Year 1 – Year 3 (Post-Installation)

1. Water young plants and seedlings a minimum of weekly for the first three months. Watering may be required more frequently during the summer months (June through August) during the first year. Try to maintain at least a 70-percent vegetation density to ensure stability.

2. Eliminate weeds using spot application of herbicide throughout the first year.

3. Check for signs of erosion or instability and make sure that aesthetics are maintained throughout the BMP footprint

4. After rainfall equaling or exceeding 0.5 in.:

    a. Ensure that vegetation and other erosion stabilizing mechanisms are intact and check inlet/outlet structures and surrounding area for signs of erosion or instability

    b. Inspect all inlet/outlets and repair or restore clogged flow structures as needed

    c. Remove sediment and debris from pretreatment BMP or Forebay

    d. Confirm drainage system functions and bank stability.

5. At one year after installation, inspect vegetation and all other supporting structure. Replace dead plants and remove invasive plant species.

6. Removed sediments should be tested for toxicants and should comply with local disposal requirements.

Long Term: Year 3 – later

1. In early spring, mow or trim vegetation to an approximate height of 6 in. above grade. Remove accumulated debris.
2. Inspect vegetation one to two times each year and remove weeds and invasive species.
3. Trim back or remove overgrown vegetation.
4. Repair or restore clogged high flow structures as needed.
5. At least twice a year, check for subsidence, erosion, tree growth on the embankment, sediment accumulation around the outlet, and erosion within the basin and banks.
6. Removed sediments should be tested for toxicants and should comply with local disposal requirements.

8.6.8.5 Submittal Requirements

For review purposes prior to construction, the following minimum submittal requirements are recommended:

1. Drainage area map, including drainage area to the retention wet pond.
2. Existing and proposed contour map of site (1-ft. contours recommended). Additional spot elevations may be helpful.
3. Geotechnical investigation of site (soil borings, water table location).
4. Stormwater plan/profile for site.
5. Retention wet pond plan view and profile view. Components clearly labeled with dimensions.
6. Hydrologic calculations (refer to Design Example) and average annual water budget analysis.
7. Detail of any proposed outlet and overflow structures with dimensions for construction. Include appropriate design calculations (refer to Design Example).
8. Vegetation plan with schedule for installation and initial maintenance. Appropriate erosion control measures should be included.
9. An as-built survey of the retention wet pond is recommended to confirm actual construction adheres to approved construction plans.
10. Long-term inspection/maintenance plan with responsible party and dedicated funding source.

8.6.8.6 Design Calculations

A short summary of design calculations is presented below. A detailed design example is outlined in Section 8.6.8.7.

Step 1 Determine the WQCV and retention wet pond design live-storage volume (V_D). The WQCV is based on 0.5 in. of runoff. If routing of impervious area to pervious area (i.e. cascading planes) occurs within the drainage area of the retention wet pond, the design volume of the retention wet pond can be reduced because a portion of the WQCV from the impervious area is infiltrated. Refer to Section 8.3 to determine the reduced WQCV to use for sizing the retention wet pond.

V_D= WQCV or if cascading planes exist in EDDB drainage area, see Section 8.3.4 for calculation of V_D.

Step 2 Determine the Rational runoff coefficient for the tributary area.

$$C = 0.3 + 0.6 * (\frac{I}{100}) \tag{8-28}$$

Where:

C = Rational runoff coefficient (unitless)
I = Percent Imperviousness of Drainage Area (unitless)

Step 3 Determine the permanent pool volume. Determine the permanent pool volume required to provide a minimum detention time of 14 days.

$$V = \frac{C*A_T*R_{14}}{12\ in} \tag{8-29}$$

Where:

V = Permanent pool volume (ac.-ft.)
C = Rational runoff coefficient (unitless)
A_T = Tributary area (ac.)
R_{14} = 14-day wet season rainfall for Omaha, NE (1.6 in.)

Step 4 Size the outlet. Determine the outlet type and size such that the V_D is detained and released over 12 hrs. Outlet design must also consider facility dimensions and site constraints. For sizing all retention wet pond outlets, first calculate the average discharge rate for the V_D using Equation 8-30.

Average Discharge Rate

$$Q_{AVG} = \frac{V_D*43{,}560\frac{ft^2}{acre}}{12\ hrs*3{,}600\frac{sec}{hr}} \tag{8-30}$$

Where:

Q_{AVG} = Average discharge rate for the V_D (cfs)
V_D = Design live-storage volume for retention wet pond (ac.-ft.)

Next the Q_{AVG} is used to calculate dimensions for a single orifice or v-notch weir outlets.

Single Orifice

$$D_O = 2\left(\frac{Q_{AVG}}{C_O \times \pi \times (2 \times g \times H_{avg})^{0.5}}\right)^{0.5} * \frac{12\ in}{ft} \tag{8-31}$$

Where:

D_O = Orifice diameter (in.)
Q_{AVG} = Average discharge rate for the V_D (cfs)
V_D = Design live-storage volume for retention wet pond (ac.-ft.)
C_O = Orifice discharge coefficient, Where C_O = 0.66 for weir plate thickness ≤ orifice diameter, and 0.80, otherwise
g = acceleration due to gravity (32.2 ft./s.)
H_{avg} = Average head of V_D(ft.)

V-notch Weir

Dimensions of the V-notch weir outlet include the V-notch weir angle and the top width of the V-notch opening.

$$\theta = 2 * \frac{180}{\pi} * \tan^{-1}\left(\frac{Q_{AVG}}{C_V * H_{avg}^{5/2}}\right) \text{** Note: set angles to radians on calculators and spreadsheets} \quad (8\text{-}32)$$

$$W_V = 2 * Z_{max} * \tan\left(\frac{\theta * \pi}{2 * 180}\right) \text{** Note: set angles to radians on calculators and spreadsheets} \quad (8\text{-}33)$$

Where:

θ	=	Required V-notch weir angle, 20° minimum (degrees)
Q_{AVG}	=	Average discharge rate for the V_D (cfs)
C_V	=	V-notch weir coefficient (2.5)
H_{2yr}	=	Average head of V_D volume over orifice invert (ft.)
W_V	=	Top width of V-notch weir (ft.)
Z_{max}	=	Max V_D depth above outlet (ft.)

Step 5 Size Outlet Protection to avoid clogging. If the chosen outlet structure discharges to a closed system, or if debris in the outlet works would be difficult to remove, determine the appropriate outlet protection to avoid clogging. Outlet protection to avoid clogging may include trash racks, hoods, or reversed slope pipes. Follow guidance in Chapter 6 Storage Facilities to estimate the minimum trash rack size versus outlet diameter or minimum dimensions.

Step 6 Determine the forebay volume. The minimum forebay volume should be 10 percent of the design volume (V_D).

Step 7 Determine the littoral bench dimensions. Determine littoral bench dimensions based on permanent pool volume and littoral bench design guidelines.

8.6.8.7 Example

Design retention wet pond to accept runoff from an 18-ac., single-family residential development (30% impervious). The developer would like to design a wet pond with a single orifice outlet.

Step 1 Determine the WQCV and retention wet pond design live-storage volume (V_D). The drainage area to the retention wet pond is 18 ac. Using 0.5 in. of runoff, the WQCV is calculated as:

$$WQCV = \frac{0.5\ inches}{12\ inches} * 18\ acres = 0.75\ acre - feet$$

Routing of impervious area to pervious area (i.e. cascading planes) reduces the design volume of the retention wet pond because a portion of the runoff from the impervious area is infiltrated. When cascading planes are used, estimate the retention wet pond design volume using Section 8.3.4. For this example, no cascading planes are present so the design live-storage volume (V_D) is equal to the WQCV.

$$V_D = WQCV = 0.75\ acre - feet$$

Step 2 Determine the Rational Runoff Coefficient for the tributary area. To calculate the permanent pool volume, the rational runoff coefficient must first be calculated. Using Equation 8-28:

$$C = 0.3 + 0.6 * \left(\frac{30}{100}\right) = 0.48$$

Step 3 Determine the permanent pool volume. Determine the permanent pool volume required to provide a minimum detention time of 14 days. Using Equation 8-29, the permanent pool volume is:

$$V = \frac{0.48 * 18\ acres * 1.6\ inches}{12\ inches} = 1.15\ acre - feet$$

Step 4 Size the outlet. The developer would like to install a single orifice outlet for this particular wet pond. To size the outlet, first the water quality discharge should be calculated using Equation 8-30.

$$Q_{AVG} = \frac{0.75\ acre - ft * 43{,}560 \frac{ft^2}{acre}}{12\ hrs * 3{,}600 \frac{sec}{hr}} = 0.76\ cfs$$

Then, the orifice diameter should be calculated using Equation 8-31. The desired average depth of the V_D above the outlet is 2 ft.

$$D_O = 2\left(\frac{0.76\ cfs}{0.66 * \pi * (2 * 32.2 * 2\ ft)^{0.5}}\right)^{0.5} * \frac{12\ in}{ft} = 4.3\ inches$$

Step 5 Size Outlet Protection to avoid clogging. For this example, the orifice outlet discharges to a closed system; therefore, a trash rack is provided. The openings for the trash rack should be calculated based on the orifice diameter calculated. The purpose of sizing openings for the trash rack is to find the optimal size of the openings to let the required discharge pass while protecting the outlet from clogging. First the area of the orifice opening is calculated.

$$A_{ot} = \frac{1}{4}\pi * 4.3^2 = 14.5\ in^2$$

Next, using Figure 6-13 in Chapter 6, the minimum area that the trash rack should cover around the outlet is calculated:

$$A_t = 14.5 * 77e^{-0.124*4.3} = 655\ in^2, or\ 4.5\ ft^2$$

Step 6 Determine the forebay volume. The minimum volume of the forebay is equal to 10-percent of the design volume (V_D). With a design volume of 0.75 ac.-ft. the forebay volume is 0.075 ac.-ft. The minimum length to width ratio should be greater than 2:1.

Step 7 Determine the littoral bench dimensions. The littoral bench slopes should be no steeper than 2:1 and should extend inward at least 10 ft. from the perimeter of the permanent pool. The benches should be between 6 in. to 12 in. below the permanent pool surface.

8.6.8.8 References

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8.6.9 Soil Conditioning

Soil conditioning is a post-construction practice intended to improve disturbed and low organic soils through mechanical compaction reduction and compost amendment in order to increase macroporosity and improve water retention. This practice is intended to reduce the generation of runoff from the area where it is implemented.

Design Considerations	
Location characteristics (Slope, Soil Type)	Slope: Sites < 10% Soil type: All Outside of tree dripline. Water table > 1.5 feet from surface Greater than 10 feet from building foundation.
Contributing drainage area	This BMP is not designed to control runoff from other areas. It is designed to control the rainfall falling on it.
Design Size	Soil depth of at least 6 inches.
Pollutant Removal Efficiency[1,2]	TSS = 65%, TKN = 72%, NH4-N = 54%, TP = 20-76% Dissolved P = 89%
Implementation and Maintenance Considerations	
Potential for use with other BMPs	Upstream of structural BMPs
Maintenance	Vegetation establishment, maintenance, and weed management.

