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.

Improvement Plans 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. Concept plans should illustrate how the site layout addresses critical objectives and performance standards. Final improvement plans 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.

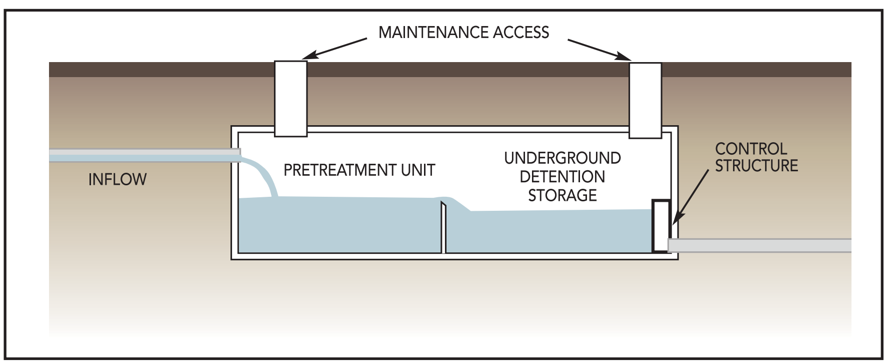


Figure 8-2 Schematic of Reinforced Concrete Structure used for Underground Detention

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.

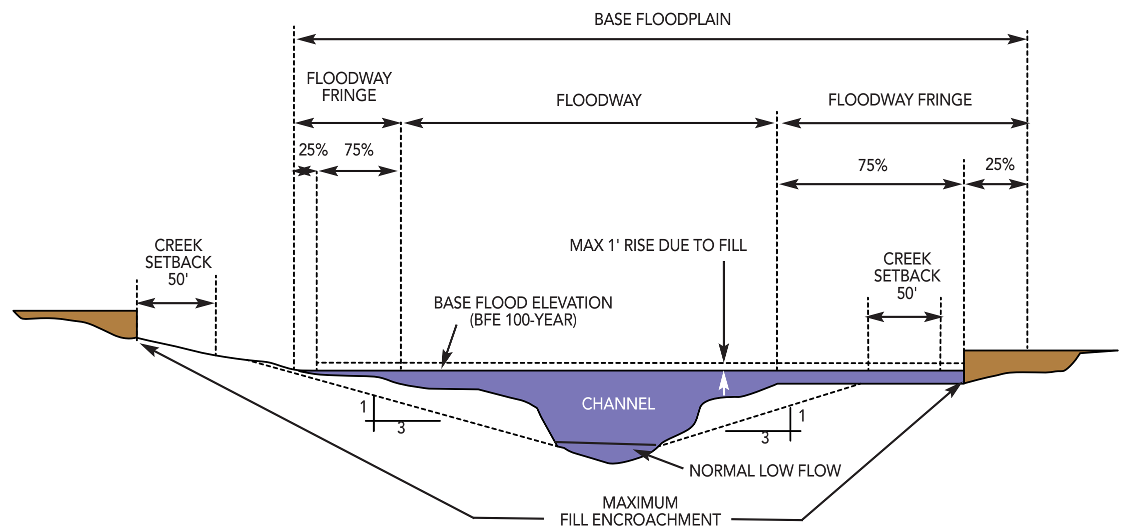
The PCWP stream setback policy includes:

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).

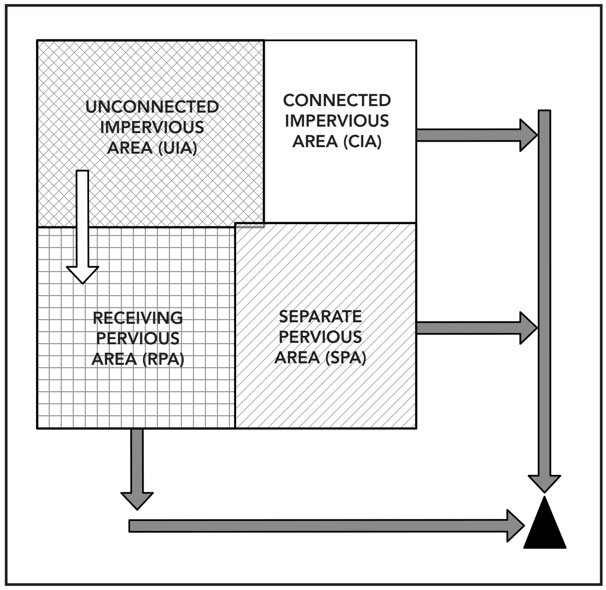


Figure 8-4 Schematic of Cascading Planes Concept. Source: *Incentive Index Developed to Evaluate Storm-Water Low-Impact Designs*. (Guo, et.al., ASCE Journal of Environmental Engineering, December 2010.)

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.

