Section 4
Structural BMPs

4.1 Hydrology Methods

Sizing BMPs properly is critical to their success. Two hydrology methods are typically used in calculations depending on whether the intent of the BMP is to capture and treat the Water Quality Volume (WQ_V), or to handle the peak discharge of the WQ_V. Table 4-1 is a summary of BMP hydrologic calculation types and their application.

Table 4-1 BMP Hydrologic Calculation Types

	Hydrology Methods	
BMP	**WQv**	**Peak WQ Discharge**
Bioretention	X	
Extended Dry Detention Basin	X	
Extended Wet Detention Basin	X	
Filter Strip		X
Infiltration Trench	X	
Swales		X

4.1.1 Water Quality Volume (WQv)

Detention and retention BMPs should be designed to capture and treat the WQv. Conveyance BMPs should be designed to handle the peak discharge of the WQv. WQv is based on the water quality rainfall event and volumetric runoff coefficient of the drainage area. The water quality rainfall event for each city can be found using Table 4-2.

Table 4-2 The 90-percent and 85-percent Water Quality Rainfall Event by City

City	County	KS Region	90% (inch)	85% (inch)
Dodge City	Ford	West	0.79	0.58
Garden City	Finney	West	0.89	0.60
Hays	Ellis	West	0.90	0.70
Great Bend	Barton	Central	1.00	0.80
Manhattan	Riley	Central	1.10	0.82
Newton	Harvey	Central	1.20	0.90
Salina	Saline	Central	1.07	0.80
Arkansas City	Cowley	Central	1.20	0.92
Hutchinson	Reno	Central	1.20	0.90
Winfield	Cowley	Central	1.20	0.92
Coffeyville	Montgomery	East	1.50	1.10
Lawrence	Douglas	East	1.18	0.90
Ottawa	Franklin	East	1.20	0.90
Emporia	Lyon	East	1.20	0.90

Two methods can be used to estimate the WQv for a proposed development – the Short-Cut Method and the Small-Storm Hydrology Method.

4.1.1.1 Short-Cut Method (Claytor and Schueler 1996)

The Short-Cut Method should only be used for sites with one predominant land cover type and a drainage area less than 10 acres. This method can be utilized for larger drainage areas if the percent site imperviousness is known.

Short-Cut Method Equations

Equation 4.1 Volumetric Runoff Coefficient

$$R_v = 0.05 + 0.009(\mathrm{I})$$

Where:
R_V = Volumetric runoff coefficient (unitless)
I = Percent impervious of tributary area (%)

Equation 4.2 Water Quality Volume (Short-Cut Method)

$$WQ_V = \frac{P_{WQ} \times R_V \times A_T}{12}$$

Where:
WQ_V = Water quality volume (acre-feet)
P_{WQ} = Water quality rainfall event (inches) from Table 4-2
R_V = Volumetric runoff coefficient
A_T = Tributary area (acres)

4.1.1.2 Small Storm Hydrology Method (Claytor and Schueler 1996)

The Small Storm Hydrology Method is based on the volumetric runoff coefficient (R_V), which accounts for specific characteristics for the pervious and impervious surfaces of the tributary drainage area. The method may be used for all drainage areas. R_Vs are determined by land cover type.

A reduction factor may be applied to the R_V values for drainage areas with disconnected impervious surfaces. The pervious surface flow path below an impervious area must be at least twice the impervious flow path. A summary of volumetric runoff coefficients are provided in Tables 4-3 and 4-4.

Table 4-3 Volumetric Coefficients for Urban Runoff for Directly Connected Impervious Areas (adapted from Pitt, 1987)

Rainfall (inches)	Flat roofs and large unpaved parking lots	Pitched roofs and large impervious areas (large parking lots)	Small impervious areas and narrow streets	Silty soils HSG-B	Clayey soils HSG-C and D
0.50	0.76	0.94	0.62	0.09	0.17
0.75	0.82	0.97	0.66	0.11	0.20
1.00	0.84	0.97	0.70	0.11	0.21
1.25	0.86	0.98	0.74	0.13	0.22
1.50	0.88	0.99	0.77	0.15	0.24

Table 4-4 Reduction Factors to Volumetric Runoff Coefficients for Disconnected Impervious Surfaces (adapted from Pitt, 1987)

Rainfall (inches)	Strip commercial and shopping center	Medium to high density residential with paved alleys	Medium to high density residential without alleys	Low density residential
0.50	0.95	0.18	0.18	0.17
0.75	0.99	0.27	0.21	0.20
1.00	0.99	0.38	0.22	0.21
1.25	0.99	0.48	0.22	0.22
1.50	0.99	0.59	0.24	0.24

Note: To use the reduction factors for disconnected impervious surfaces listed above, the impervious area uphill from a pervious area (a cover type that allows stormwater to infiltrate) should be less than one-half the area of the pervious surface, and the flow path through the pervious area should be at least twice the impervious surface flow path. For example, a 10-foot wide sidewalk would be a "disconnected impervious surface" if separated from the conveyance system by a 20-foot grassed strip other pervious cover.

Small Storm Hydrology Method

Equation 4.3 Weighted volumetric runoff coefficient

$$R_{v,w} = \frac{\Sigma(R_{V1} * A_{C1}) + (R_{V2} * A_{C2}) + \ldots(R_{Vi} * A_{Ci})}{A_T}$$

Where:

$R_{V,W}$ = Weighted volumetric runoff coefficient
R_{Vi} = Volumetric runoff coefficient for cover type i
A_{Ci} = Area of cover type i (acre)
A_T = Total tributary area (acre)

Equation 4.4 Water Quality Volume (Small Storm Method)

$$WQ_V = \frac{P_{WQ} \times R_{V,weighted} \times A_T}{12}$$

Where:

WQ_V = Water quality volume (acre-feet)
P_{WQ} = Water quality rainfall (inches)
$R_{v,weighted}$ = Weighted volumetric runoff coefficient
A_T = Tributary area (acres)

4.1.2 Rational Method

A conveyance BMP should be designed by calculating the peak discharge for the water quality rainfall event using the Rational Method.

Rational Method

Equation 4.5 Runoff Coefficient (Rational Method)

$$C = 0.3 + (0.6xI)$$

Where:

C	=	Runoff Coefficient
I	=	Percent impervious divided by 100

Equation 4.6 Peak Runoff Rate (Rational Method)

$$Q = C \times i \times A$$

Where:

Q	=	Peak rate of runoff (cfs)
C	=	Runoff Coefficient
i	=	Rainfall intensity for water quality rainfall event from Appendix A at the duration equal to the calculated time of concentration (inches/hr)
A	=	Tributary drainage area (acres)

Time of Concentration (Tc)

Equation 4.7 Time of Concentration

$$T_C = T_I + T_T$$

Where:

T_C	=	Time of concentration (minutes)
T_I	=	Overland flow time to the most upstream inlet or point of entry (minutes)
T_T	=	Travel time in an enclosed system or channel (minutes)*

**For this manual, this is only used in instances where concentrated flow is entering a BMP.*

(Source: Section 5602.7 of APWA 5600, November 2005)

Overland Flow Time (T_I)

Use the following formula or other method approved by the reviewing agency to calculate overland flow time. Overland flow time shall not be greater than 15 minutes.

Travel Time in an Enclosed System or Channel (T_T)

Equation 4.8 Overland Flow Time

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

Where:

T_I	=	Overland flow time to the most upstream inlet or point of entry (min)
C	=	Overland Flow Runoff Coefficient for cover type
D	=	Overland flow distance parallel to slope (feet); 100 feet shall be the maximum distance for overland flow
S	=	Slope of overland flow path (%)

Use the following formula or other method approved by the reviewing agency to calculate the travel time in an enclosed system or channel by dividing the length of travel by the velocity of flow.

Equation 4.9 Channelized Travel Time

$$T_T = \frac{D_C}{V}$$

Where:

T_T	=	Channelized travel time (min)
D_C	=	Channelized flow distance (feet)
V	=	Velocity of flow (ft/min) calculated using Manning's equation

4.1.3 References

MARC and APWA. 2008. *Manual of Best Management Practices for Stormwater Quality*.