[1] Median Effluent Concentrations were not available for this BMP.

[2] Tyler, et.al. 2010.

8.6.9.1 General Application

Soil conditioning is acceptable for any pervious area where the soils have lost their inherent infiltration and water storage capacity through compaction and disturbance. This BMP will be most applicable for large site areas proposed for turfgrass, low maintenance lawn, or nature prairie plantings. It can also be applied to areas within landscape beds given all specifications are met for soil conditioning and tilling within the bed areas, and the beds are not bermed with slopes greater than 10%. Soil conditioning for the purpose of stormwater management should be used:

1. When slopes are less than 10%
2. Outside the dripline of a tree, to avoid damaging the root system
3. When existing soils are not saturated or seasonally wet
4. Greater than ten (10) ft. of the foundation of a building
5. When the water table is greater than 1.5 ft. of the soil surface
6. Where runoff velocities will not damage or undermine vegetation

8.6.9.2 Advantages and Disadvantages

Advantages	Disadvantages
Reduces runoff peaks and volumes for small storm events and the initial rainfall of larger events.	Compost must meet specifications or performance may be diminished
Simple design, construction, and maintenance	If compaction occurs on soil conditioned area performance will be diminished
Encourages healthy plant growth	Cannot be used to control runoff from off-site areas.
Increases biological diversity and activity in the soil complex	

8.6.9.3 Design Requirements and Considerations

The following steps should be followed to properly condition disturbed soils for stormwater management.

- Step 1 – Ensure site conditions are dry prior to beginning the soil conditioning process to avoid further compacting soils.
- Step 2 – Remove existing vegetation, including turf, and till the ground to a minimum depth of 6 in.
- Step 3 – Place a 3-in. deep layer of specified compost on top of the tilled ground and till compost into a depth of 6 in. of existing soil. See Table 8-20 for compost specifications.
- Step 4 –Fine grade the site with minimum equipment passes (no more than two (2) passes) to reduce the potential for soil compaction. Finalizing all preliminary critical spot elevation, slopes and positive drainage criteria for the site should be completed as much as possible prior to finish grading in order to ensure that equipment compaction is minimized after soil is worked an amended.
- Step 5 – Firm soil using one pass of a 50-pound roller if vegetative cover will be drill seeded or plugged to help ensure successful plant establishment.
- Step 6 – Establish vegetative cover immediately after finish grading and take steps to prevent erosion during establishment, including but not limited to installing erosion control blankets, silt fence or straw waddles. Vegetation may be sodded, seeded, or plugged. For seeding or plugging, all standard procedures shall be followed for the appropriate mulching of bare soil surface areas until vegetation is fully established. Expectations of early plant performance must be understood (i.e. – there may be a short period of plant stress due to nutrient cycling in compost) and incorporated into the management plan (see Step 7).
- Step 7 – A management plan is required in the PCSMP for all areas that have undergone soil conditioning. The plan must be followed during the first two years of plant establishment. Components include weed management, spot reseeding, maintaining moisture during germination and initial establishment, inspection and intensive rainfall events, and the removal of erosion control measures as needed.

Compost Specifications

The compost used in soil conditioning shall be derived from plant material, and the result of biological degradation and transformation of plant derived materials under conditions that promote anaerobic decomposition. The material shall be well composted, free of viable weed seeds, and stabilized with regard to oxygen consumption and carbon dioxide generation. The compost shall have a moisture content that has no visible free water or dust produced when handling the material. It shall meet the criteria presented in Table 8-20, as reported by the U.S. Composting Council STA Compost Technical Data Sheet provided by the vendor. OmaGro is a locally produced compost product that is acceptable for use in soil conditioning.

Table 8-20
Compost Criteria for Soil Conditioning

Compost Criteria
One hundred percent of the material must pass through a half inch screen
The pH of the material shall be between 6 and 8
Manufactured inert material (plastic, concrete, ceramics, metal, etc.) shall be less than 1.0%by weight.
Organic matter should be between 35 and 65 %
Soluble salt content shall be less than 6.0 mmhos/cm
Maturity should be greater than 80 %
Stability shall be 7 or less
Carbon/nitrogen ratio shall be less than 25:1
Trace metal test result = “pass”
The compost must have a dry bulk density ranging from 40 to 50 lbs/ft^3.

Vegetation

Perennial grasses are usually specified and native grasses are preferred. A range of plant material can be used in conditioned soils areas including legumes, deep rooted grasses, shrubs, and trees.

8.6.9.4 Inspection and Maintenance

Short Term: Year 1 – Year 3 (Post-Installation)

1. Water vegetation a minimum of weekly for the first three months. Watering may be required more frequently during the summer months (June through August) during the first year. Try to maintain at least a 70% vegetation density to ensure stability.

2. Eliminate weeds manually or by using spot application of herbicide throughout the first year.

3. Check for signs of erosion or instability and make sure that aesthetics are maintained throughout the soil conditioned area.

4. After rainfall equaling or exceeding 0.5 in., ensure that vegetation and other erosion stabilizing mechanisms are intact.

5. At one year after installation, inspect vegetation and all other supporting structures. Replace dead plants and remove invasive plant species.

Long Term: Year 3 – later

1. In early spring, mow or trim vegetation to a height no less than 6 in. Remove accumulated debris.

2. Inspect vegetation one to two times each year and remove weeds and invasive species.

3. Trim back or remove overgrown vegetation.

4. At least twice a year, check for subsidence, erosion, and sediment accumulation.

8.6.9.5 Submittal Requirements

For review purposes prior to construction, the following minimum submittal requirements are recommended:

1. Area of proposed soil conditioning and any proposed or existing downstream BMPs.
2. Existing and proposed contour map of site (1-ft. contours with elevations tied to NAVD 1988 datum recommended). Additional spot elevations may be helpful.
3. Geotechnical investigation of site (soil borings, water table location).
4. Vegetation plan with schedule for installation and initial maintenance. Appropriate erosion control measures should be included.
5. Long-term inspection/maintenance plan with responsible party and dedicated funding source.

8.6.9.6 Design Calculations

Equation 8-31 can be used to calculate the volume of compost required for an area proposed for soil conditioning.

$$V_{compost} = A_{SC} * d_{compost} * \frac{1\,ft}{12\,inches} * \frac{1\,yd^3}{27\,ft^3} \quad (8\text{-}34)$$

Where:

$V_{compost}$ = volume of compost needed for soil conditioning, $yd.^3$
A_{SC} = Area of proposed soil conditioning, $ft.^2$
$d_{compost}$ = depth of compost, in.

8.6.9.7 Example

Calculate the volume of compost necessary to implement 5,000 $ft.^2$ of soil conditioning, to be planted with turf grass. Use the recommended compost depth of 3 in.

$$V_{compost} = 5{,}000\,ft^2 * 3\,inches * \frac{1\,ft}{12\,inches} * \frac{1\,yd^3}{27\,ft^3} = 46.3\ yards\ of\ compost$$

8.6.9.8 References

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8.7 Lot-Level/Homeowner Non-Structural Best Management Practices

Stormwater pollution is the untreated contaminated water that drains from lawns, parking lots and streets through the municipal storm drain system. When harsh chemicals from lawns and toxic substances from spills enter waterways, harmful pollutants kill fish, destroy wildlife habitats, decrease aesthetic value and contaminate the water people use as a source for drinking, boating and swimming. Non-structural BMPs that help reduce the quantity of pollutants from reaching the waterways include but are not limited to; using sustainable methods of lawn care and landscape maintenance, reducing trash and pet waste, and sweeping and cleaning impervious surfaces (University of Nebraska-Lincoln Water, 2011).

8.7.1 Lawn Care and Landscape Maintenance

Stormwater runoff from a healthy dense lawn (with up to medium dense soil) rarely occurs, except during intense rainfall events (University of Minnesota, 2006: Bierman, et al., 2010). In order to maintain a healthy lawn, it is important to select lawns that are adapted to the region's climate. The Nebraska Master Gardener Program, hosted by the University of Nebraska-Lincoln, provides a link to a sustainable urban landscape website hosted by the University of Minnesota. The University of Minnesota website contains links to assist in lawn management and sustainable landscape design. Additionally, maintenance is important to keep a healthy lawn system. Maintenance techniques that support a healthy lawn include; soil care, method of fertilizing, and method of mowing.

8.7.1.1 Soil Care

Soil is the foundation for a healthy lawn and landscape. Soil should be tested to determine the soil's compaction. Soil testing can be done by pushing a screwdriver into the soil. If the screwdriver requires pounding to enter the soil, the soil is compacted (Center for Watershed Protection, 2000). Soil preservation and amendments are discussed in previous Sections. Soil that is compacted can be treated using a hand corer or a mechanical aerator. Compost can be applied to existing lawns to improve soil compaction (Center for Watershed Protection, 2000).

8.7.1.2 Reduce Turf Area

Conversion of lawns to groundcover, trees, shrubs or meadow plantings can greatly reduce the quantity of stormwater runoff as well as reduce the cost, time and effort needed to maintain turf based yard (Center for Watershed Protection, 2000). Areas that are not suited for lawn are best for conversion. These areas include frost pockets, exposed areas, shaded areas, steep areas and wet areas (Center for Watershed Protection, 2000). Additionally areas that are difficult or dangerous to mow are good candidates for conversion. Areas that are difficult to fertilize or water evenly are also good conversion candidates. Refer to Section 8.5.1 Rain Gardens in Residential Areas for possible replacement landscape designs.

8.7.1.3 Fertilizer Methods

Test soil's pH and fertility level, prior to adding any fertilizers, as many soils do not need additional fertilizers to support a healthy lawn. The USDA provides a soil quality test guide. Nebraska Department of Agriculture provides a list of soil and plant testing laboratories that are in compliance with the Nebraska Soil and Plant Analysis Laboratory Act Regulations. Additionally many home improvement and hardware stores carry inexpensive soil testing kits, in their lawn and garden departments.

Leaving grass clippings or mulch/mowing typically provides adequate levels of nitrogen and phosphorus to maintain a healthy lawn. If a commercial fertilizer is used it is best to use a minimal amount of fertilizer with encapsulated nitrogen and no or low phosphorus. The use of slow release (encapsulated) nitrogen and low or no phosphorus fertilizer helps reduce stormwater pollutant loads. Unless a soil test indicates otherwise, phosphorous is only needed during the first year of establishment of the lawn. When applying common off the shelf commercial fertilizer use half the manufacturer's recommended rate, as studies have found most lawns do not require high doses of fertilizer (Center for Watershed Protection, 2000). Additionally when to apply fertilizer is equally important: cool season grasses are best fertilized once in the fall and warm season grasses are best fertilized in several small doses during the summer (Center for Watershed Protection, 2000). By applying fertilizers during the correct season the turf utilizes more of the nutrients which relates to a more cost effective method of maintaining the turf as well as helping to reduce the amount of pollutants in stormwater runoff.