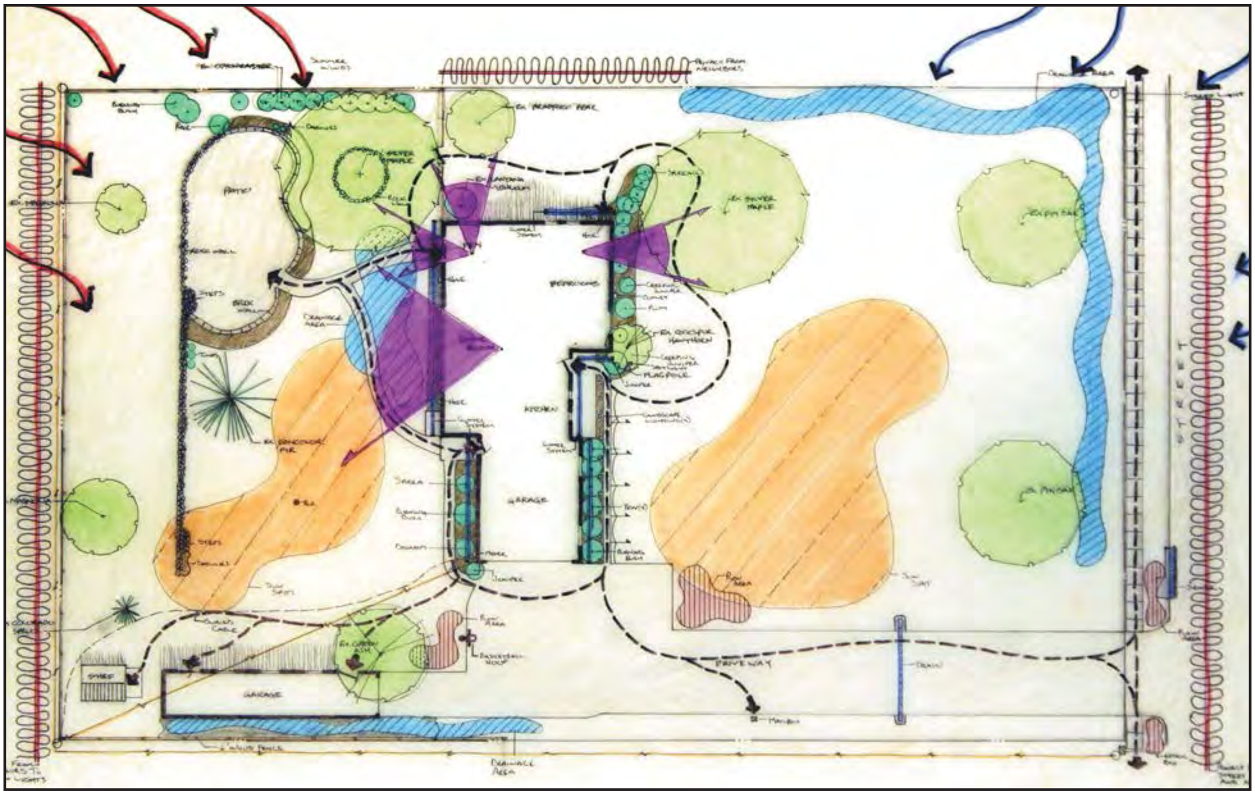


Figure 8-6 Example Placement of Rain Gardens on a Residential Lot

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.

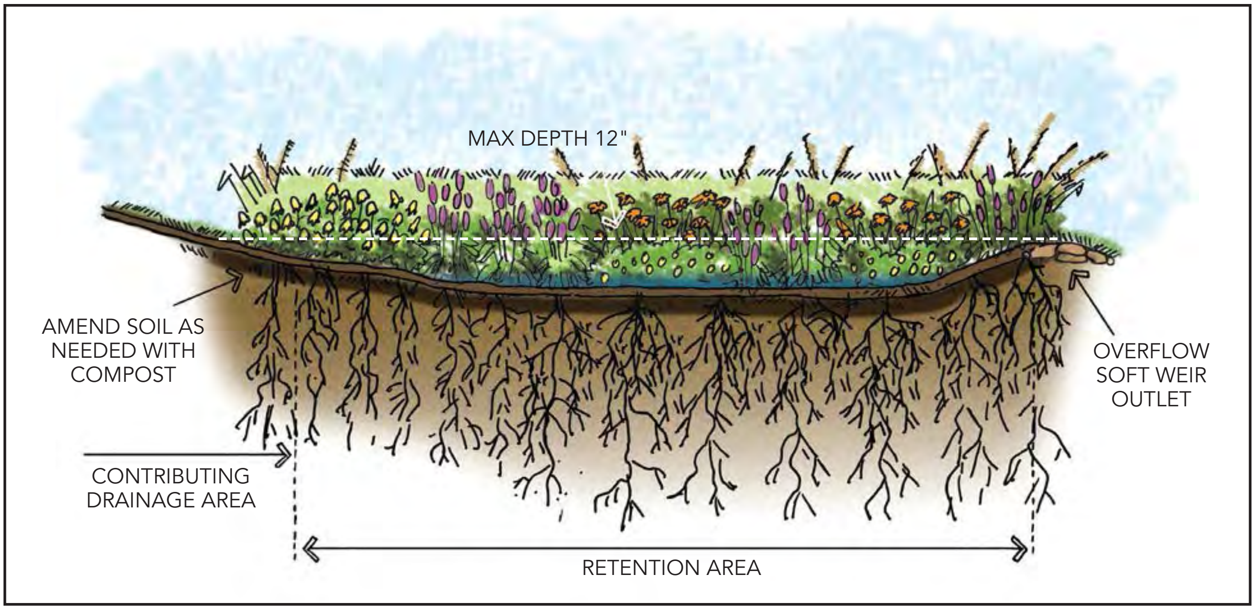


Figure 8-7 Rain Garden Cross Section

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

Roof Area (square feet)	Rainfall (inches)									
	0.1	0.2	0.3	0.4	0.5[1]	0.6	0.7	0.8	0.9	1
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.