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Section 4
Structural BMPs

4.2 Lot Level BMPs

Lot level BMPs are defined as a localized practice that is appropriate for private land owners and concerned citizens to install and operate. These BMPs are relatively easy to maintain as they can only accept and treat stormwater from a small drainage area less than one acre. However, on a watershed level, a single lot level BMP will only have a limited impact on water quality or quantity. Lot level BMPs should be executed as a regional or a neighborhood wide effort in order to improve stormwater runoff quality in a watershed.

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Lot Level BMPs
4.2.1 Rain Gardens

A rain garden is a depressed area of native vegetation that is designed to capture and hold stormwater. A rain garden is designed to accept runoff from very small areas such as roof tops, driveways, or residential streets. Runoff from surrounding impervious areas should enter the rain garden as sheetflow. Direct discharge from rain spouts and gutters should enter the garden through an energy dissipater device. Individual gardens aid in controlling the volume of runoff from individual lots that would otherwise combine with and contribute to runoff from other properties into the stormwater sewer system. However, to provide an effective contribution to stormwater management, rain gardens must be sufficient in number and common throughout an area (MARC, 2008). Figure 4-1 is an example of a rain garden BMP.

Figure 4-1 Rain Garden at University of Missouri-Kansas City

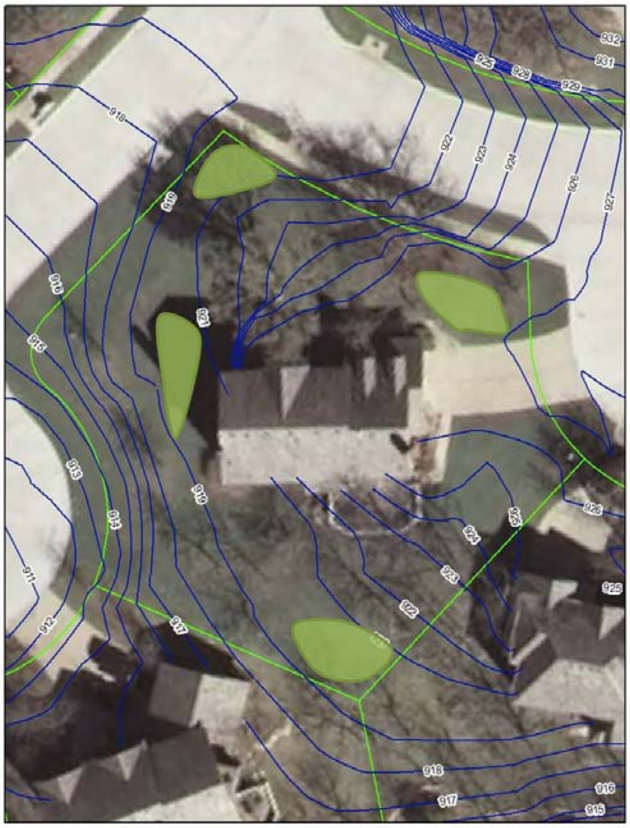
Location characteristics (Slope, Soil Type)	Slope: < 5 percent Soil Type: A, B
Contributing drainage area	< 1 acres
Design size	10 to 20% drainage area
Detention time for WQv treatment	24-48 hours
Pollutant removal efficiencies[1]	82-95% TSS, 80-85% TN, 65% TP
Potential for education and outreach	**High.** Lot level private gardens can be part of your NPDES outreach activities
Potential for use with other BMPs	**Moderate.** As a downstream infiltration BMP, can be used in treatment train.
Implementation Category	Short Term: **Easy** Long Term: **Difficult** (See Section 5.4.1)
Maintenance	**High.** Sediment/debris removal, vegetation upkeep (See Section 5.4.1)

[1]New York State, 2003

4.2.1.1 General Application

Rain gardens can be used to improve the quality of urban/suburban runoff coming from roof tops, driveways, and lawns of residential neighborhoods, small commercial areas, and parking lots. They are typically most effective for catchments less than one acre. Rain gardens work well with other BMPs such as downstream infiltration management practices. 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 4-2. Refrain from placing a rain garden in just any location where water typically pools for long periods as this may indicate low soil infiltration rates (Ellingson, 2008).

Figure 4-2 Example Placement of Rain Gardens on a Residential Lot



4.2.1.2 Design Requirements

Rain gardens require that captured rainfall and runoff be infiltrated below the surface within 24 to 48 hours. 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. Therefore, the critical design requirement is the rate at which water can infiltrate into the soil.

Site Considerations

- Rain gardens should be placed in the lowest portion of a yard to ensure that runoff will flow into it. Do not place the garden in an area that typically has ponded water (indicating poor infiltration) or that is not the lowest point. Ponded water in other areas of the yard may indicate soils with low permeabilities.

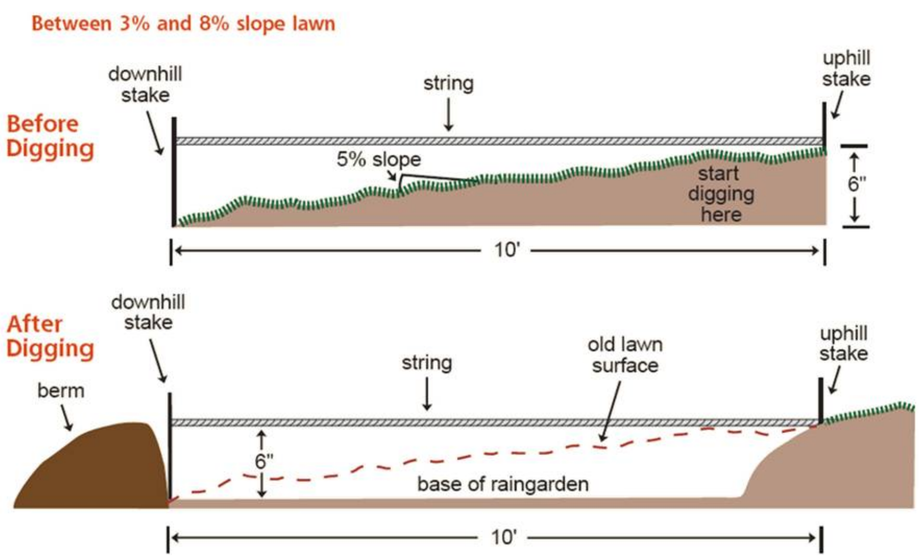
- Perform a percolation test to determine the infiltration rate of the soil. To perform this test, choose a level ground location. Cut the bottom from a can or other hard-material cylinder and push it 2 to 3 inches into the ground. Fill can with water, measure water level with a ruler, and time how long it takes water to completely drain. Infiltration rate can then be calculated by dividing the measured water level by the total time to drain.

- To build an infiltration garden in an area with low permeability, augment native soil with an engineered soil with a 1:1 ratio of sand to compost mix
- An organic-rich top soil will initiate plant growth and soak up excess runoff
- The rain garden should not be placed in the proximity of a foundation or in any other area where ponded water may create problems

Rain Garden Configuration

- The ponding depth shall be the depth of water that will infiltrate into the soil in 24 hours based on the percolation test results (3 to 6 inches typical). The soil from excavation can be used to create a berm on the side of the rain garden as shown in Figure 4-3.
- The garden shall be sized to treat and accept the WQv and shall have a flat bottom to ensure even infiltration into the soil across the garden.
- Plant selection should include native species that are tolerant of both wet and dry cycles. This will achieve the highest level of success in a rain garden.
- Route stormwater away from the garden initially until vegetation becomes established, typically for a 30 to 60 day timeframe.
- Irrigate as needed during the first 60 days to establish plants.