In order to keep the fertilizer on the lawn and plants where it provides its benefits and to keep the fertilizer out of waterways, avoid using fertilizer just before it rains.

8.7.1.4 Lawn Care

Follow the three in. rule: never cut your lawn shorter than three in. By increasing the mowing height the health of the turf is improved as it helps prevent the grass crowns from being exposed to sunburn. Taller grass also keeps the soil from being exposed to sunlight which can cause weed seeds to germinate. Increased mowing height encourages deeper root growth which in turns causes the grass to be healthier. Cutting the grass at a taller height also helps retain the moisture during drier seasons.

8.7.2 Trash and Pet Waste Reduction

Litter disposed of in a storm drain can choke, suffocate and disable aquatic life. Dispose of litter by throwing it in a trash can or recycling it.

8.7.2.1 Trash Reduction

Cleaning products and other household chemicals should never be dumped outside, down the sink or down a storm drain. You can dispose of your household chemicals for free at Under the Sink, the City's household hazardous waste disposal facility. Check their website for drop off information.

Landscape waste while organic can still be problematic to waterways; the problem occurs as this waste contributes to higher levels of nutrients entering the waterways which in turn encourage algae and rooted plants to grow in lakes and streams (Janssen and Barrow, 2008). Methods to reduce yard waste include utilizing lawn care tips listed in Section 8.7.1.4 and composting in backyards. University of Nebraska-Lincoln Extension provides a website (Stormwater Management: Yard Waste Management) with information regarding how to start composting and other waste reduction tips.

8.7.2.2 Pet Waste Reduction

Pet waste dumped in storm drains goes straight into rivers and lakes, contaminating the water. Pet waste left on lawns can cause harmful bacteria and viruses to enter waterways, causing pollutant damage. It is best to dispose of pet waste in the trash or flush it down the toilet. The waste will be properly treated in the landfill or wastewater treatment plant. When taking your dog for a walk, remember to take some plastic bags to clean up after them. Do not throw the bag down the storm drain. Stormwater runoff is not treated.

Incorporate pet waste stations in multi-family and apartment complex common areas and public parks and trail systems.

8.7.3 Sweeping and Cleaning of Impervious Areas

Do not leave grass clippings and leaves on impervious areas (such as driveways and streets) when you are doing yard work. This debris can enter the storm drain and cause clogs and pollute the water. Decaying leaves deplete water's oxygen levels which can harm aquatic organisms.

Pick up any spilled chemicals, fertilizers, oils, etc as rainwater can pick these pollutants up and deposit them into the waterways. Use cat litter or other absorbents to soak up liquid spills and then sweep up and properly dispose of the used absorbent.

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Appendix 8-A

Simple Method to Calculate Urban Stormwater Pollutant Loads and BMP Performance

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Appendix 8-A
Simple Method to Calculate Urban Stormwater Pollutant Loads and BMP Performance

Estimating the expected annual pollutant loads from the developed area will assist in determining target pollutants. Estimating BMP pollutant removal performance helps in selecting post-construction BMPs that are most effective in removing the targeted constituents from site runoff.

Estimating Annual Pollutant Loads from Developed Areas

To quickly calculate the expected pollutant load from an urban area, the Stormwater Center's Simple Method to Calculate Urban Stormwater Pollutant Loads[1] can be used. The Stormwater Center has summarized event mean concentrations (EMCs) of pollutants from different land uses. A summary of the Stormwater Center data is shown in Table A-1. EMCs for bacteria have also been published by the Papillion Creek Partnership and are shown in Table A-2.

Table A-1
Simple Method Model Default Value EMC

Pollutant	Land Use			
	Residential	Commercial	Roadway	Industrial
Total Nitrogen (mg/l)	2.2	2	3	2.5
Total Phosphorus (mg/l)	0.4	0.2	0.5	0.4
Total Suspended Solids (mg/l)	100	75	150	120

Source: Stormwater Center Website, accessed July 2011.

The Simple Method estimates pollutant loads for chemical constituents as a product of annual runoff volume and EMC, as:

$$L = 0.226 \times R \times C \times A$$ *Equation A-1*

Where:

L = Annual load (lbs)
R = Annual runoff (inches)
C = EMC (mg/L)
A = Area (acres)
0.226 = Unit conversion factor

For bacteria, the equation is slightly different, to account for the differences in units. The modified equation for bacteria is:

$$L = 1.03x10^{-3} \times R \times C \times A$$ *Equation A-2*

Where:

L	=	Annual load (Billion Colonies)
R	=	Annual runoff (inches)
C	=	EMC bacteria (Colony Forming Units (CFU)/100 ml)
A	=	Area (acres)
1.03 * 10-3	=	Unit conversion factor

The annual runoff in inches is calculated as a product of annual runoff volume, and a runoff coefficient (Rv). Runoff volume is calculated as:

$$R = P \times P_j \times R_v$$ *Equation A-3*

Where:

R	=	Annual runoff (inches)
P	=	Annual rainfall (inches) Table A-3
P_j	=	Fraction of annual rainfall events that produce runoff (usually 0.9)
R_v	=	Runoff Coefficient

$$R_v = 0.05 + 0.9 \times I_a$$ *Equation A-4*

Where:

I_a	=	Impervious fraction

Table A-2
Possible Sources and Concentrations of Fecal Coliform and E. coli in the Papillion Creek Drainage Basin

Land Use Category	Effective Percent Impervious	Fecal Coliform Bacteria (CFU/100 mL)	Equivalent E. Coli Bacteria Loading (CFU/100 mL)
Agriculture	Varies	88,400	55,700
Parks and Open Areas	5%	11,600	7,300
Rural Estate (Homes on 3 to 10 acres)	10%	17,100	10,800
Low Density Residential (Homes on 1 to 3 acres)	16%	23,700	14,900
Medium Density Residential (Homes on approx. 0.25 Acres)	38%	48,100	30,300
Churches, Schools, and Civic	50%	61,300	38,600
High Density Residential (Multi-Family Apartment Complexes)	65%	77,900	49,100
Industrial Areas	72%	85,600	53,900
Commercial and Retail Businesses	85%	100,000	63,000

Source: Papillion Creek Partnership. 2009. Final Papillion Creek Watershed Management Plan.

Table A-3
Annual rainfall for Municipalities within the Omaha Region

City	NRCS (30 year average)
Blair	30.06
Boys Town	28.89
Chalco	28.89
Elkhorn	30.15
Gretna	29.69
Las Vista	28.89
Omaha	30.26
Papillion	28.89
Ralston	28.89
Waterloo	30.15

[1]Rain gauge information for Douglas and Washington County for NRCS based on time frame of 1971-2000

[2]Rain gauge information for Sarpy County (Ashland) for NRCS based on time frame of 1961-1990

Once the pollutant load from a particular land use is estimated, the BMP performance can also be estimate. The pollutant load leaving a BMP is a function of the volume of water leaving the BMP and the effluent concentration. The Simple Method to Calculate BMP Performance (Simple Method) can be used to estimate the pollutant removal effectiveness of BMP types. The Simple Method provides an estimate of BMP performance; **actual pollutant removal performance for a particular BMP can only be verified using post-construction monitoring data.**

Simple Method to Calculate BMP Performance

BMP performance can be estimated by comparing the pollutant load entering the BMP to the pollutant load exiting the BMP. The International BMP database recommends using effluent concentrations and outflow volumes to measure BMP performance. The reasons for this recommendation are summarized in the Percent Removal Factsheet[2].

The pollutant load entering the BMP is estimated using Equation A-1 or A-2. The pollutant load exiting the BMP is estimated using Equation A-5 which multiplies the median effluent concentration based on BMP type and by the outflow volume.

$$E = 0.226 \times O \times C \times A$$ *Equation A-5*

Where:

E = Effluent Pollutant Load (lbs)
O = Outflow Volume in Watershed Inches (inches)
C = Median Effluent Concentration of BMP (mg/l)
A = Area (acres)
0.226 = Unit conversion factor

http://www.stormwatercenter.net/monitoring%20and%20assessment/simple%20meth/simple.htm

For BMPs that do not provide significant reduction in stormwater volume, then the outflow volume is equal to the inflow volume. The International BMP Database indicated that volume reduction is most significant in filter strips, grassed swales and bioretention BMPs. Volume percent removal estimates for these BMPs are provided in Table A-4.

Table A-4
International Stormwater BMP Database Percent Volume Reduction

BMP Category	25th Percentile	Median	75th Percentile	Average
Biofilter – Grass Strips	18%	34%	54%	38%
Biofilter – Grass Swales	35%	42%	65%	48%
Bioretention	45%	57%	74%	61%
Detention Basins – Surface Grass Lined	26%	33%	43%	33%

Relative Volume Reduction = Study total Inflow Volume – Study Total Outflow Volume / Study Total Inflow Volume

Source: Wright Water Engineers and Geosyntec Consultants, 2011

The International BMP Database also publishes median effluent concentrations for several BMP types. A summary of the BMP Database information on median effluent concentrations for common pollutants (except bacteria) is provided in Table A-5. Table A-6 shows the median effluent concentrations of select BMPs for bacteria

Table A-5
Structural BMP Median Influent and Effluent Concentrations from the International BMP Database