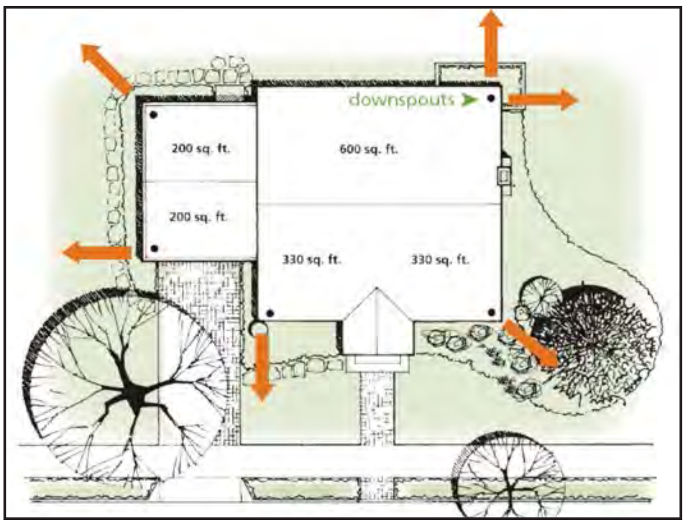


Figure 8-12 Typical Lot Diagram (Adapted From City of Portland, Date Unknown

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

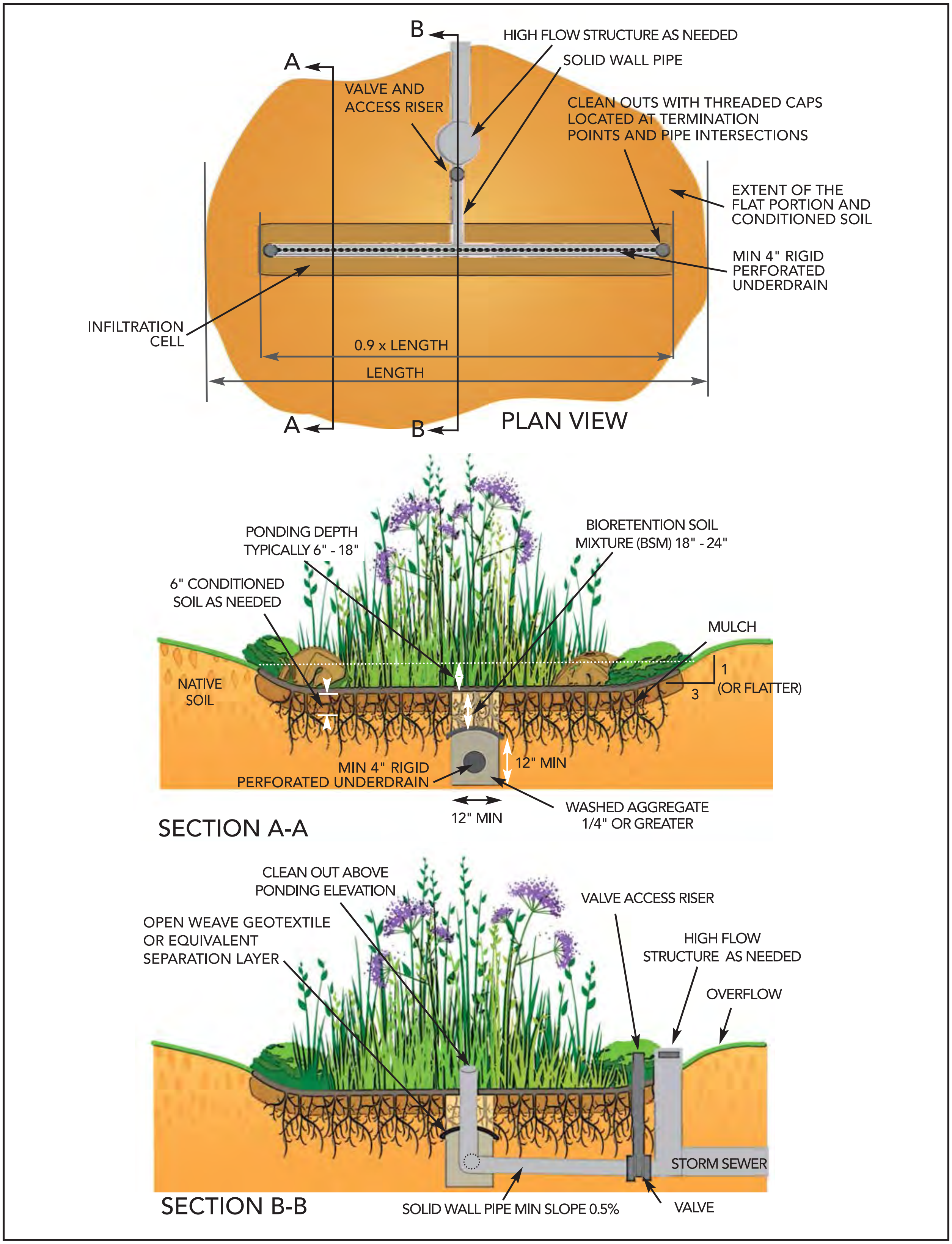


Figure 8-15 Cross Section Schematic of Bioretention System with Valve Outlet or Orifice Control

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."