Figure 4-3 Example of Where to Place Excavated Soil When Building Rain Garden (University of Wisconsin-Extension, 2003)



Vegetation Selection

Utilize native vegetation in the rain garden design. These plants have deep roots that can sustain periods of drought. Determine the following specific for the rain garden site in order to select proper vegetation:

- Soil types (soil tests, soil maps in Appendix B) and organic matter
- Annual precipitation with dates for wet/dry season (Maps in Appendix A)
- Ecoregion and corresponding vegetation (Map and table in Appendix C)
- Previous land use

Provide the soil type, precipitation, previous land use, and ecoregion information to a local nursery or landscaping specialist for planting suggestions (vegetation types, seeding rates, establishment procedures, maintenance procedures). Use the "typical vegetation" listed in Appendix C as a guideline to check final list. Native vegetation contacts and links are listed in Appendix C.

4.2.1.3 Submittal Requirements

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

- Drainage area map to rain garden.
- Percolation test results.
- Dimensions of rain garden (L x W x D).
- Plan view. Components clearly labeled with dimensions. Distances from structures and locations of downspouts should be noted.
- Vegetation plan stating typical height of plants along with schedule for installation and initial maintenance.

4.2.1.4 Web-Based Resources

10,000 Raingarden initiative: www.rainkc.com/_ccLib/image/pages/PDF2-66.pdf

Citizen's Guide to Protecting Wilmington's Waterways:
www.wilmingtonnc.gov/Portals/_default/stormwater/cguide.pdf

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

Native vegetation establishment: www.grownnative.com,
www.kansasnativeplantsociety.org,

www.oznet.ksu.edu/library/crpsl2/MF2291.pdf

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

MARC *Raingarden Design Brochure*:
www.marc.org/environment/Water/bmp_manual.htm

4.2.1.5 References

Ellingson, Sue. 2008. *Sue's rules for raingardens*. Located at sueellingson.com/raingardens.

MARC and APWA. 2008. *Manual of Best Management Practices for Stormwater Quality*. Located at www.marc.org/environment/Water/bmp_manual.htm

University of Wisconsin-Extension. 2003. *Rain Gardens: A how-to manual for homeowners*. Located at clean-water.uwex.edu/pubs/raingarden/rgmanual.pdf

New York State. 2003. *New York State Stormwater Design Manual*. Located at www.westchester.gov.com/planning/environmental/soilwater reports/altpractices.pdf

Lot Level BMPs
4.2.2 Rain Barrels and Cisterns

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. By diverting water from storm drainage systems, rain barrels and cisterns reduce pollutants and the volume of runoff entering local rivers and streams.

Location characteristics (Slope, Soil Type)	Slope: N/A Soil Type: All
Contributing drainage area	Rooftop drainage
Design size	50-60 gallons (rain barrel) 50-5000 gallons (cistern)
Detention time for WQv treatment	N/A
Pollutant removal efficiencies	N/A
Potential for education and outreach	**High.** Lot level practices can be part of your NPDES outreach activities
Potential for use with other BMPs	**Moderate.** Can be used for BMP irrigation during dry periods
Implementation Category	Short Term: **Easy** Long Term: **Easy**
Maintenance	**Moderate.** Keep barrel free of organic material, mesh screens and olive oil will keep mosquitoes from breeding, use stormwater regularly to allow adequate storage room for future rain events

4.2.2.1 Rain Barrels

A rain barrel is typically a 50-60 gallon tank to which downspouts are directed. An example of a rain barrel is shown in Figure 4-4. 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.

Design and Installation Requirements

- **Components.** 50 to 60 gallon covered plastic tank with an opening at the top for downspout discharge, an overflow outlet, and a valve and hose adapter at the bottom. It is recommended that the barrel have a sealed, child resistant top that can be easily removed for cleaning.

- **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. Downspouts should be cut to allow a three inch gap between the top of the barrel and the end of the downspout, allowing for space to remove the lid and clean the inside of the barrel. Overflow outlets should be routed away from foundations and to pervious areas. Additional rain barrels will increase the quantity of water stored. Table 4-5 provides the total runoff volume generated based on a roof's square footage and the amount of rainfall.

Figure 4-4 (left) Rain Barrel Diagram (townofblackmountain.org) (right) Residential Rain Barrel in River Falls, Wisconsin (rfcity.org)

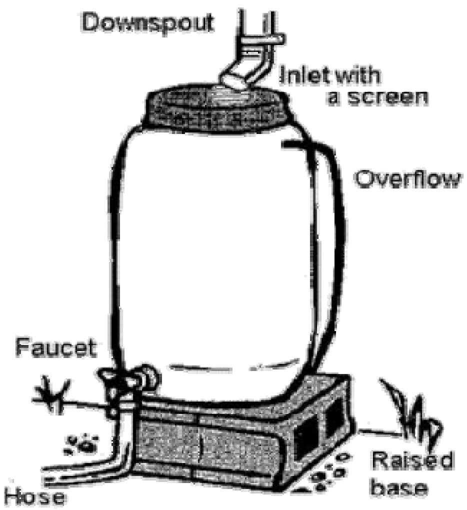


Table 4-5 Total Runoff Volume Generated Based on Roof's Square Footage

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

Web Based Resources

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

Watershed Activities to Encourage Restoration:
www.watershedactivities.com/projects/spring/rainbarl.html

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

Town of Black Mountain Rain Barrel Information:
www.townofblackmountain.org/rain_barrel.htm

EPA Fact Sheet: www.epa.gov/region03/p2/what-is-rainbarrel.pdf

MARC Rain Barrel Information:
www.marc.org/Environment/Water/buildrainbarrel.htm
www.mtwatercourse.org/NSP/KSMO_buildarainbarrel.pdf

Rain Barrel Guide: www.rainbarrelguide.com

Where to Purchase Rain barrels

Check with local hardware store.

www.gardeners.com

www.rainbarrelsource.com

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

www.bayteccontainers.com

4.2.2.2 Cisterns

Cisterns are only distinguishable from rain barrels given their large size, 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 using an electric pump. An example of a cistern is shown in Figure 4-5.

Figure 4-5 Residential Aboveground Cistern in Portland, Oregon (www.rwh.in)

Design and Installation Requirements

- **Components.** Variable size tank constructed of an impervious, water retaining material. 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.

Web Based Resources

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

Boston Metro Area Planning Council LID Toolkit
www.mapc.org/regional_planning/LID/cisterns_barrels.html

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

Lot Level BMPs
4.2.3 Disconnect Impervious Areas

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. 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. Figure 4-6 provides an example of green space that runoff could be redirected to.

Figure 4-6 Sidewalk Median in Topeka, KS Provides Pervious Area

4.2.3.1 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.

4.2.3.2 Design Requirements

Disconnecting impervious areas requires little construction and few materials. 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 4-7 and 4-8 demonstrate typical lot diagrams for disconnecting impervious areas. Figure 4-7 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, 2008). Figure 4-8 demonstrates a highly urbanized area where there are potential disconnection locations available adjacent to buildings and other impervious area.

Figure 4-7 Typical Lot Diagram (Adapted From Portland, 2008)

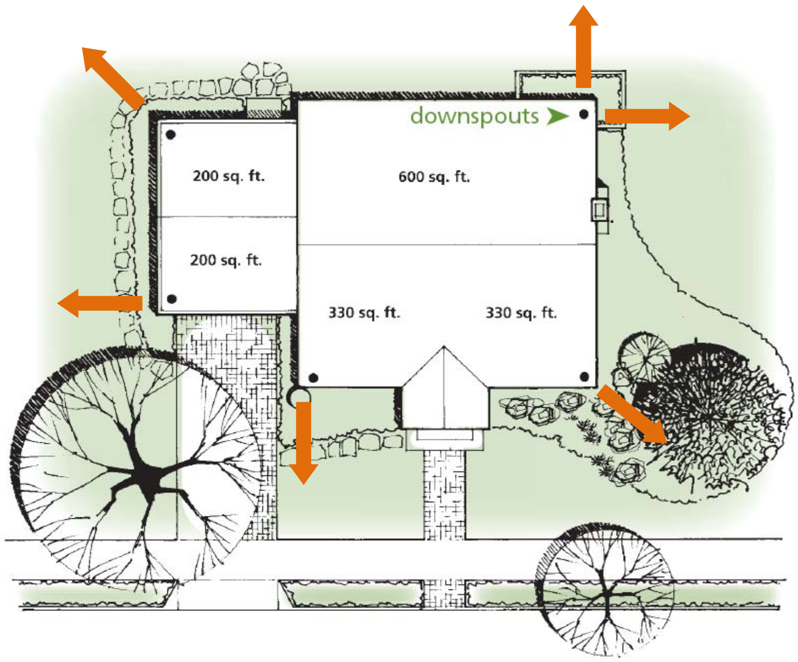


Figure 4-8 Sprint Campus in Overland Park Provides Pervious Area Around Buildings for Downspout Discharge