Constituents	Sample Location	Detention Pond (n=25)[1]	Wet Pond (n=46)[1]	Wetland Basin (n=19)[1]	Biofilter (n=57)[1]	Media Filter (n=38)[1]	Porous Pavement (n=6)[1]
Suspended Solids (mg/L)	Influent	72.65	34.13	37.76	52.15	43.27	
	Effluent	31.04	13.37	17.77	23.92	15.86	16.96
Total Cadmium (µg/L)	Influent	0.71	0.49	0.36	0.54	0.25	
	Effluent	0.47	0.27	0.24	0.30	0.19	xx
Dissolved Cadmium (µg/L)	Influent	0.24	0.19		0.25	0.16	
	Effluent	0.25	0.11	xx	0.21	0.13	xx
Total Copper (µg/L)	Influent	20.14	8.91	5.65	31.93	14.57	
	Effluent	12.10	6.36	4.23	10.66	10.25	2.78
Dissolved Copper (µg/L)	Influent	6.66	7.33		14.15	7.75	
	Effluent	7.37	4.37	xx	8.40	9.00	xx
Total Chromium (µg/L)	Influent	7.36	6.00		5.63	2.18	
	Effluent	3.18	1.44	xx	4.64	1.48	xx
Total Lead (µg/L)	Influent	25.01	14.36	4.62	19.53	11.32	
	Effluent	15.77	5.32	3.26	6.70	3.76	7.88
Dissolved Lead (µg/L)	Influent	1.25	3.40	0.50	2.25	1.44	
	Effluent	2.06	2.48	0.87	1.96	1.18	xx
Total Zinc (µg/L)	Influent	111.56	60.75	47.07	176.71	92.34	
	Effluent	60.20	29.35	30.71	39.83	37.63	16.60
Dissolved Zinc (µg/L)	Influent	26.11	47.46		58.31	69.27	
	Effluent	25.84	32.86	xx	25.40	51.25	xx
Total Phosphorus (mg/L)	Influent	0.19	0.21	0.27	0.25	0.20	
	Effluent	0.19	0.12	0.14	0.34	0.14	0.09
Dissolved Phosphorus (mg/L)	Influent	0.09	0.09	0.10	0.09	0.09	
	Effluent	0.12	0.08	0.17	0.44	0.09	xx
Total Nitrogen (mg/L)	Influent	1.25	1.64	2.12	0.94	1.31	
	Effluent	2.72	1.43	1.15	0.78	0.76	xx
Nitrate-Nitrogen (mg/L)	Influent	0.70	0.36	0.22	0.59	0.41	
	Effluent	0.58	0.23	0.13	0.60	0.82	xx
TKN (mg/L)	Influent	1.45	1.26	1.15	1.80	1.52	
	Effluent	1.89	1.09	1.05	1.51	1.55	1.23

1 Actual number of BMPs reporting a particular constituent may be greater or less than the number reported in this table, which was based on number of studies reported in database based on BMP category. Notes: xx- Lack of sufficient data to report median and confidence interval. Values in parenthesis are the 95% confidence intervals about the median. Differences in median influent and effluent concentrations does not necessarily indicate that there was a statistically significant difference between influent and effluent. See "Analysis of Treatment System Performance, International Stormwater BMP Database (1997-2007) (Geosyntec and Wright Water Engineers and Geosyntec Consultants 2007) for more detailed information. Source: International Stormwater BMP Database June 2008 (www.bmpdatabase.org)
2Source: Wright Water Engineers and Geosyntec Consultants, Pollutant Category Summary: Fecal Indicator Bacteria, December 2010

Table A-6
Structural BMP Median Influent and Effluent Concentrations from the International BMP Database.

Constituent	Sample Location	Detention Pond (n=11)	Wet Pond (n=6,7)	Grass Swale/Strip (n=9)	Media Filter (n=12,14)
Fecal Coliform (CFU per 100 mL)[2]	Influent	749	1971	2628	605
	Effluent	813	133	4724	216

[2]Source: International Stormwater BMP Database, *Pollutant Category Summary: Fecal Indicator Bacteria*, December 2010

The section below provides an example of calculating pollutant removal effectiveness of two BMP options.

Example Calculation of Pollutant Removal Effectiveness

Compare the Fecal Coliform bacteria removal effectiveness of an extended dry detention BMP and a retention wet pond BMP for a medium density residential development. The TMDL for bacteria requires discharge from the drainage area to be below 23 Billion Colonies of Fecal Coliform. The drainage area to the BMP is 10 acres with percent imperviousness of 40-percent.

Step 1: Calculate the expected annual pollutant load from the development.

Use Equation A-2 to calculate the annual load of bacteria from the residential development. The EMC for bacteria from medium density residential land use is 48,100 CFU / 100 mL as shown in Table A-2. The annual precipitation total for Omaha is 30.26 inches (Table A-3).

$$L = 1.03x10^{-3} \times R \times C \times A$$

Where:

L	=	Annual load (Billion Colonies)
R	=	Annual runoff (inches)
C	=	EMC bacteria (CFU/100 ml)
A	=	Area (acres)
$1.03 * 10^{-3}$	=	Unit conversion factor

Use Equation A-4 to estimate the runoff coefficient for the residential development.

$$R_v = 0.05 + 0.9 \times .40 \ = 0.41$$

The runoff coefficient is used in Equation A-3 to calculate the annual runoff volume (R) in inches.

$$R = 30.26 \times \ 0.9 \times 0.41 = 11.2 \ inches$$

The runoff volume is used in Equation A-2 to estimate the bacteria load from the residential development.

$$R = 30.26 \times \ 0.9 \times 0.41 = 11.2 \ inches$$

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$$R = 30.26 \times \ 0.9 \times 0.41 = 11.2 \ inches$$

The runoff volume is used in Equation A-2 to estimate the bacteria load from the residential development.

$$L = 1.03x10^{-3} \times 11.2 \ inches \times 48{,}100 \frac{CFU}{100mL} \times 10 \ acres = 5{,}550 \ Billion \ Colonies$$

Step 2: Calculate the expected annual pollutant load from a dry pond BMP.

Use Equation A-6 to calculation the annual bacteria load from a dry pond BMP. The dry pond receives the full annual runoff volume from the residential development and the median effluent concentration from Table A-6 is used. Dry pond BMPs are expected to reduce annual runoff volumes between 26 and 43 percent. The median percent reduction in volume is 33 percent.

Calculate the outflow volume (O) as a portion of the inflow volume (R).

$$O\ inches = Inflow - Inflow\frac{Reduction\,\%}{100} = 11.2\ inches - 11.2\ inches\ \frac{33}{100} = 7.5\ inches$$

$$E = 1.03x10^{-3} \times 7.5\ inches \times 813\frac{CFU}{100mL} \times 10\ acres = 62.8\ Billion\ Colonies$$

Step 3: Calculate the expected annual pollutant load from a retention wet pond BMP.

Use Equation A-6 to calculate the annual bacteria load from a wet pond BMP. The wet pond receives the full annual runoff volume from the residential development and the median effluent concentration from Table A-6 is used. The wet pond is not expected to significantly reduce runoff volumes; therefore, the outflow volume is equal to the inflow volume.

$$E = 1.03x10^{-3} \times 11.2\ inches \times 133\frac{CFU}{100mL} \times 10\ acres = 15.3\ Billion\ Colonies$$

Step 4: Evaluate BMP alternatives.

The expected annual pollutant load from the extended dry detention basin is 62.8 Billion Colonies of Fecal Coliform bacteria which is greater than the limit of 23 Billion Colonies allowed by the TMDL. If an extended dry detention basin is used for this site, additional treatment of the extended detention basin effluent may be required to reduce Fecal Coliform bacteria to the TMDL limit. However, if a retention wet pond is used, the annual pollutant load is 15.3 Billion Colonies of Fecal Coliform which is below the TMDL limit. This analysis gives an estimate of pollutant loads. Actual pollutant loads can only be verified using monitoring data once the BMP has been constructed and is operating. This type of analysis is useful when planning BMP selection to determine the greatest likelihood of achieving downstream water quality goals.

Appendix 8-B

USEPA Class V Well Memorandum

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UNITED STATES ENVIRONMENTAL PROTECTION AGENCY
WASHINGTON, D.C. 20460

JUN 13 2008

OFFICE OF WATER

MEMORANDUM

SUBJECT: Clarification on which stormwater infiltration practices/technologies have the potential to be regulated as "Class V" wells by the Underground Injection Control Program

TO: Water Division Directors, Regions 1-10

FROM: Linda Boornazian, Director
Water Permits Division (MC 4203M)

Steve Heare, Director
Drinking Water Protection Division (MC 4606M)

Over the past several years stormwater infiltration has become an increasingly effective tool in the management of stormwater runoff. Although primary stormwater management responsibilities within EPA fall under the Clean Water Act (CWA), the infiltration of stormwater is, in some cases, regulated under the Safe Drinking Water Act (SDWA) with the goal of protecting underground sources of drinking water (USDWs). Surface and ground water protection requires effective integration between the overlapping programs. This memorandum is a step forward in that effort and is meant to provide clarification on stormwater implementation and green infrastructure, in particular under the CWA, which is consistent with the requirements of the SDWA's Underground Injection Control (UIC) Program.

In April 2007, EPA entered into a collaborative partnership with four national groups (the Association of State and Interstate Water Pollution Control Administrators, the Low Impact Development Center, the National Association of Clean Water Agencies, and the Natural Resources Defense Council) to promote green infrastructure as a cost-effective, sustainable, and environmentally friendly approach to stormwater management. The primary goals of this collaborative effort are to reduce runoff volumes and sewer overflow events through the use of green infrastructure wet weather management practices.

Within the context of this collaborative partnership, green infrastructure includes a suite of management practices that use soils and vegetation for infiltration, treatment, and evapotranspiration of stormwater. Rain gardens, vegetated swales, riparian buffers and porous pavements are all common examples of green infrastructure techniques that capture and treat stormwater runoff close to its source. Green infrastructure management practices typically do not include commercially manufactured or proprietary infiltration devices or other infiltration practices such as simple drywells, which do not provide for pre-treatment prior to infiltration.

The partnership is promoting green infrastructure as an effective approach to stormwater management because these practices are associated with a number of environmental benefits. In addition to reducing and delaying runoff volumes, green infrastructure approaches can also reduce pollutant levels in stormwater, enhance ground water recharge, protect surface water from stormwater runoff, increase carbon sequestration, mitigate urban heat islands, and increase wildlife habitat.

Given the multiple benefits that green infrastructure can provide, EPA and its partners have increased efforts to incorporate green infrastructure techniques into stormwater management strategies nationwide. In recent years, public support for these practices has gradually increased. For more information on green infrastructure, please visit www.epa.gov/npdes/greeninfrastructure.

There are cases where stormwater infiltration practices are regulated as Class V wells under the UIC program, and State and local stormwater managers report that some developers are hesitant to incorporate green infrastructure practices because they fear regulatory approvals will slow the process and increase costs. EPA believes those fears are unfounded and notes that most green infrastructure practices do not meet the Class V well definition and can be installed without regulatory oversight by the UIC Program. However, EPA remains committed to the protection of USDWs and emphasizes the need for UIC program compliance (per 40 CFR 144).

To provide clarification on which stormwater infiltration techniques meet EPA's UIC Class V well definition, EPA's Office of Water has developed the attached "Class V Well Identification Guide." State or Regional stormwater and nonpoint source control programs, developers, and other interested parties are requested to contact the State or Regional UIC Program Director with primary authority for the UIC Class V program when considering the use of practices that have been identified, or potentially identified, as Class V wells. UIC program managers should consider the proximity to sensitive ground water areas when looking at the suitability of stormwater infiltration practices. Depending on local conditions, infiltration without pretreatment may not be appropriate in areas where ground waters are a source of drinking water or other areas identified by federal, state, or local governments as sensitive ground water areas, such as aquifers overlain with thin, porous soils.

Please share this memo and the attached guide with your State and Regional stormwater, nonpoint source control, UIC and other ground water managers, as well as with appropriate green infrastructure contacts. These programs are encouraged to coordinate on stormwater management efforts when sensitive ground water issues arise.