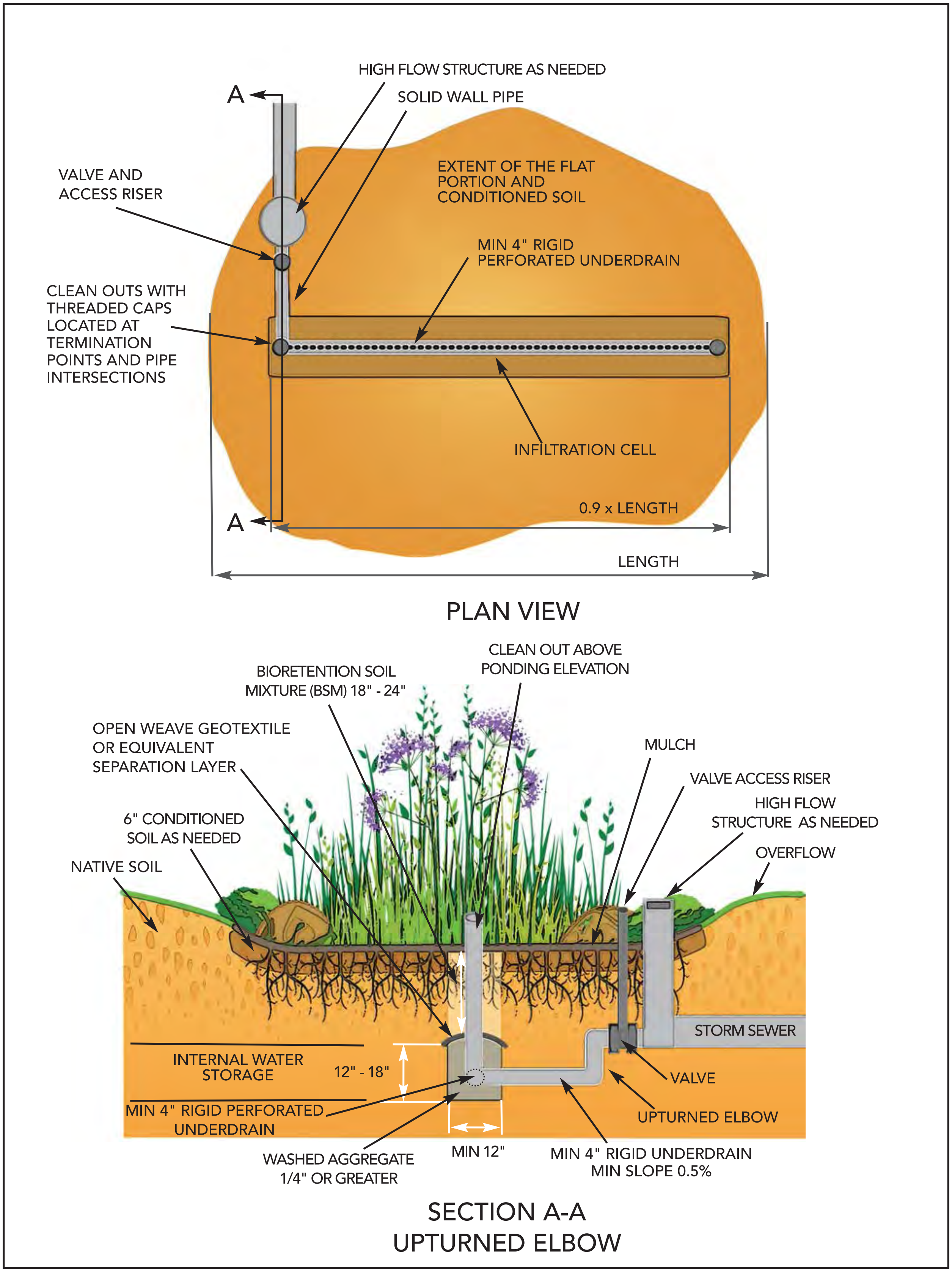


Figure 8-16 Cross Section Schematic of Bioretention System with Upturned Elbow Outlet Control

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.

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

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.

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

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

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[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

USEPA. 1999. *Stormwater Technologies Fact Sheet-Bioretention*.EPA 832-F99_012: www.epa.gov/owm/mtb/bioretn.pdf

Hartsig, Ted and Rodie, Steven. 2009. Bioretention Gardens: A Manual for Contractors in the Omaha Region to Design and Install Bioretention Gardens. University of Nebraska – Lincoln Extension Water: Property Design: http://water.unl.edu/web/propertydesign/publications (listings by chapter)

Li, Houng and Davis, Allen P. 2009. Water Quality Improvement through Reductions of Pollutant Loads using Bioretention. American Society of Civil Engineers Journal of Environmental Engineering.