Section 4
Structural BMPs

4.3 Bioretention

Bioretention utilizes vegetation to accept and treat stormwater runoff through infiltration into layers of plant roots and the growing medium. Reductions in stormwater runoff are achieved via natural plant processes and movement through soil media. Runoff volumes are also decreased by temporary storage in the soil media and permanent removal by evapotranspiration from the vegetation. Bioretention facilities should be designed so that runoff in excess of the water quality volume (WQv) may bypass the facility through an overflow structure. The WQv is the volume of runoff that must be captured to achieve water quality benefits. The WQv is allowed either to infiltrate into the surrounding soil or be collected by an underdrain system that discharges to the storm sewer system. Thus, bioretention facilities can be designed to be on or off-line of existing stormwater systems. Figure 4-9 provides examples of bioretention cells.

Sections from this manual that may need to be referenced for additional information are: Section 2; Section 4.1; Section 5.

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 Minimum (W x L): 15 ft x 40 ft[1]
Detention time for WQv treatment	1-3 days[1]
Pollutant removal efficiencies[1]	40% TN, 65% TP, and 80 to 90% Zn, Cu, Pb reduction[2,3]
Potential for education and outreach	High (highly trafficked areas-education, aesthetics)
Potential for use with other BMPs	Works well with upstream source controls and filter strips and swales
Implementation Category	Long term: 15-20 year lifespan based on metal accumulation[4]
Maintenance	High initially, lower with establishment of BMP (Refer to Section 5.4.1)

[1]MARC, 2008, [2]Davis et al., 2003, [3]Hunt et al., 2006, [4]Mac, 2005

Figure 4-9 Series of Bioretention Cells on Jackson Street, Topeka, Kansas (Source: GreenTopeka.org)

4.3.1 General Application

Bioretention is a good BMP to be used in urban areas because of the minimal land requirement and thus is usually located in highly trafficked areas. This provides opportunities for BMP public education and signage. Bioretention facilities should be located upland from inlets that receive sheet flow from graded areas or in recessed areas that receive runoff from imperious urban infrastructure. 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 area or conveyed into the bioretention area by a curb and gutter collection system (EPA, 1999, UDFCD, 2008). 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, 2008). Bioretention cells will not function correctly in an area under construction or with exposed soil, as inundation with suspended sediment will prevent infiltration from occurring in the bioretention cells (MARC, 2008).

4.3.2 Advantages and Disadvantages

Advantages	Disadvantages
High volume reduction (+/- 90-percent)	Easily clogged with suspended sediment
40-percent TN, 65-percent TP, and 80 to 90-percent Zn, Cu, Pb reduction[1,2]	Higher construction costs per impervious acre[4]
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 stormwater infrastructure elsewhere	Cannot be used in drainage areas with slopes > 20-percent[3]
Effective in a “treatment train" with BMPs that reduce sediment loads	May not effectively remove pollutants when first brought on-line
Minimal footprint (1-15-percent of drainage area)	
Function increases with time	
May contribute to groundwater recharge	

[1]Davis et al. (2003), [2]Hunt et al. (2006), [3]EPA, 1999

4.3.3 Design Requirements and Considerations

The procedure for designing a bioretention cell is outlined below. The design components are described in the order of construction starting with the underdrain and continuing through bioretention media, planting soil, vegetation, ponding area, and overflow system.

4.3.3.1 Overall Design Guidance

- Bioretention facilities shall not be constructed until all tributary areas are permanently stabilized against erosion and sedimentation or a pre-treatment practice is implemented. Heavy sediment loads to the cell will reduce infiltration rates and require reconstruction of the cell to restore its defined performance.

- The bioretention facility shall be designed to capture the WQv. The WQv should filter through the facility’s planting soil bed in 1 to 3 days.

- The bottom area should be sized such that standing water is present less than 24 hours.

- Any facilities wider than 20 feet shall be twice as long as they are wide (UDFCD, 2005).

- The tributary area for a bioretention area shall be less than 4 acres. Multiple bioretention areas may be required for larger tributary areas (EPA, 1999).

4.3.3.2 Excavation

Excavation is almost always required to meet the design requirements except in an area with soils with high permeability with no underdrain. The bioretention facility can be excavated before final stabilization of the tributary area and utilized for erosion and sediment purposes, such as a sediment basin; however, the bioretention soil mixture and underdrain system shall not be placed until the entire tributary area has been stabilized. Bioretention facility side slopes shall be excavated at 4:1 or flatter. Low ground-contact pressure equipment, such as excavators and backhoes, is preferred on bioretention facilities to minimize disturbance to established areas around the perimeter of the cell. No heavy equipment shall operate within the perimeter of a bioretention facility during underdrain placement, backfilling, planting, or mulching of the facility

4.3.3.3 Underdrain/Outlet

The underdrain/outlet is always required for bioretention cells in highly urbanized areas or in soils with a low permeability where excess overflow may be a concern. An underdrain structure allows operators to control the stormwater detention time and allows detained runoff to be released into an existing storm sewer system. The underdrain also increases airflow into the soil media keeping it aerobic. Figure 4-10 shows a side view of the underdrain configuration.

Figure 4-10 Underdrain Configuration Side View (Source: MARC, 2008)