Attachment

Underground Injection Control (UIC) Program Class V Well Identification Guide

This reference guide can be used to determine which stormwater infiltration practices/technologies have the potential to be regulated as “Class V” wells. Class V wells are wells that are not included in Classes I through IV. Typically, Class V wells are shallow wells used to place a variety of fluids directly below the land surface. By definition, a well is “any bored, drilled, driven shaft, or dug hole that is deeper than its widest surface dimension, or an improved sinkhole, or a subsurface fluid distribution system” and an “injection well” is a “well” into which “fluids” are being injected (40 CFR §144.3). Federal regulations (40 CFR §144.83) require all owners/operators of Class V wells to submit information to the appropriate regulatory authorities including the following:

1. Facility name and location
2. Name and address of legal contact
3. Ownership of property
4. Nature and type of injection well(s)
5. Operating status of injection well(s)

For more information on Class V well requirements, please visit http://www.epa.gov/safewater/uic/class5/comply_minrequirements.html. For more information on green infrastructure, please visit http://www.epa.gov/npdes/greeninfrastructure.

The stormwater infiltration practices/technologies in rows A through I below are generally not considered to be wells as defined in 40 CFR §144.3 because typically they are not subsurface fluid distribution systems or holes deeper than their widest surface dimensions. If these practices/technologies are designed in an atypical manner to include subsurface fluid distribution systems and/or holes deeper than their widest surface dimensions, then they may be subject to the Class V UIC regulations. The stormwater infiltration practices/technologies in rows J through K however, depending upon their design and construction probably would be subject to UIC regulations.

	Infiltration Practice/Technology	Description	Is this Practice/Technology Generally Considered a Class V Well?
A	Rain Gardens & Bioretention Areas	Rain gardens and bioretention areas are landscaping features adapted to provide on-site infiltration and treatment of stormwater runoff using soils and vegetation. They are commonly located within small pockets of residential land where surface runoff is directed into shallow, landscaped depressions; or in landscaped areas around buildings; or, in more urbanized settings, to parking lot islands and green street applications.	No.
B	Vegetated Swales	Swales (e.g., grassed channels, dry swales, wet swales, or bioswales) are vegetated, open-channel management practices designed specifically to treat and attenuate stormwater runoff. As stormwater runoff flows along these channels, vegetation slows the water to allow sedimentation, filtering through a subsoil matrix, and/or infiltration into the underlying soils.	No.
C	Pocket Wetlands & Stormwater Wetlands	Pocket/Stormwater wetlands are structural practices similar to wet ponds that incorporate wetland plants into the design. As stormwater runoff flows through the wetland, pollutant removal is achieved through settling and biological uptake. Several design variations of the stormwater wetland exist, each design differing in the relative amounts of shallow and deep water, and dry storage above the wetland.	No.
D	Vegetated Landscaping	Self-Explanatory.	No.
E	Vegetated Buffers	Vegetated buffers are areas of natural or established vegetation maintained to protect the water quality of neighboring areas. Buffer zones slow stormwater runoff, provide an area where runoff can infiltrate the soil, contribute to ground water recharge, and filter sediment. Slowing runoff also helps to prevent soil and stream bank erosion.	No

	Infiltration Practice/Technology	Description	Is this Practice/Technology Generally Considered a Class V Well?
F	Tree Boxes & Planter Boxes	Tree boxes and planter boxes are generally found in the right-of-ways alongside city streets. These areas provide permeable areas where stormwater can infiltrate. The sizes of these boxes can vary considerably.	No.
G	Permeable Pavement	Permeable pavement is a porous or pervious pavement surface, often built with an underlying stone reservoir that temporarily stores surface runoff before it infiltrates into the subsoil. Permeable pavement is an environmentally preferable alternative to traditional pavement that allows stormwater to infiltrate into the subsoil. There are various types of permeable surfaces, including permeable asphalt, permeable concrete and even grass or permeable pavers.	No.
H	Reforestation	Reforestation can be used throughout a community to reestablish forested cover on a cleared site, establish a forested buffer to filter pollutants and reduce flood hazards along stream corridors, provide shade and improve aesthetics in neighborhoods or parks, and improve the appearance and pedestrian comfort along roadsides and in parking lots.	No.
I	Downspout Disconnection	A practice where downspouts are redirected from sewer inlets to permeable surfaces where runoff can infiltrate.	In certain circumstances, for example, when downspout runoff is directed towards vegetated/pervious areas or is captured in cisterns or rain-barrels for reuse, these practices generally would not be considered Class V wells.
J	Infiltration Trenches	An infiltration trench is a rock-filled trench designed to receive and infiltrate stormwater runoff. Runoff may or may not pass through one or more pretreatment measures, such as a swale, prior to entering the trench. Within the trench, runoff is stored in the void space between the stones and gradually infiltrates into the soil matrix. There are a number of different design variations.	In certain circumstances, for example, if an infiltration trench is "deeper than its widest surface dimension," or includes an assemblage of perforated pipes, drain tiles, or other similar mechanisms intended to distribute fluids below the surface of the ground, it would probably be considered a Class V injection well.

	Infiltration Practice/Technology	Description	Is this Practice/Technology Generally Considered a Class V Well?
K	Commercially Manufactured Stormwater Infiltration Devices	Includes a variety of pre-cast or pre-built proprietary subsurface detention vaults, chambers or other devices designed to capture and infiltrate stormwater runoff.	These devices are generally considered Class V wells since their designs often meet the Class V definition of subsurface fluid distribution system.
L	Drywells, Seepage Pits, Improved Sinkholes.	Includes any bored, drilled, driven, or dug shaft or naturally occurring hole where stormwater is infiltrated.	These devices are generally considered Class V wells if stormwater is directed to any bored, drilled, driven shaft, or dug hole that is deeper than its widest surface dimension, or has a subsurface fluid distribution system.

Appendix 8-C

PCWP Stream Setback Policy

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PAPILLION CREEK WATERSHED
STORMWATER MANAGEMENT POLICIES

POLICY GROUP #3: LANDSCAPE PRESERVATION, RESTORATION, AND CONSERVATION

ISSUE: Natural areas are diminishing, and there is a need to be proactive and integrate efforts directed toward providing additional landscape and green space areas with enhanced stormwater management through restoration and conservation of stream corridors, wetlands, and other natural vegetation.

"ROOT" POLICY: Utilize landscape preservation, restoration, and conservation techniques to meet the multi-purpose objectives of enhanced aesthetics, quality of life, recreational and educational opportunities, pollutant reduction, and overall stormwater management.

SUB-POLICIES:

1) Incorporate stormwater management strategies as a part of landscape preservation, restoration, and conservation efforts where technically feasible.
2) Define natural resources for the purpose of preservation, restoration, mitigation, and/or enhancement.
3) For new development or significant redevelopment, provide a creek setback of 3:1 plus 50 feet along all streams as identified in the Papillion Creek Watershed Management Plan and a creek setback of 3:1 plus 20 feet for all other watercourses.
4) All landscape preservation features as required in this policy or other policies, including all stormwater and LID strategies, creek setbacks, existing or mitigated wetlands, etc., identified in new or significant redevelopment shall be placed into an out lot or within public right of way or otherwise approved easement.

<u>REFERENCE INFORMATION</u>

DEFINITIONS

1) <u>Creek Setback.</u> See Figure 1 below and related definitions in Policy Group #5. A setback area equal to three (3) times the channel depth plus fifty (50) feet (3:1 plus 50 feet) from the edge of low water on both sides of channel shall be required for any above or below ground structure exclusive of bank stabilization structures, poles or sign structures adjacent to any watercourse defined within the watershed drainage plan. Grading, stockpiling, and other construction activities are not allowed within the setback area and the setback area must be protected with adequate erosion controls or other Best Management Practices, (BMPs). The outer 30 feet adjacent to the creek setback limits may be credited toward meeting the landscaping buffer and pervious coverage requirements.

   A property can be exempt from the creek setback requirement upon a showing by a licensed professional engineer or licensed landscape architect that adequate bank stabilization structures or slope protection will be installed in the construction of said structure, having an estimated useful life equal to that of the structure, which will provide adequate erosion control conditions coupled with adequate lateral support so that no portion of said structure adjacent to the stream will be endangered by erosion or lack of lateral support. In the event that the structure is adjacent to any stream which has been channelized or otherwise improved by any agency of government, then such certificate providing an exception to the creek setback requirement may take the form of a certification as to the adequacy and protection of the improvements installed by such governmental agency. If such exemption is granted, applicable rights-of-way must be provided and a minimum 20 foot corridor adjacent thereto.

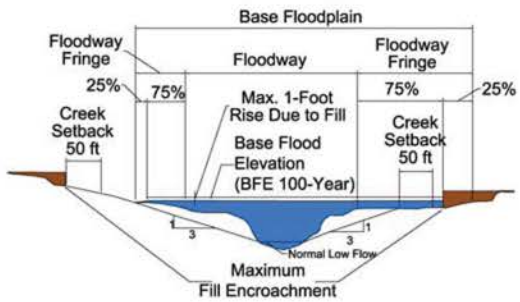

Figure 1 – Floodway Fringe Encroachment and Creek Setback Schematic

DEFINITIONS

1) Base Flood. The flood having a one percent chance of being equaled or exceeded in magnitude in any given year (commonly called a 100-year flood). *[Adapted from Chapter 31 of Nebraska Statutes]*
2) Floodway. The channel of a watercourse and the adjacent land areas that are necessary to be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation more than one foot. *[Adapted from Chapter 31 of Nebraska Statutes].* The Federal Emergency Management Agency (FEMA) provides further clarification that a floodway is the central portion of a riverine floodplain needed to carry the deeper, faster moving water.
3) Floodway Fringe. That portion of the floodplain of the base flood, which is outside of the floodway. *[Adapted from Chapter 31 of Nebraska Statutes]*
4) Floodplain. The area adjoining a watercourse, which has been or may be covered by flood waters. *[Adapted from Chapter 31 of Nebraska Statutes]*
5) Watercourse. Any depression two feet or more below the surrounding land which serves to give direction to a current of water at least nine months of the year and which has a bed and well-defined banks. *[Adapted from Chapter 31 of Nebraska Statutes]*
6) Low Chord Elevation. The bottom-most face elevation of horizontal support girders or similar superstructure that supports a bridge deck.
7) Updated Flood Hazard Maps. The remapping of flooding sources within the Papillion Creek Watershed where Digital Flood Insurance Rate Maps (DFIRMs) are based on 2004 or more recent conditions hydrology and full-build out conditions hydrology. West Papillion Creek and its tributaries are currently under remapping and will become regulatory in 2009. Updating flood hazard maps for Big Papillion Creek and Little Papillion Creek are planned to be completed in the future.
8) New Development. New development shall be defined as that which is undertaken to any undeveloped parcel that existed at the time of implementation of this policy.