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

Pennsylvania Department of Environmental Protection. 2006. Pennsylvania Stormwater Best Management Practices Manual: http://www.elibrary.dep.state.pa.us/dsweb/View/Collection-8305

Rodie, Steven N. and Todd, Kim W. 2011. Nebraska Bioretention and Rain Gardens Plants Guide. University of Nebraska – Lincoln Extension EC-1261: https://middleloup.unl.edu/index.php?main_page=index&cPath=120

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)

8.6.2 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

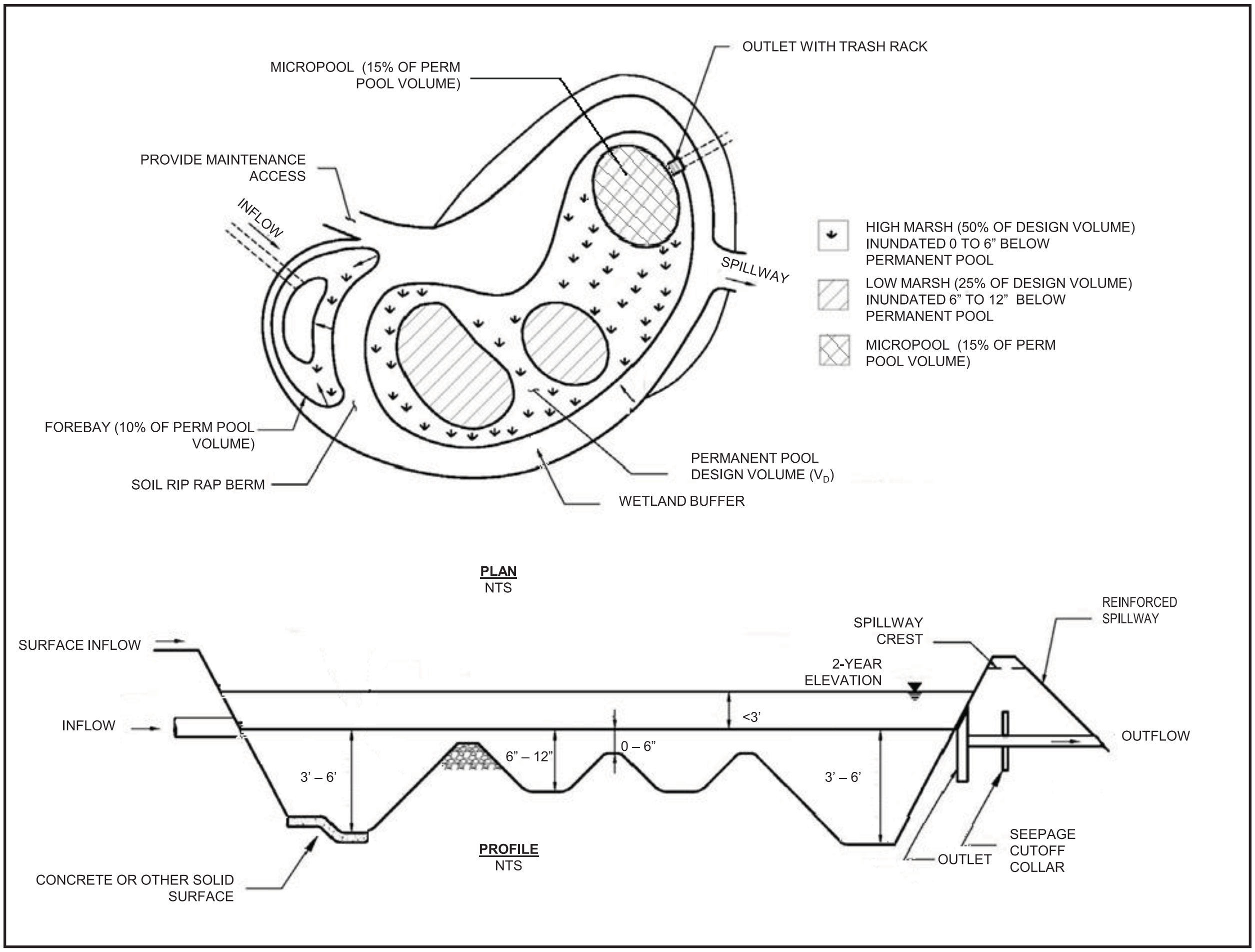

Figure 8-18 Example of a Constructed Wetland Plan and Profile View (Modified from UDFCD,2010)

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.

$$V_P = V_D \times 1.2 \tag{8-6}$$

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.

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

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.

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

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.

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

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

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.

$$L = \frac{Q_{AVG}}{C{H_{2yr}}^{1.5}} \quad (8\text{-}11)$$

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

CASQA. 2003. *California Stormwater Quality Association Stormwater Best Management Practice Handbook*. Available at: http://www.dot.ca.gov/hq/construc/stormwater/manuals.htm

Florida Department of Environmental Regulation. 1988. The Florida Development Manual.

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

Iowa. 2009. Stormwater Management Manual, Version 3: http://www.iowadnr.gov/Portals/idnr/uploads/water/stormwater/manual/part2i.pdf

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

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)

8.6.3 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.