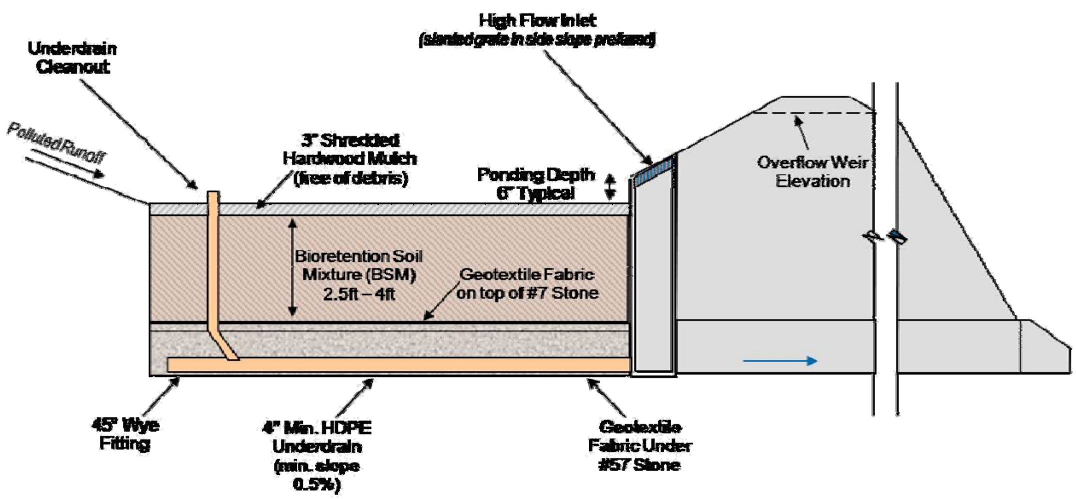
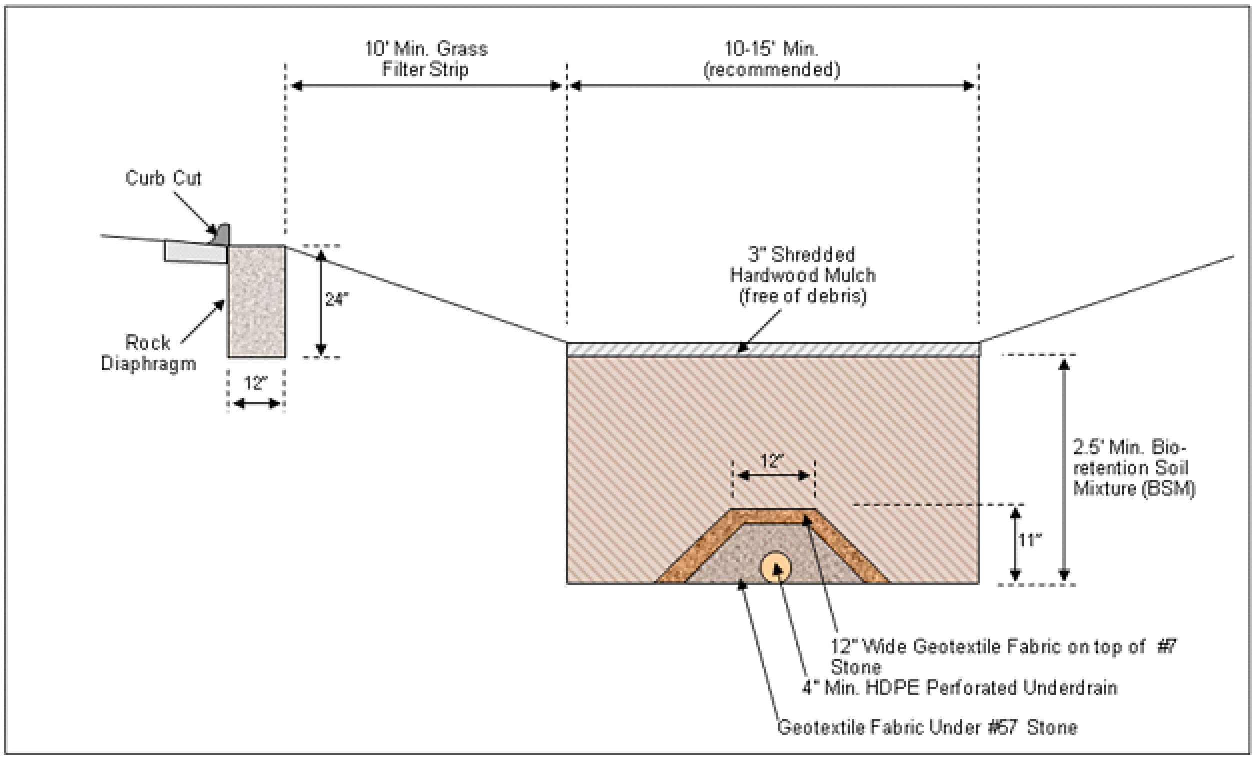


Figure 4-11 provides an example of a bioretention underdrain. Key components of an underdrain/outlet for a bioretention cell include:

- Four-inch or larger perforated pipe with perforations between 0.25-0.375 inches spaced at 6-inch centers with a minimum of 4 holes per row.

- The minimum grade of the underdrain must be 0.5-percent with one cleanout run every fifty feet.
- A valve/cap system at the end of the underdrain allows operator to plug the system and increase the detention time.
- The underdrain shall be covered with 8-inch coarse rock in a trapezoid shape. Filter fabric shall be on the top of the trapezoid only.

Figure 4-11 Bioretention Underdrain Example



4.3.3.4 High Flow Structures

An overflow system is crucial in commercial and industrial settings to ensure that the stormwater does not back up onto surrounding parking lots and public areas. Having a high flow structure also reduces the possibility of hydraulic overload on the bioretention area. If the bioretention facility will be utilized with existing stormwater management systems, the overflow should be connected to this system. An example of an overflow device is shown in Figure 4-12.

**Figure 4-12 Mize Lake, Lenexa, Kansas
Bioretention High Flow Structure**

Source: CDM

4.3.3.5 Bioretention Soil Mixture (BSM)

It is recommended that bioretention facilities utilize native soil with an organic-rich top soil. The bioretention soil mixture must meet the BSM specification in Appendix B.4 (MARC, 2008). The soil must have the appropriate chemical and physical properties to support a diverse microbial and plant community.

The depth of BSM shall be sized to hold the Water Quality volume. The minimum depth shall be 2.5 feet.

4.3.3.6 Ponding Area

The aboveground storage of runoff must drain within 24 hours, but the ponding depth should be minimized to reduce the hydraulic load on soils. Ponding depths should range from 6 to 12 inches.

4.3.3.7 Flow Entrance

Typically, bioretention areas are constructed in space-limited urban settings like parking lots and medians. However, care must be taken to ensure that all runoff entering the bioretention area is in sheet-flow. Runoff must be evenly distributed in order to minimize erosion and loss of vegetation. If curb cuts, cut parking blocks, or other concentrated flow generators are adjacent to the cell, energy dissipation is necessary. The designer should show in design calculations that flow is unconcentrated prior to entering the bioretention cell. An example of where flow enters a bioretention cell is shown in Figure 4-13.

Figure 4-13 Vegetated Swale Guides Runoff From Surrounding Parking Lots Into Bioretention Cell Kansas City, MO (Source: CDM)

4.3.3.8 Vegetation

Native tall-grass prairie plant species are believed to improve soil physical and chemical processes in a Midwestern bioretention cell. Tall-grass species are associated with exceptionally productive soil systems and have extremely dense root structure. Native grasses can withstand the climatic variability typical throughout Kansas. Guidelines for using native vegetation are outlined in Section 5.

4.3.4 Design Calculations

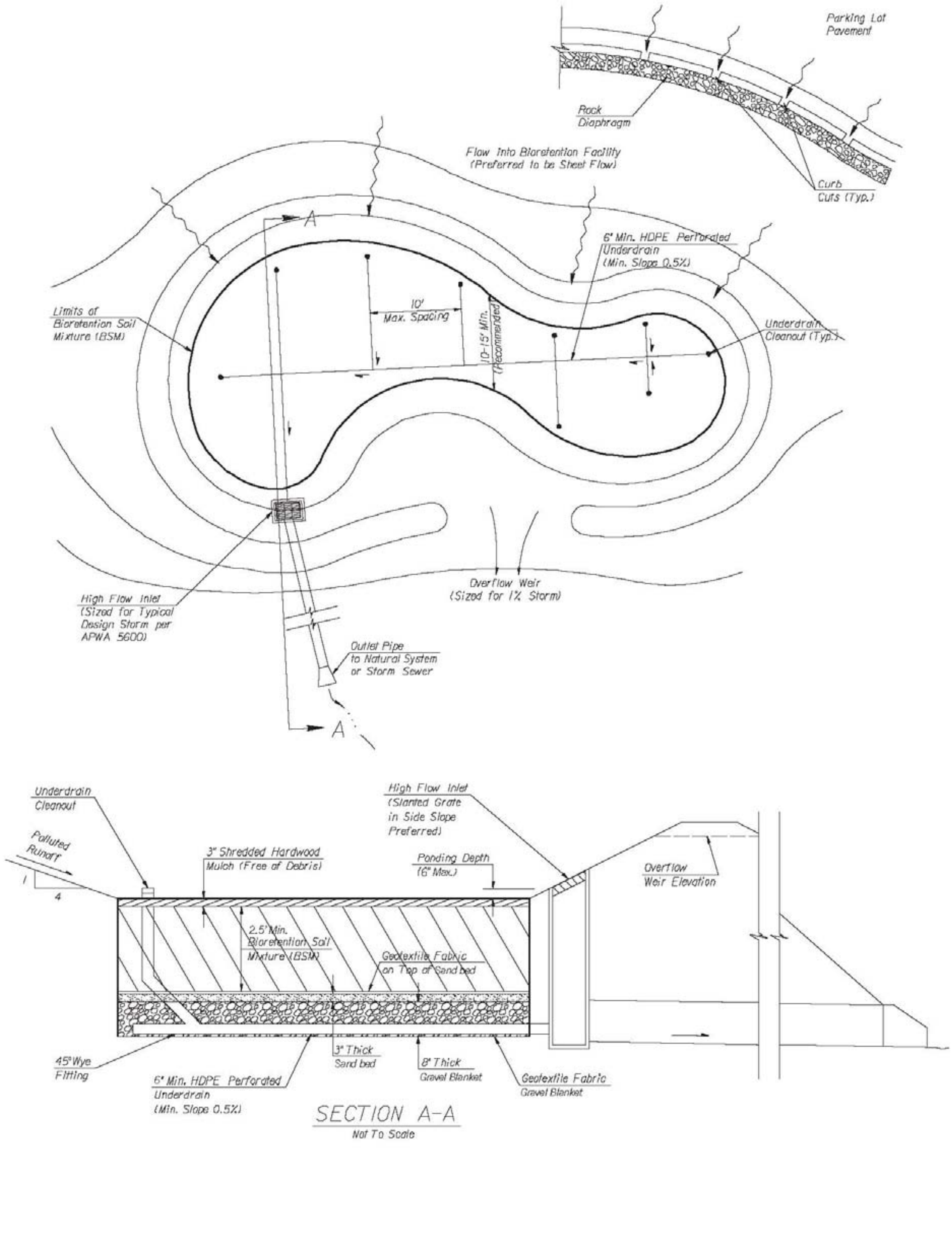
A short summary of the design calculations is presented below. A detailed design example is outlined in 4.3.5. A typical bioretention plan and profile is shown in Figure 4-14.