PAPILLION CREEK WATERSHED
STORMWATER MANAGEMENT POLICIES

POLICY GROUP #5: FLOODPLAIN MANAGEMENT

ISSUE: Continued and anticipated development within the Papillion Creek Watershed mandates that holistic floodplain management be implemented and maintained in order to protect its citizens, property, and natural resources.

"ROOT" POLICY: Participate in the FEMA National Flood Insurance Program, update FEMA floodplain mapping throughout the Papillion Creek Watershed, and enforce floodplain regulations to full build-out, base flood elevations.

SUB-POLICIES:

1) Floodplain management coordination among all jurisdictions within the Papillion Creek Watershed and the Papio-Missouri River Natural Resources District (P-MRNRD) is required.
2) Flood Insurance studies and mapping throughout the Papillion Creek Watershed shall be updated using current and full-build out conditions hydrology.
3) Encroachments for new developments or significant redevelopments within floodway fringes shall not cause any increase greater than one (1.00) foot in the height of the full build-out base flood elevation using best available data.
4) Filling of the floodway fringe associated with new development within the Papillion Creek System shall be limited to 25% of the floodway fringe in the floodplain development application project area, unless approved mitigation measures are implemented. The remaining 75% of floodway fringe within the project area shall be designated as a floodway overlay zone. For redevelopment, these provisions may be modified or waived in whole or in part by the local jurisdiction.
5) The low chord elevation for bridges crossing all watercourses within FEMA designated floodplains shall be a minimum of one (1) foot above the base flood elevation for full-build out conditions hydrology using best available data.
6) The lowest first floor elevation of buildings associated with new development or significant redevelopment that are upstream of and contiguous to regional dams within the Papillion Creek Watershed shall be a minimum of one (1) foot above the 500-year flood pool elevation.

REFERENCE INFORMATION

DEFINITIONS (See Figure 1 below and related definitions in Policy Group #3: Landscape Preservation, Restoration, and Conservation).

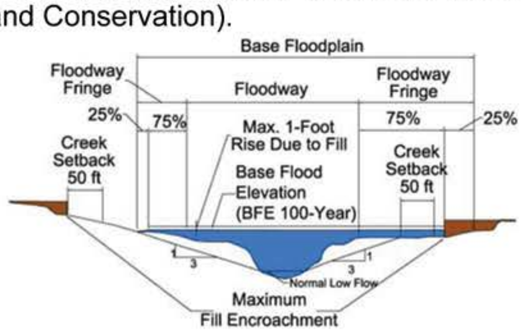

Figure 1 – Floodway Fringe Encroachment and Creek Setback Schematic

PAPILLION CREEK WATERSHED
STORMWATER MANAGEMENT POLICIES

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4) Floodplain. The area adjoining a watercourse, which has been or may be covered by flood waters. *[Adapted from Chapter 31 of Nebraska Statutes]*
5) Watercourse. Any depression two feet or more below the surrounding land which serves to give direction to a current of water at least nine months of the year and which has a bed and well-defined banks. *[Adapted from Chapter 31 of Nebraska Statutes]*
6) Low Chord Elevation. The bottom-most face elevation of horizontal support girders or similar superstructure that supports a bridge deck.
7) Updated Flood Hazard Maps. The remapping of flooding sources within the Papillion Creek Watershed where Digital Flood Insurance Rate Maps (DFIRMs) are based on 2004 or more recent conditions hydrology and full-build out conditions hydrology. West Papillion Creek and its tributaries are currently under remapping and will become regulatory in 2009. Updating flood hazard maps for Big Papillion Creek and Little Papillion Creek are planned to be completed in the future.
8) New Development. New development shall be defined as that which is undertaken to any undeveloped parcel that existed at the time of implementation of this policy.

BASIC FEMA REQUIREMENTS

On March 1, 2003, FEMA became part of the U.S. Department of Homeland Security (DHS). In order for a community to participate in the FEMA National Flood Insurance Program, it must first define base flood elevations and adopt a floodway for all its major streams and tributaries. Once a community adopts its floodway, the requirements of *44 CFR 60.3(d)* must be fulfilled. The key concern is that each project in the floodway must receive an encroachment review; i.e., an analysis to determine if the project will increase flood heights or cause increased flooding downstream. Note that the FEMA regulations call for preventing any increase in flood heights. Projects, such as filling, grading or construction of a new building, must be reviewed to determine whether they will obstruct flood flows and cause an increase in flood heights upstream or adjacent to the project site. Further, projects, such as grading, large excavations, channel improvements, and bridge and culvert replacements should also be reviewed to determine whether they will remove an existing obstruction, resulting in increases in flood flows downstream. *[Adapted from Federal Emergency Management Agency guidance]*

Appendix 8-D
Derivation of Peak Flow Rate for the Water Quality Storm

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Appendix 8-D
Derivation of Peak Flow Rate for the Water Quality Storm

The current policy outlined in the City of Omaha Post-Construction Stormwater Management Planning Guidance for selecting a design storm for flow-through BMPs is:

"For stormwater BMPs that provide treatment based on a flow rate, the Designer may submit calculations that demonstrate water quality flow rates that are equivalent to treating the first one-half inch (0.5 inches) of stormwater runoff. On sites where the Rational Method is suitable and the time of concentration is 5 minutes, designers may estimate i using the 1-yr IDF curve with 20-minute duration. Designers may also use WinTR-55 to estimate flow rate, however, the model must show a correlation to a 0.5 inches runoff depth in the output report. Proprietary stormwater BMPs shall be pre-approved for use by the City of Omaha Public Works Department."

The variable "i" is the rainfall intensity in inches per hour for storms with duration equal to the time of concentration of the site. Use of the Rational Method requires the designer to select a value for the runoff coefficient (C) which represents a ratio of runoff to rainfall for future land-use conditions. BMPs sized using the City's volume criteria are sized with 0.5 inches of runoff regardless of the future land use conditions.

To provide an approach for sizing of flow-through BMPs that is consistent with the City's volume design criteria, CDM Smith's NetSTORM program was run using approximately 61 years of rainfall data collected within the Omaha Region. The results of the NetSTORM analysis were used to create capture curves for a BMP with a 24-hour draindown time (Figure D-1).

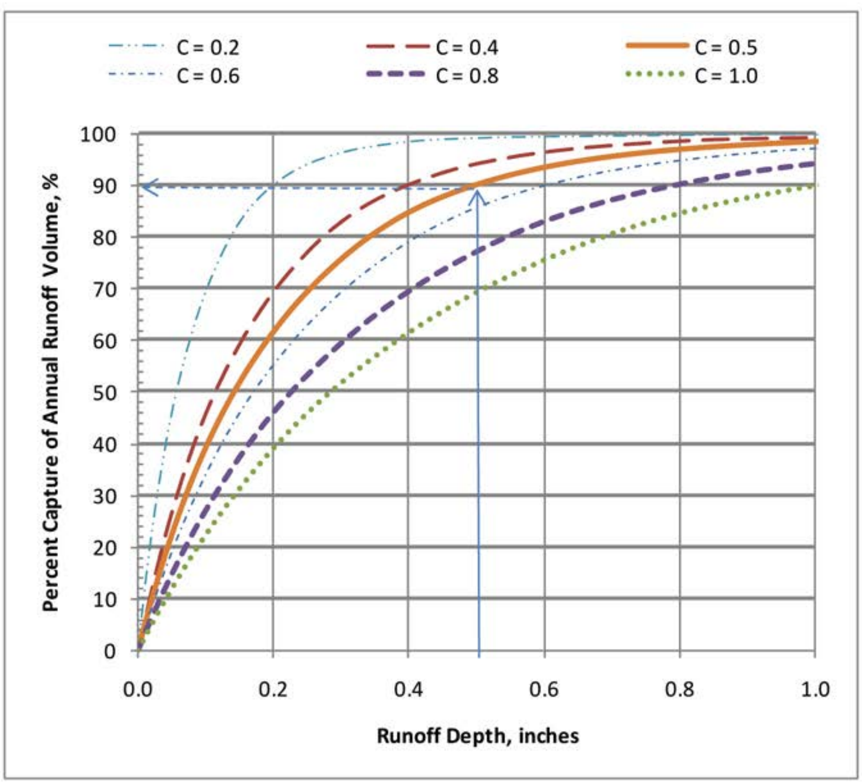

Figure D-1 Capture Curves for Omaha, Nebraska for BMP with 24-hour draindown time.

Figure D- 1 shows that a BMP sized for 0.5 inches of runoff and a 24-hour draindown captures and treats between 70 to greater than 95 percent of annual runoff events depending upon the runoff coefficient (C) applied to the development. If we apply a 90-percent treatment criteria to flow through BMPs (equal to a site with C = 0.5 capturing 0.5 inches of runoff), the 90-percent intensity can be used to calculate a peak flow rate per acre required for treatment.

NetSTORM was used to separate the rainfall depths into event of 1-hour, 6-hour, and 24-hour duration and the 90-percent rainfall intensity was calculated. Figure D-2 shows the resulting plot for the 90-percent along with that of the 1-year return interval storm event. Values for durations other than 1-, 6-, and 24-hours were estimated using the slope of the 1-year intensity curve.

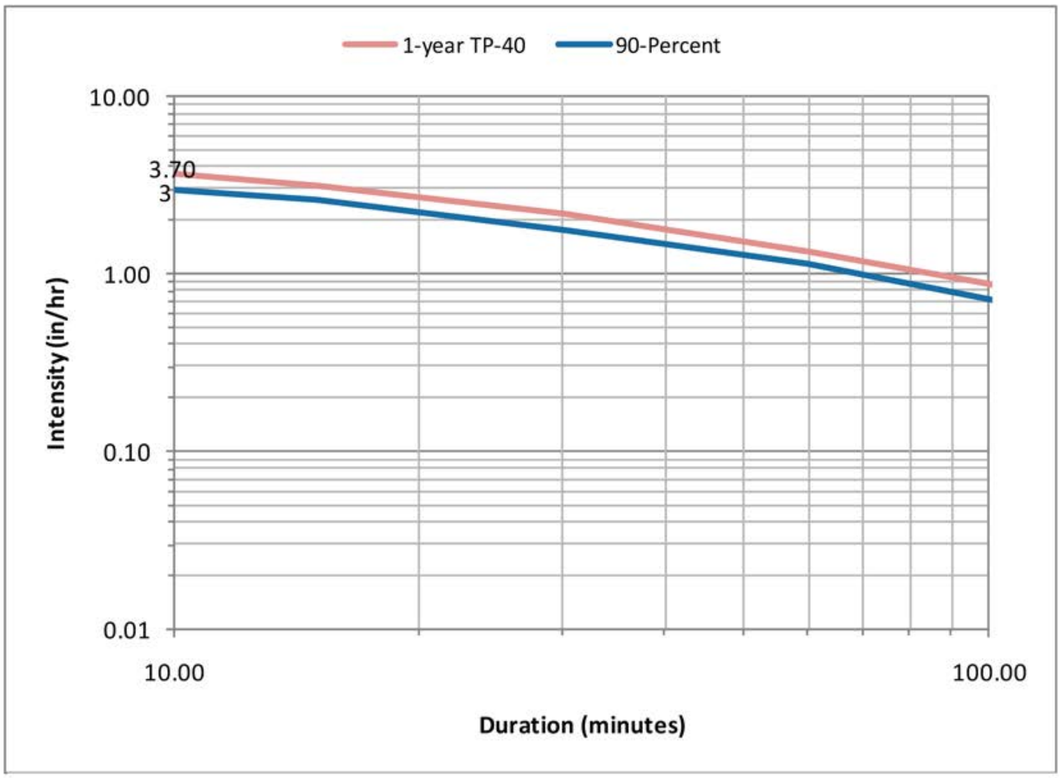

Figure D-2 Intensity-Duration-Frequency (IDF) Curves for Omaha, Nebraska. (Adapted from TP-40).