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

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

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

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

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

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.

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

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

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.

$$L = \frac{Q_{AVG}}{CH_{EDDB}{}^{1.5}} \tag{8-17}$$

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

CASQA. 2003. *California Stormwater Quality Association Stormwater Best Management Practice Handbook:* http://www.dot.ca.gov/hq/construc/stormwater/manuals.htm

USEPA. 2006. Pennsylvania Stormwater Best Management Practices Manual. USEPA Bureau of Watershed Management: http://www.elibrary.dep.state.pa.us/dsweb/Get/Document-68851

Galli, F. J. 1992. Analysis of Urban BMP Performance and Longevity in Prince George's County Maryland, Metropolitan Washington Council of Governments. Washington D.C.

Geosyntec Consultant and Wright Water Engineers, Inc. 2011. Technical Summary Volume Reduction. ISBMPD: http://www.bmpdatabase.org/Docs/Volume%20Reduction%20Technical%20Summary%20Jan%202011.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

Iowa. 2009. Stormwater Management Manual, Version 3: http://www.intrans.iastate.edu/pubs/stormwater/index.cfm

KC Metro APWA. 2006. Division V Section 5600 Storm Drainage Systems and Facilities.

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

Muthukrishnan, S. Field, R. and Sullivan, D. 2006. The use of best management practices (BMPs) in urban watersheds (ed. 1, 118-124) Field, R., Tafuri, A., Muthukrishnan, S., Acquisto, B., and Selvakumar, A. (Eds.), Pennsylvania, U.S.: Destech Publications.

Nashville Metropolitan-Davidson County. 2006. Stormwater Management Manual Volume 4: Best Management Practices: www.nashville.gov/stormwater/regs/SwMgt_ManualVol04_2006.htm

NRCS- Maryland Natural Resources Conservation Service. 2000. Code No. 378 Pond Standards/Specifications.

Schuler, T.; Hirschman, D.; Novotney, M.; and Zielinski, J. 2007. Office of Wastewater Management. 2007. Urban Subwatershed Restoration Manual No. 3 – Urban Stormwater Retrofit Practices. Volume 1.0. Center for Watershed Protection.

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).

8.6.4 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.

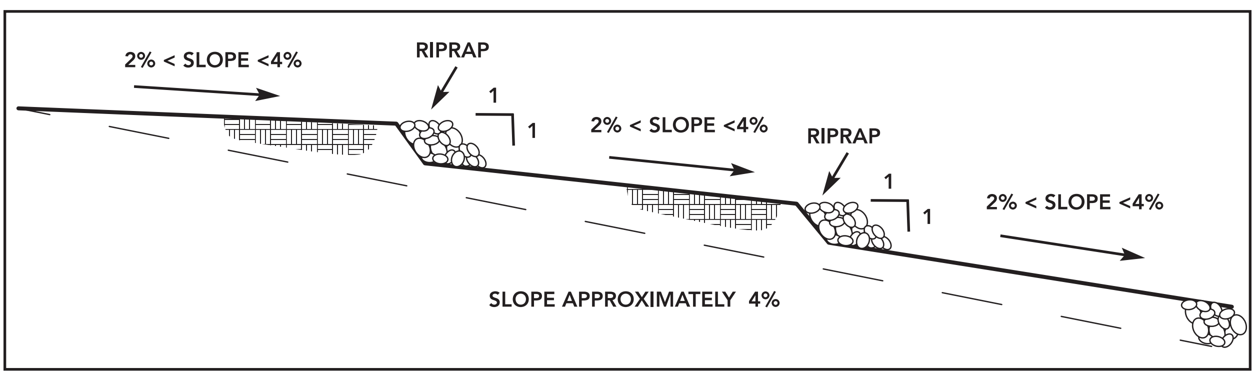

Figure 8-20 Longitudinal Slope Terracing

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.

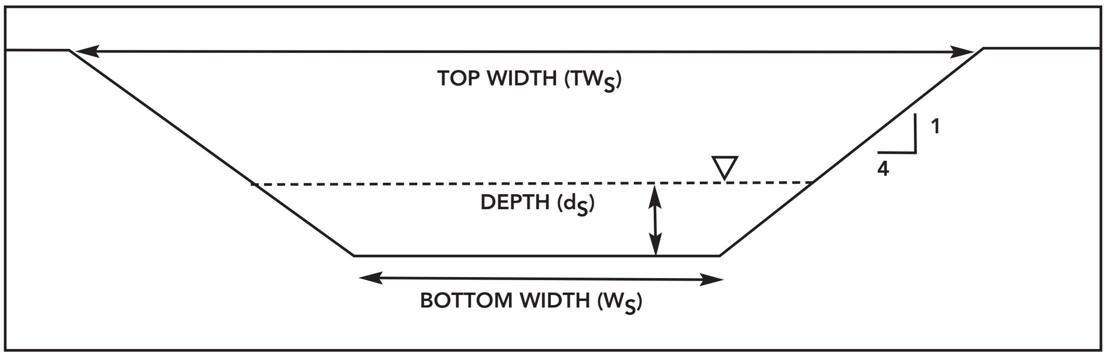

Figure 8-21 Trapezoidal Cross Section with 4:1 Side Slopes

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.