- **Step 1** Determine WQ_V based on drainage area and regional precipitation information according to Equation 4. 1 and 4.2.

Equation 4.1 $$R_v = 0.05 + 0.009(\mathrm{I})$$

Equation 4.2 $$WQ_V = \frac{P_{WQ} \times R_V \times A_T}{12\,(in)}$$

Figure 4-14 Bioretention Plan and Profile
Source: MARC, 2008



- **Step 2** Design a pretreatment entity to slow runoff and retain sediments, such as a swale or filter strip.

- **Step 3** Size the bioretention soil bed and planting area based on WQ_V and soil characteristics according to Equation 4.10 and 4.11. Equation 4.11 is valid to calculate a bioretention cell length based on the recommended 2:1 length to width ratio.

- **Step 4** Design the underdrain for connection to existing stormwater infrastructure or to drain soils with low permeabilities. Find the number of transverse collector pipes using equation 4.12.

Equation 4.10 Filter Bed Surface Area

$$A_F = \frac{WQ_V \times d_f}{k \times t_f \times (h_{avg} + d_f)}$$

Where:

A_F	=	Filter bed surface area (acres)
WQ_V	=	Water quality volume (acre feet)
k	=	Coefficient of soil permeability (feet/day)
t_f	=	Time required for WQ_V to filter through soil (days)
h_{avg}	=	Average ponding depth above plant in soil bed (feet)
d_f	=	Planting soil bed depth (feet)

Equation 4.11 Filter Bed Length (Assuming L:W = 2:1)

$$L_f\,(ft) = \sqrt{87120 \times A_f}$$

Where:

L_F	=	Filter bed length (feet)
A_F	=	Filter bed surface area (acres)

Equation 4.12 Number of Transverse Collector Pipes

$$N_{TU} = \frac{L_f}{S_{TU}}$$

Where:

N_{TU}	=	Number of transverse collector pipes
L_F	=	Filter bed length (feet)
S_{TU}	=	Transverse collector pipe spacing (inches)

- **Step 5** Install appropriate vegetation using methodology provided by local native vegetation experts. Provide an overflow to maintain vegetation integrity during high flow.

4.3.5 Design Example

Design a bioretention area for a small parking lot median of a local grocery store in Arkansas City, KS. The median will drain a 0.5 acre parking lot and 1 acre of roof runoff with a total of 99-percent imperviousness. The parking lot is graded to drain to the bioretention cell. The parking lot is located in southeast Arkansas City with type C soils.

4.3.5.1 Basin Water Quality Volume

Determine the tributary drainage area to the bioretention area (A_T)
The tributary area, A_T, is 1.5 acres. Due to the fact that A_T = 1.5 acres and the percentage imperviousness is known, we shall utilize the Short-Cut Hydrology Method.

Calculate the Rv based on equation 4.1
The tributary area is 99-percent impervious. Thus, $R_v = 0.05 + 0.009(99) = 0.941$

Calculate the WQ_V based on equation 4.2
For Arkansas City, KS, the water quality event is 1 inch.
Thus, $WQ_V = (1*0.941*1.5)/12 = 0.12$ ac-ft

4.3.5.2 Pretreatment

Runoff that flows directly from an impervious area is likely to concentrate and cause erosion in the bioretention area. Thus, a pretreatment device is strongly suggested. Vegetated filter strips and vegetated swales work to reduce the velocity of runoff and promote settling of suspended sediments. In situations where area is limited, utilize underground proprietary devices to detain and slow runoff (MARC, 2008).

4.3.5.3 Planting Soil Bed and Ponding Area

Choose planting soil bed depth (d_f)
The planting soil bed depth is a design decision. Typical depths are from 3-5 feet. For this example, d_f = 4 feet.

Soil permeability (k)
An soil matrix (Appendix B.4) was utilized since the existing soil types are of the hydrologic group C with lower than average permeabilities. The soil matrix used has a permeability of 1 foot per day.

Maximum ponding depth (h_{max})
Ponding depths should range from 6 to 12 inches. To maximize infiltration and reduce the hydraulic load on soils, we will design for a conservative 6 inch ponding depth. The value of h_{max}, should be in feet. Thus, h_{max} = 0.5 feet.

Average height of water above bioretention bed (h_{avg})
The average height of water above the bioretention bed is defined as half the ponding depth. $h_{avg} = h_{max}/2 = 0.25$ feet

Filtration time (t_f)
Ideally, it should take three days for the WQV to filter through the planting soil bed. In this example, t_f = 3 days.

Filter bed surface area (A_f)
The required filter bed surface area is calculated using equation 4.10.

For this example, $A_F = (0.12 \text{ ac-ft}*4 \text{ ft})/((1 \text{ ft d}^{-1})*3 \text{ d }*(0.25 \text{ ft} +4 \text{ ft})) = 0.04$ acres.

Filter bed length (L_f)
At a minimum, the facility should be 40 feet long. Use equation 4.11 to determine the appropriate length.

For this example, $L_f = (87120 \text{ ft}^2 \text{ ac}^{-1} *0.04 \text{ ac})^{1/2} = 57$ feet.

Filter bed width (W_f)
At a minimum, the facility should be 15 feet wide or approximately half the filter bed length. W (feet) = L (feet) /2 = 57/2 = 28.5 feet.

4.3.5.4 Underdrain

Pipe diameter (D_U)
The underdrain pipe diameter should be at least 4 inches to prevent clogging. For this example, we will utilize the 4 inch diameter pipe.

Gravel depth (Z_g)
The depth of the gravel layer above the underdrain pipe should be at least 4 inches greater than the pipe diameter. Thus, the minimum depth for this example should be 8 inches.

Perforation diameter (D_P)
The recommended perforation diameter is 0.375 inches. We will use this recommendation for this example.

Perforation spacing (S_P)
The recommended longitudinal center to center perforation spacing is 6 inches. We will use this recommendation for this example.

Perforations per row (n_P)
A minimum of 4 perforations per row is recommended. We will use the minimum for this example.

Transverse collector pipe spacing (S_{TU})
When the facility width is greater than 20 feet, it will be necessary to install transverse collector pipes that run perpendicular to and connect to the main underdrain pipe. The center to center spacing of the transverse collector pipes should be less than or equal to 10 feet. For this example, we will choose a spacing of 10 feet.

Number of transverse collector pipes (N_{TU})
The number of transverse collector pipes is found using equation 4.12.

For this example, N_{TU} = (57/10) = 5.7. We will use 5 collector pipes.

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

4.3.5.5 Overflow

If the 1-percent event is to pass through the facility, the maximum velocity shall be kept below 3 feet per second to avoid erosion of the soil matrix.