The 90-percent curve in Figure D-2 can be used to estimate the rainfall intensity to be used in the Rational equation for flow-through BMPs. The 90-percent rainfall intensities are less than those for the 1-year return interval; however, they can be related back to a 0.5 inches runoff capture, and the 90-percent intensity curve can be used for multiple durations.

To remain consistent with the City's ordinance on the 0.5 inches of runoff, CDM Smith calculated the peak flow per acre of drainage area for a variety of storm durations based on a C = 0.5 as shown in Figure D-3. The graph in Figure D-3 can be used to determine a required peak flow rate for flow through BMPs. Using Figure D-3 to determine a peak flow rate is consistent with the BMP sizing criteria of capturing 0.5 inches of runoff, regardless of subarea runoff coefficient.

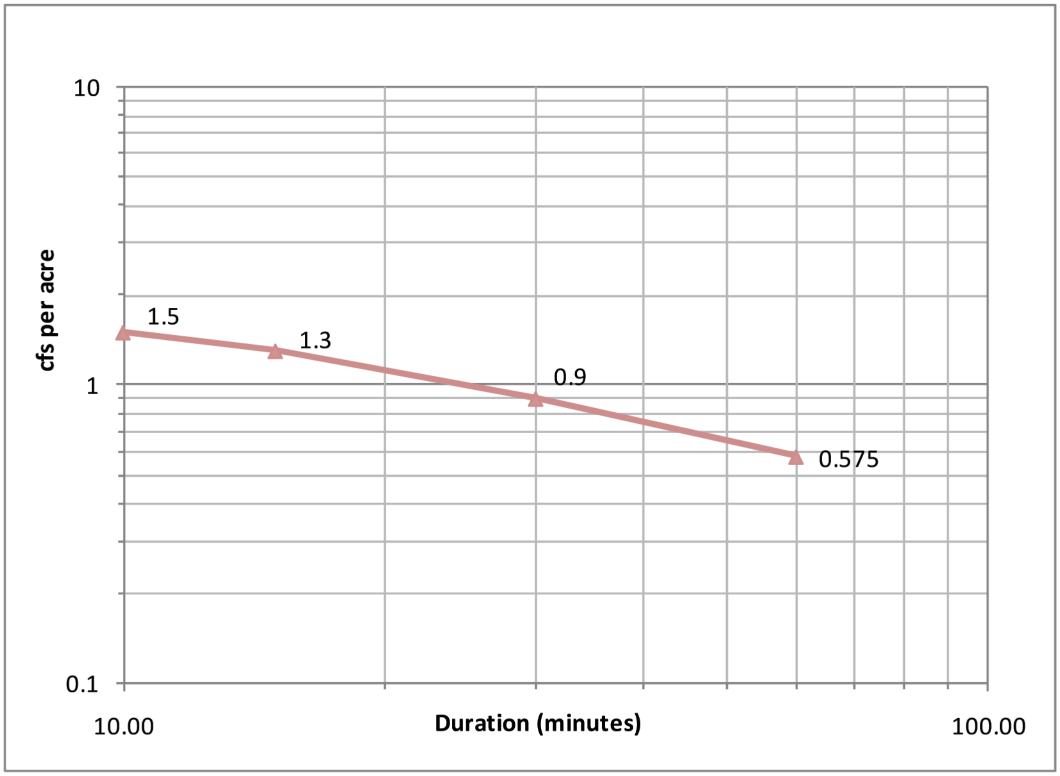

Figure D-3 CFS Per Acre for C=0.5 and 90-Percent Intensity

Alternatively, flow-through BMPs perform best when applied to small drainage areas that have a small time of concentration. Therefore, the City could decide to simplify the design standard by choosing an estimated time of concentration and design duration, for example: 10 minutes, and require all flow-through BMPs be sized using the corresponding cubic feet per second (cfs) per acre (1.5 cfs per acre for 10-minute duration).

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Appendix 8-E

Background Information on Cascading Planes

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Appendix 8-E
Background Information on Cascading Plane

When stormwater that is generated as runoff from impervious areas is conveyed through pervious areas (through swales, strips, turf areas, etc.) the runoff volume and peak flow rate is reduced[1]. When pervious areas receive runoff from impervious areas the concept is known as cascading planes.[2] When pervious areas receive runoff from impervious areas the concept is known as cascading planes.[3] Figure E-1 provides an illustration of the cascading planes concept.

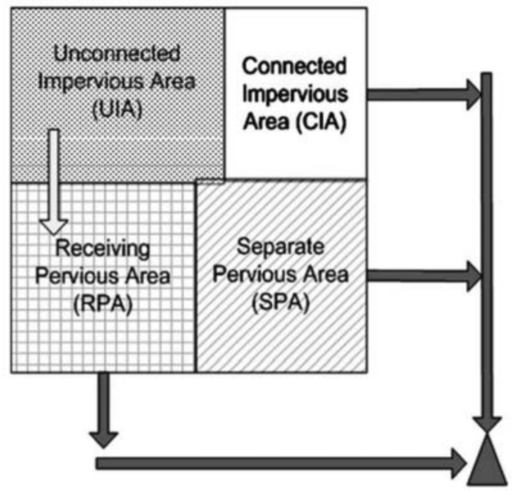

Figure E-1 Schematic of Cascading Planes Concept. Source: Guo, et.al. Incentive Index Developed to Evaluate Storm-Water Low-Impact Designs. ASCE Journal of Environmental Engineering December 2010.

Effective impervious (I_E) can be used to represent the runoff volume reduction due to cascading planes. The Urban Drainage and Flood Control District of Denver uses the concept of effective impervious to account for runoff volumes that are reduced by using LID conveyance BMPs such as grass swales, vegetated buffers, disconnection of roof drains and other impervious areas draining to pervious areas.[4] The effective impervious area concept is described in Incentive Index Developed to Evaluate Storm-Water Low-Impact Designs by Guo, et.al.[5]

The paragraphs below describe using the relationships described in Guo, 2010 as a method of estimating volume reduction for site developments that use LID conveyance BMPs in the City of Omaha. The reduction in the design volume of the downstream structural BMP is based on the idea that a portion of the 0.5 inches of runoff is "captured and controlled" within the conveyance BMP or pervious area. The amount controlled is calculated using the following equation:

Depth of Runoff Controlled, inches = 0.5 - 0.5 x K   Equation E-1

Where:

K	=	$e^{[-0.0052(100-I_A)f/i]}$ = pavement-area-reduction factor (PARF), equation provided by Guo, 2010.
I_A	=	area-weighted imperviousness percent for cascading plane = UCIA / (UCIA + RPA)
UCIA	=	unconnected impervious area, acres
RPA	=	receiving pervious areas, acres
f	=	infiltration rate on the pervious surface, in/hr
i	=	average rainfall intensity, in/hr = 0.6 inches per hour for the City of Omaha (Figure 3-1 from Urban Drainage and Flood Control District Urban Storm Drainage Criteria Manual Volume 3, November 2010)

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[1] Guo, et.al. Incentive Index Developed to Evaluate Storm-Water Low-Impact Designs. ASCE Journal of Environmental Engineering December 2010.
[2] Guo, et.al. Incentive Index Developed to Evaluate Storm-Water Low-Impact Designs. ASCE Journal of Environmental Engineering December 2010.
[3] Guo, et.al. Incentive Index Developed to Evaluate Storm-Water Low-Impact Designs. ASCE Journal of Environmental Engineering December 2010.
[4] Urban Drainage and Flood Control District Urban Storm Drainage Criteria Manual, Vol. 3 November 2010, page 3-15
[5] Guo, et.al. Incentive Index Developed to Evaluate Storm-Water Low-Impact Designs. ASCE Journal of Environmental Engineering December 2010.

Table E-1 shows the results of Equation E-1 for varying percent imperviousness of cascading planes and soil infiltration rates. An example application using Table E-1 is provided below.

Table E-1
Depth of Runoff Controlled (in inches) by Cascading Planes

Percent Impervious of Cascading Planes, %	f, in/hr[1]								
	0.12	0.16	0.26	0.34	0.43	0.83	1.04	1.92	5.85
	Soil Texture Classification								
	Clay	Sandy Clay	Clay Loam	Sandy Clay Loam	Loam	Silt Loam	Sandy Loam	Loamy Sand	Sand
1	0.049	0.064	0.100	0.127	0.154	0.255	0.295	0.404	0.497
10	0.045	0.059	0.092	0.116	0.142	0.238	0.278	0.388	0.495
20	0.040	0.053	0.082	0.105	0.129	0.219	0.257	0.368	0.491
30	0.035	0.046	0.073	0.093	0.115	0.198	0.234	0.344	0.486
40	0.030	0.040	0.063	0.081	0.100	0.175	0.209	0.316	0.476
50	0.025	0.033	0.053	0.068	0.085	0.151	0.181	0.282	0.460
60	0.025	0.027	0.043	0.056	0.069	0.125	0.151	0.243	0.434
70	0.015	0.020	0.033	0.042	0.053	0.097	0.118	0.196	0.391
80	0.010	0.014	0.022	0.029	0.036	0.067	0.082	0.142	0.319
90	0.005	0.007	0.011	0.015	0.018	0.035	0.043	0.077	0.199
100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

[1]Values for conveyance-based BMPs from Urban Drainage and Flood Control District Urban Storm Drainage Criteria Manual Volume 3, page 3-17

Example Application

Consider a 10-acre site with future impervious area of 70-percent. The ordinance requires the capture and treatment of the first 0.5 inches of runoff equating to a WQCV of 5 acre-inches or 0.417 acre-feet.