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

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.

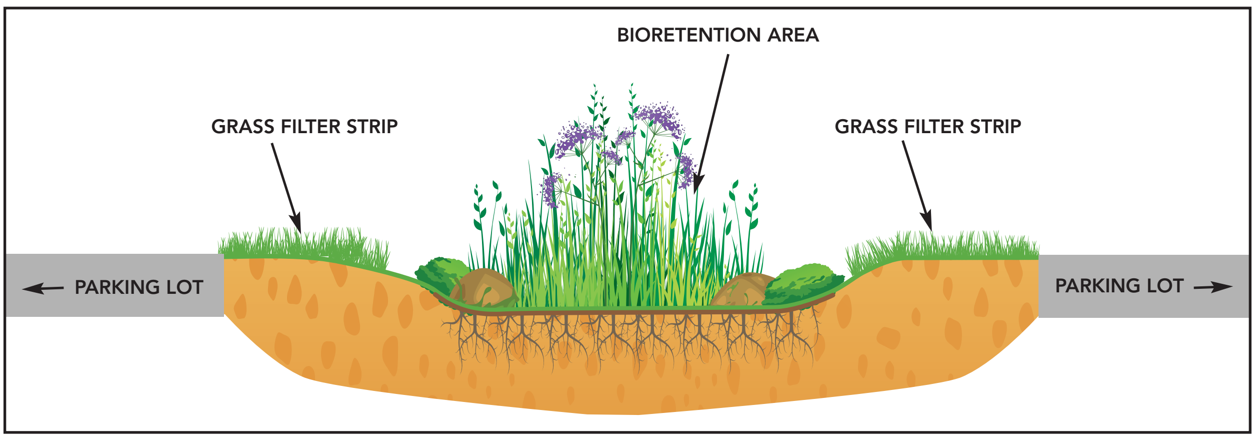

Figure 8-22 Grass Filter Strip Used for Pretreatment

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.

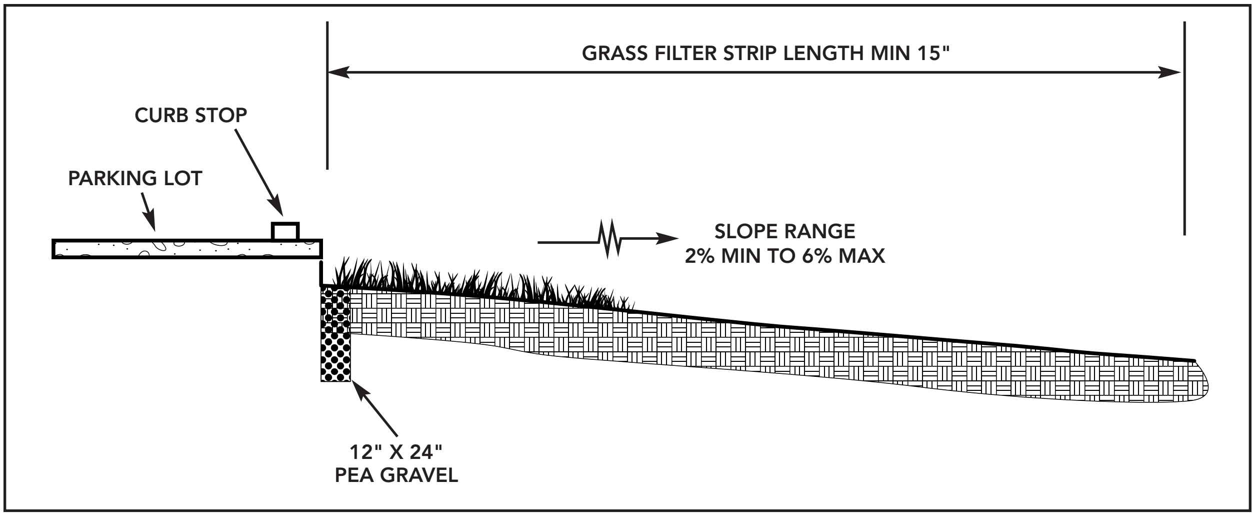

Figure 8-23 Grass Filter Strip Profile (Source Claytor and Schueler, 1996)

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).