If facilities are designed with a bypass, it shall be designed to safely pass runoff flows from events up to and including the 1 percent event.

The overflow can be designed as a vegetated or stabilized channel or a yard inlet catch basin. Vegetated or stabilized channel overflows shall be designed using Manning's equation or a standard–step backwater method using the energy equation, as appropriate. Overflows designed as open channels shall conform to local agency design criteria for open channels. Overflow inlets shall conform to local agency design criteria for inlet design.

4.3.5.6 Vegetation

Determine the following specific for the bioretention site:

- Soil types (soil tests, soil maps in Appendix B)
- Annual precipitation with dates for wet/dry season (Maps in Appendix A)
- Ecoregion and corresponding vegetation (Map and table in Appendix C)
- Previous land use

Provide the soil type, precipitation, previous land use, and ecoregion information to a native vegetation expert for planting suggestions (vegetation types, seeding rates, establishment procedures, maintenance procedures). Use the "typical vegetation" listed in Appendix C as a guideline to check final list. Native vegetation contacts and links are listed in Appendix C.

4.3.6 Submittal Requirements

Figure 4-13 provides an example of a bioretention plan and profile. For review purposes prior to construction, the following minimum submittal requirements are recommended:

- Drainage area map, including drainage area to bioretention cell(s).

- Existing and proposed contour map of site (1-foot contours recommended). Additional spot elevations may be helpful.
- Geotechnical investigation of site (soil borings, water table location).
- In situ infiltration test of bioretention soil mixture demonstrating infiltration rate of 1 foot/day or higher.
- Stormwater plan/profile for site.
- Bioretention cell plan view 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 cell.
- Detail of any proposed underdrain 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 cell is recommended to confirm actual construction adheres to approved construction plans.
- Long-term inspection/maintenance plan.

4.3.7 References

CASQA. 2003. *California Stormwater Quality Association Stormwater Best Management Practice Handbook.*

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MARC and APWA. 2008. *Manual of Best Management Practices for Stormwater Quality*.

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Section 4
Structural BMPs

4.4 Vegetated Swales

Vegetated swales are open vegetated channels with dense vegetation covering the side slopes and channel bottom. They are used to treat 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, 2008). The vegetation covering the sides and bottom of the channel provide a filtration surface and slows runoff velocities, traps particulate pollutants, and promotes infiltration. Figure 4-15 is a photo of a grass swale located near a roadway.

Sections from this manual that may need to be referenced for additional information are: Section 2; Section 4.1; Section 5.

Location characteristics (Slope, Soil Type)	Slope: < 1-2% Soil Type: All
Contributing drainage area	< 5 acres
Design size	Varies
Detention time for WQv treatment	N/A
Pollutant removal efficiencies[1]	60-85% TSS, 15-90% TP, 10-90% TN, 69-88% Zn, Cu 45-80%
Potential for education and outreach	**Moderate** (recreation, landscaping, wildlife habitat)
Potential for use with other BMPs	**High** Best when used as pretreatment for other BMPs such as bioretention
Implementation Category	Short Term: **Easy** Long Term: **Easy**
Maintenance	**Low** Sediment/debris removal, vegetation upkeep

[1]CRWA, 2008

4.4.1 General Application

Grass swales 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 (CASQA, 2003). Vegetated swales are best utilized in treating areas of 5 acres or less, and are only effective in conveying shallow concentrated flow for water quality benefits. Swales are especially effective when used with a series of stormwater BMP practices, such as when receiving water from a filter strip, or conveying water to a detention pond (See treatment train in Section 2).

Figure 4-15 Grass Swale Located Near a Roadway (Source: US Army Corps)

4.4.2 Advantages and Disadvantages

Advantages	Disadvantages
Improves water quality by filtering stormwater through dense vegetation.	Provides effective water quality control in light to moderate runoff conditions, but control 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 down runoff to surrounding streams and rivers	Is not effective in reducing bacteria levels in stormwater
Minimizes erosion when used with recommended slope requirements (see Section 4.3.4)	Require more maintenance than curb and gutter systems

4.4.3 Design Requirements and Considerations

4.4.3.1 General Guidelines

The main criteria to consider in the water quality design of a vegetated swale are channel capacity in relation to residence time and minimization of erosion (IA State, 2008):

- Runoff velocity shall not exceed 1 foot per second (fps) during the peak discharge associated with the water quality design rainfall event.

- The total length of the swale should provide at least 3 to 5 minutes residence time, with a minimum length of 100 feet.

4.4.3.2 Site Location and Soils

- Grass swales shall be used to treat drainage areas of less than 5 acres.

- The bottom of the channel shall be constructed at least three feet above groundwater to prevent the bottom from remaining moist or contamination of groundwater (Metro Council, 2001).

- In order to provide the best means for plant survival, vegetated swales cannot be constructed in gravelly and coarse sandy soils (MARC, 2008).

- Select vegetation that can withstand relatively high-velocity flows at entrances, and both wet and dry periods (MARC, 2008, 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, 2008).

4.4.3.3 Slope, Shape, and Design

- 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 (IA State, 2008). Installation of check dams is recommended for slopes above 2-percent.

- 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, 2008).

- Swales shall be parabolic or trapezoidal in shape (IA State, 2008; MARC, 2008; Metro Council, 2001). 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 design principles will govern parabolic cross-sections, except for the cross-sectional geometry (IA State, 2008). Figure 4-16 shows a cross-section of a swale.

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

    – The bottom width is a dependent variable in the calculation of velocity based on Manning's equation (Iowa Stormwater Management Manual, 2008).

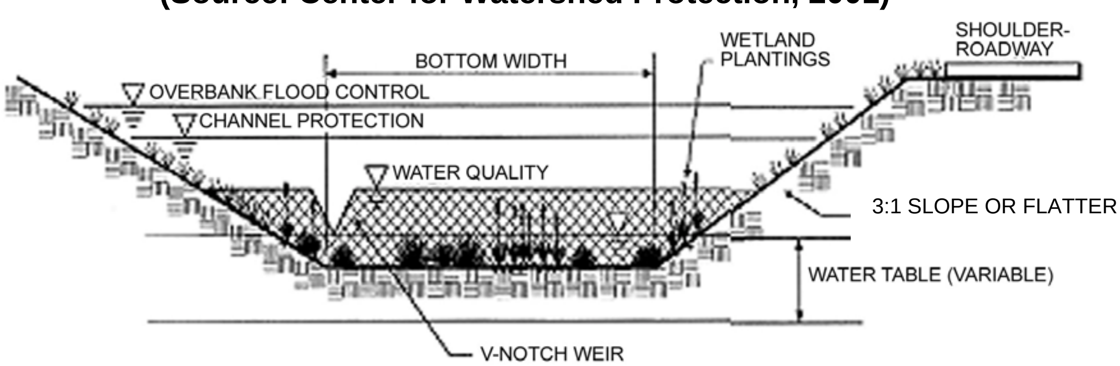
- Generally, swale length is a function of site drainage constraints (IA State, 2008). The minimum longitudinal length of a vegetated swale should be 100 feet to provide adequate water quality treatment (MARC, 2008).

- Identify the swale bottom width, depth, length and slope necessary to convey the water quality flow rate with a shallow ponding depth. The depth should relate to the height of the vegetation used in the swale, as increased water depth would provide conveyance rather than residency time needed for the water quality storm. This depth typically ranges from 1 to 4 inches.

- 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. Typical Manning's roughness coefficients for sheet flow are:

	Manning's Roughness Coefficient "n" for Sheet flow
Short grass prairie	0.15
Dense grasses (weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures)	0.24
Bermuda grass	0.41

Figure 4-16 Example of a Swale Cross Section
(Source: Center for Watershed Protection, 2001)



4.4.4 Design Calculations

- **Step 1:** Find flow (Q) for tributary area to swale for water quality rainfall event using Rational Method.

  Equation 4.6 $Q = C \times i \times A$

- **Step 2:** Solve Manning's equation for a specified variable. For this example, we will calculate bottom width of the swale. This step is most easily accomplished using a spreadsheet or solver program.