Scenario 1- Traditional Development

The 7 acres of impervious area is directed to the storm drain. There is no contribution of runoff from the impervious areas to the pervious areas. In this scenario, the design water quality volume for the site is 0.417 acre-feet as there is no allowance for reducing runoff volume.

Scenario 2- LID Conveyance Development

In Scenario 2, 6 of the 7 acres of impervious area is directed to the storm drain. The remaining 1 acre of impervious area flows to one acre of turf lawn on sandy-clay-loam soil with infiltration rates of 0.34 inches per hour. The volume runoff from the 1 acre of impervious area which flows to the pervious areas is reduced. First the percent imperviousness of the cascading planes is calculated.

I_A = UCIA / (UCIA + RPA) = (1 acre / (1 acre + 1 acres) = 50%

Then, using Table E-1, the WQCV allowance for I_A = 50 percent and f = 0.34 inches per hour is 0.068 inches.

The design WQCV is reduced to 0.432 inches (0.5 inches – 0.068 inches) for the 2 acres of cascading planes and the remainder of the site does not qualify for a reduction in WQCV. If a structural BMP is placed downstream of the cascading planes, then it will be sized using the 0.432 inches of runoff. If a structural

BMP is placed at the downstream end of the entire site, the WQCV allowance applies only to the cascading planes portion. Therefore, the design volume for the BMP would be the area weighted total calculated as:

$$WQCV_{site} = 0.432\ inches * \frac{1\ ft}{12\ inches} * 2\ acres + 0.5\ inches * \frac{1\ ft}{12\ inches} * 8\ acres = 0.405\ acre - feet$$

Table E-2
WQCV Allowance Summary

Scenario	Area	Description	Site WQCV
Scenario 1	10 Acres	Typical Site Design – 70% Impervious	0.417 acre-feet
Scenario 2	10 Acres	LID Design – 70% Impervious 1 acre of UCIA drains to 1 acre of RPA	0.405 acre-feet

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Appendix 8-F

Example Bioretention Facility Specifications

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Appendix 8-F
Example Bioretention Facility Specification

BIORETENTION GARDENS

9003.1 Description

Bioretention gardens are small landscaped basins intended to provide water quality management by filtering stormwater runoff before release into storm drain systems or natural channels. This work shall consist of installing bioretention gardens as specified in the Contract Documents, including all materials, equipment, labor and services required to perform the work.

9003.2 Materials

A. Bioretention Soil Mixture: The Bioretention Soil Mixture (BSM) is composed of the following materials:

Item	Composition By Volume	Reference
Organic Compost	50%	See below.
Sand	50%	ASTM C33 Fine Aggregate

The BSM shall be a uniform mix, free of plant residue, stones, stumps, roots or other similar objects larger than two inches excluding mulch. No other materials or substances shall be mixed or dumped within the bioretention garden that may be harmful to plant growth, or prove a hindrance to the planting or maintenance operations.

B. Organic Compost: The compost used in the BSM and soil conditioned areas shall be derived from plant material, and the result of biological degradation and transformation of plant derived materials under conditions that promote anaerobic decomposition. The material shall be well composted, free of viable weed seeds, and stabilized with regard to oxygen consumption and carbon dioxide generation. The compost shall have a moisture content that has no visible free water or dust produced when handling the material. It shall meet the criteria presented below as reported by the U.S. Composting Council STA Compost Technical Data Sheet provided by the vendor. OmaGro is a locally produced compost product that is acceptable for use in bioretention gardens.

Compost Criteria
One hundred percent of the material must pass through a half inch screen
The pH of the material shall be between 6 and 8
Manufactured inert material (plastic, concrete, ceramics, metal, etc.) shall be less than 1.0% by weight.
Organic matter should be between 35 and 65 %
Soluble salt content shall be less than 6.0 mmhos/cm
Maturity should be greater than 80 %
Stability shall be 7 or less
Carbon/nitrogen ratio shall be less than 25:1
Trace metal test result = "pass"
The compost must have a dry bulk density ranging from 40 to 50 lbs/ft^3.

C. Other Materials

Material	Specification
No. 57 Aggregate	ASTM D448
No. 7 Aggregate	ASTM D448
4-inch HDPE Plastic Pipe Underdrain	AASHTO M252
Geotextile Fabric	AASHTO M288
Mulch, 2x Shredded Hardwood Bark	See below
Water	See below.

Shredded Hardwood Mulch: Shredded hardwood mulch shall be aged a minimum of 6 months and consist of the bark and wood (50/50) from hardwood trees which has been milled and screened to a maximum 4-inch particle size and provide a uniform texture free from sawdust, clay, soil, foreign materials, and any artificially introduced chemical compounds that would be detrimental to plant or animal life.

Aggregate: No. 7 and No. 57 Aggregate shall be double-washed to reduce suspended solids and potential for clogging. The aggregate shall be placed as shown in the Contract Drawings.

Water: Water used in the planting, establishing, or caring for vegetation shall be free from any substance that is injurious to plant life.

9003.3 Construction

The underdrain or BSM shall not be placed until all contributing drainage areas are permanently stabilized against erosion and sedimentation as shown on the Contract Plans and to the satisfaction of the Engineer. Any discharge of sediment that affects the performance of the cell will require reconstruction of the cell to restore its defined performance. No heavy equipment shall operate within the perimeter of a bioretention garden during underdrain placement, backfilling, planting, or mulching of the garden.

A. Excavation: If the bioretention garden is to be used as a temporary sediment basin the bioretention garden shall be excavated to the dimensions, side slopes, and ***6 inches above*** the bottom of the BSM elevations shown on the Contract Plans. Any sediment from construction operations deposited in the bioretention garden shall be completely removed from the garden after all vegetation, including landscaping within the drainage area of the bioretention garden, has been established. The excavation limits shall then be final graded to the dimensions, side slopes, and ***final*** elevations shown on the Contract Plans. Excavators and backhoes, operating on the ground adjacent to the bioretention garden, shall be used to excavate the garden if possible, by <u>low ground-contact pressure equipment</u> or, if approved by the engineer, by excavators and/or backhoes operating on the ground adjacent to the bioretention garden. Low ground-contact pressure equipment is preferred on bioretention gardens to minimize disturbance to established areas around perimeter of cell. No heavy equipment shall be used within the perimeter of the bioretention garden before, during, or after the placement of the BSM.

Excavated materials shall be removed from the bioretention garden site. Excavated materials shall be used or disposed of in conformance with the project specifications.

B. Roto-tilling: After placing the underdrain and aggregate and before the BSM, the bottom of the excavation shall be roto-tilled to a minimum depth of 6 inches to alleviate any compaction of the garden bottom. Any substitute method for roto-tilling must be approved by the Engineer prior to use. Any ponded water shall be removed from the bottom of the garden and the soil shall be friable before roto-tilling. The roto-tilling shall not be done where the soil supports the aggregate bed underneath the "Underdrain for Bioretention". (See "Underdrain for Bioretention" specifications below.)

C. Underdrain for bioretention: The underdrain system, aggregate bed, and geotextile fabric shall be placed according to dimensions shown on the Contract Plans.

D. Observation wells/cleanouts of 4-inch non-perforated HDPE pipe shall be placed vertically in the bioretention garden as shown on the Contract Plans. The wells/cleanouts shall be connected to the perforated underdrain with the appropriate manufactured connections as shown on the Contract Plans. The wells/cleanouts shall extend 6 inches above the top elevation of the bioretention garden mulch, and shall be capped with a screw cap.

E. Placement of the BSM: The BSM shall be placed and graded using low ground-contact pressure equipment or, if approved by the engineer, by excavators and/or backhoes operating on the ground adjacent to the bioretention garden. Low ground-contact pressure equipment is preferred on bioretention gardens to minimize disturbance to established areas around perimeter of cell. No heavy equipment shall be used within the perimeter of the bioretention garden before, during, or after the placement of the BSM. The BSM shall be placed in horizontal lifts in depths not exceeding 12 inches for the entire area of the bioretention garden. The BSM shall be pre-mixed, with a moisture content low enough to prevent clumping and compaction during placement. If the BSM becomes contaminated during the construction of the garden, the contaminated material shall be removed and replaced with uncontaminated material at the Contractor's expense. Final grading of the BSM shall be performed after a 24-hour settling period. Upon final grading the surface of the BSM shall be roto-tilled to a depth of 6". Final elevations shall be within 2 inches of elevations shown on the Contract Plans.

F. Soil Conditioning of Ponding Area: Ensure there is no standing water within the ponding area prior to beginning the soil conditioning process to avoid further compacting soils. Existing vegetation, including turf, shall be removed and the ground shall be tilled to a minimum depth of 6 inches. A 3-inch deep layer of specified compost shall be placed on top of the tilled ground and tilled into a depth of 6 inches of existing soil. Fine grading of the site shall be completed with a minimum number of equipment passes (no more than two (2) passes) to reduce the potential for soil compaction. Finalizing all preliminary critical spot elevation, slopes and positive drainage criteria for the site shall be completed as much as possible prior to finish grading in order to ensure that equipment compaction is minimized after soil is worked an amended. Soil shall be firmed using one pass of a 50-pound roller if vegetative cover will be seeded or plugged to help ensure successful plant establishment. Vegetative cover shall be established immediately after finish grading and erosion shall be prevented during establishment, including but not limited to installing erosion control blankets, silt fence or straw waddles. Vegetation may be sodded, seeded, or plugged. For seeding or plugging, all standard procedures shall be followed for the appropriate mulching of bare soil surface areas until vegetation is fully established.

G. Mulching: Once grading is complete, the entire surface of the BSM shall be mulched to a uniform thickness of 3 inches. Mulching shall be complete within 24 hours to reduce the potential of silt accumulation on the surface. Well aged shredded hardwood bark mulch is the only acceptable mulch. Mulching shall be done immediately after grading to reduce potential of any silt accumulation on the surface.

H. Plant Installation: Trees, shrubs, and other plant materials specified for Bioretention Gardens shall be planted as specified in the Contract Plans and applicable landscaping standards with the exception that pesticides, herbicides, and fertilizer shall not be applied during planting under any circumstances. Furthermore, pesticides, fertilizer, and any other soil amendments shall not be applied to the bioretention garden during landscape construction, plant establishment, or maintenance.

9003.4 Method of Measurement

Bioretention gardens will be measured by the square foot and will be paid for at the Contract Unit Price.

9003.5 Basis of Payment

The payment will be full compensation for all material, labor, equipment, tools, and incidentals necessary to satisfactorily complete the work. Biological plantings will be paid for separately under other items of the contract.

References:

MARC and APWA. 2009. Manual of Best Management Practices for Stormwater Quality.

Wisconsin Department of Natural Resources (WDNR).November 2010. Bioretention for Infiltration (1004). http://dnr.wi.gov/topic/stormwater/documents/Bioretention1004.pdf