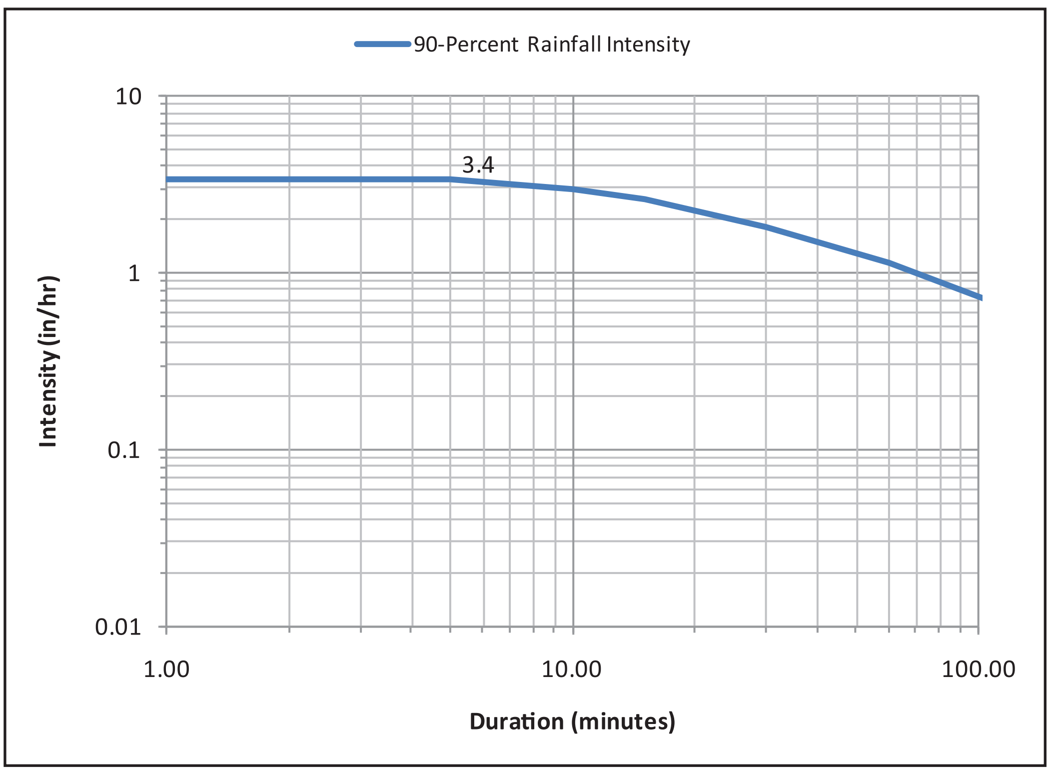

Figure 8-24 Intensity-Duration-Frequency Curve for the 90-Percent Rainfall

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.

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

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..

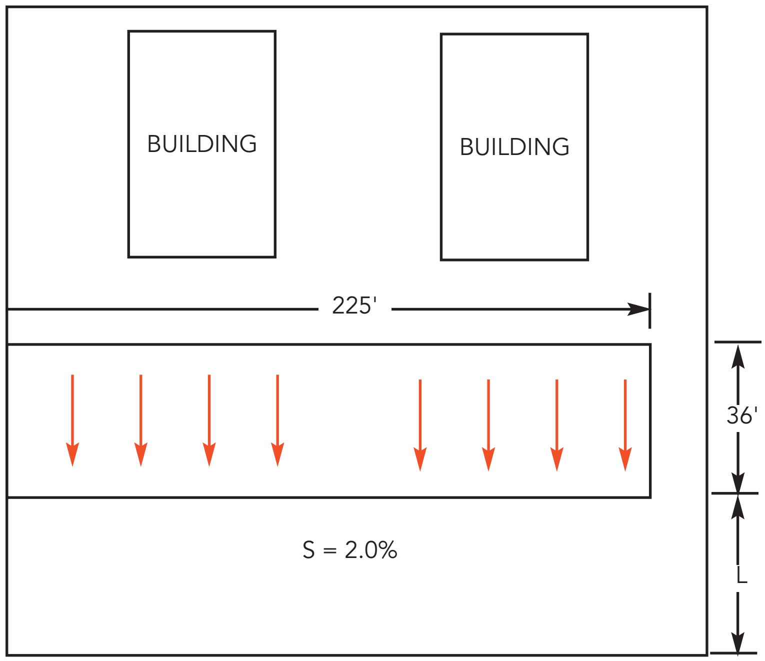

Figure 8-25 Site Plan of 1 Acre Small Business Site

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

CASQA. 2003. California Stormwater Quality Association Stormwater Best Management Practice Handbook: http://www.dot.ca.gov/hq/construc/stormwater/manuals.htm

Claytor, R. and Schueler, T. 1996. Design of Stormwater Filtering Systems. The Center for Watershed Protection. Silver Spring, Maryland.

USEPA. 2006a. USEPA. Grassed Swales BMP Fact Sheet: http://cfpub.epa.gov/npdes/stormwater/menuofbmps/index.cfm?action=browse&Rbutton=detail&bmp=75

USEPA. 2006b. *National Menu of Stormwater Best Management Practices*:

http://cfpub.epa.gov/npdes/stormwater/menuofbmps/index.cfm

Geosyntec Consultants and Wright Water Engineers, Inc. 2011. Technical Summary: Volume Reduction. ISBMPD. http://www.bmpdatabase.org/

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

Iowa. 2009. Stormwater Management Manual, Version 3:
http://www.iowadnr.gov/Portals/idnr/uploads/water/stormwater/manual/part2i.pdf

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

Metropolitan Council/Barr Engineering Co. 2001. Minnesota Urban Small Sites BMP Manual:
http://www.metrocouncil.org/environment/water/bmp/manual.htm

Muthukrishnan, S. Field, R. and Sullivan, D. (2006). *The use of best management practices (BMPs) in urban watersheds* (ed. 1, 118-124) Field, R., Tafuri, A., Muthukrishnan, S., Acquisto, B., and Selvakumar, A. (Eds.), Pennsylvania, U.S.: Destech Publications.