- **Step 3:** Solve V = Q/A for velocity using calculated variable and Q calculated in Step 1. If V is greater than 1 ft/s, the width of channel, longitudinal slope of channel, or Manning's n value may need to be adjusted to obtain a velocity less than 1 fps, and therefore appropriate for shallow flow.

- **Step 4:** Calculate minimum swale length for required residency time using L = VT where T is equal to minimum residency time. If the length calculated is less than 100 feet, a minimum length of 100 feet must be specified on construction plans.

- Note that an agency may require that a swale 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 a water quality rainfall event. Additional calculations may be necessary to size the swale for other larger events.

4.4.5 Design Example

A 5 acre site is being developed by a church (C=0.75) in Hutchinson. 0.25 acres of the site will be tributary to a proposed buffalo grass swale, with a Manning's n value of 0.25 and side slopes at 4:1. Assume a time of concentration of 10 minutes to the swale. Assume flow depth in the swale of 2 inches for the water quality event. Proposed longitudinal slope is 2.0-percent. Residency time required for stormwater in swale is a minimum of 5 minutes. Design the swale for the water quality rainfall event.

Step 1: Calculate the water quality rainfall event Q (assume 90-percent) using the Rational Method.

- Q = (0.75)*(1.47 in/hr)*(0.25acre) = 0.28 cfs

Step 2: Using Microsoft Excel solver, a bottom width was calculated using Manning's equation based on the Water Quality Storm Q.

Trapezoidal (4:1) Example Problem							
n	Depth (D) feet	Width (W) feet	Area (A) sqft	Wetted P (ft)	Hydraulic radius (ft)	Long Slope (ft/ft)	Iterated Q
0.25	0.166666667	6.350092724	1.17	7.72	0.15	0.020	0.28

This width was calculated as 6.35 feet. A width of 6.50 feet will be used.

Trapezoidal (4:1) Example Problem							
n	Depth (D) feet	Width (W) feet	Area (A) sqft	Wetted P (ft)	Hydraulic radius (ft)	Long Slope (ft/ft)	Iterated Q
0.25	0.166666667	6.5	1.19	7.87	0.15	0.020	0.29

Step 3: Calculate Velocity.

- V = (0.28 cfs)/(1.19 sq ft) = 0.24 ft/s. This is less than 1 ft/s, and therefore meets the recommendations for the Water Quality Storm.

Step 4: Calculate minimum length of swale based on residence time.

- L = (0.24 ft/s)*(5 min)*(60sec/min) = 70.6 feet. This is less than 100 feet, so L = 100 feet.

Summary: To meet design requirements and recommendations for the Water Quality Storm and the site, a vegetated swale shall be constructed that is 100 feet in length and with a bottom width of 6.5 feet.

4.4.6 Submittal Requirements

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

- Drainage area map, including drainage area to swale.
- Existing and proposed contour map of site (1-foot contours recommended). Compaction requirements should be stated, if required. Additional spot elevations may be helpful.
- Geotechnical investigation of site (soil borings, water table location).
- Stormwater plan/profile for site.
- Swale calculations, including WQv, depth of WQv in swale, and maximum velocity for WQv (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.
- Vegetation plan with schedule for installation and initial maintenance. Appropriate erosion control measures should be included.
- An as-built survey of the swale is recommended to confirm construction adheres to approved construction plans.
- Long-term inspection/maintenance plan.

4.4.7 References

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Section 4
Structural BMPs

4.5 Filter Strips

A filter strip is an area of planted or indigenous dense vegetation that accepts sheet flow runoff from adjacent surfaces. When situated properly between a pollution source and a water body that receives runoff, filter strips slow runoff velocities and improve water quality. Filter strips improve water quality by reducing sediment load and filtering pollutants absorbed to sediments. Water treatment with filter strips is most effective when sheet flow is maintained. Runoff from adjacent impervious surfaces will often concentrate and form a channel, reducing the effectiveness of the filter strip (Muthukrishnan et al. 2006). These flows must be converted to sheet flow prior to entering a filter strip treatment area. In order to achieve this, grading and level spreaders are often necessary to create a uniformly sloping area to distribute the runoff evenly across the filter strip (IA State, 2008). Figure 4-17 provides an example of filter strips.

Sections from this manual that may need to be referenced for additional information are: Section 2; Section 5.

Location characteristics (Slope, Soil Type)	Slope: < 1-6% Soil Type: All
Contributing drainage area	< 2 acres
Design size	Minimum: L 15'
Detention time for WQv treatment	N/A
Pollutant removal efficiencies[1]	90%TSS[1], 20% TN, 20% TP, 40% Heavy Metals[2]
Potential for education and outreach	High (recreation, landscaping, wildlife habitat)
Potential for use with other BMPs	Best when used as pretreatment for other BMPs such as bioretention
Implementation Category	Short Term: **Easy** Long Term: **Easy** (See Section 5.4.1)
Maintenance	**Low** Sediment/debris removal, vegetation upkeep (See Section 5.4.1)

[1]Gharabaghi et al., 2000, [2]IAState, 2008

Figure 4-17 Filter Strip

4.5.1 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 (USEPA, 2006). Filter strips are frequently used as a pretreatment system for stormwater destined for other BMPs such as an infiltration trench or bioretention systems (Metro Council, 2001). See Section 2 for information on BMPs in treatment trains.

4.5.2 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 cycle can be used as erosion and sediment control	Applications of fertilizers, herbicides, and pesticides on FS may be a source of pollutants in runoff
Substantial capture of sediment and pollutants are adsorbed onto particles	Potential failure when concentrated flows with erosive velocities develop and "short circuit" the filter strip.

4.5.3 Design Requirements and Considerations

The following guidelines shall be considered when designing filter strips:

4.5.3.1 General Guidelines

- Filter strips shall be designed to accept sheet flow runoff from small drainage areas (1 to 2 acres). Concentrated flows must be redistributed or unconcentrated prior to entering the filter strip (Metro Council, 2001).

- Where applicable, vegetated filter strips should be utilized as a pre-treatment component for structural BMPs such as bioretention areas.

- Sheet flow runoff from paved surfaces shall be limited to maximum lengths shown in Table 4-6.

- 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 shall 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 feet, with typical parking block widths ranging from 6 to 8 feet. Parking blocks should be spaced to allow a minimum of 2 feet width between them. Where parking blocks are used, a minimum additional 2 feet of surface beyond the parking block is recommended for flow to unconcentrate prior to entering the filter strip. The additional surface required will vary based on the parking lot slope toward the filter strip.

- Curbs and curb cuts are not permitted adjacent to a filter strip.

4.5.3.2 Site Location and Soils

- Filter strips shall be positioned at least two feet above the water table. Filter strips should be separated from the groundwater by between two and four feet to prevent contamination (Muthukrishnan et al. 2001).

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

- Filter strips should not be used on soils that cannot sustain a dense grass cover with high retardance (IA State, 2008).

- Designers shall 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.

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

4.5.3.3 Slope

- Filter strip slopes shall 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. This trench will act as a pretreatment device and as a level spreader (IA State, 2008).

- 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. 2001).

- A berm of sand and gravel can be designed at the toe of the slope to provide an area for temporary shallow ponding. This berm could include outlet pipes or an outflow weir.

4.5.3.4 Shape and Design

- The maximum length of pavement in the direction of flow draining to a filter strip can be determined using pavement slope and rainfall intensity, based on the 10-year storm. Refer to Table 4-6 for guidelines in determining pavement length.

- Filter strip length in the direction of flow shall be determined based on the slope of the filter strip and water quality event rainfall intensity, using the time of concentration for the drainage area to the filter strip. Refer to Table 4-7 for guidelines in determining filter strip length.

- The filter strip should stretch the entire length of the adjoining impervious surface where the stormwater originates (Muthukrishnan et al. 2001).

- Filter strips must be a minimum of 15 feet in length in the direction of flow to effectively treat run-off, greater lengths will enhance treatment (IA State, 2008).

4.5.4 Design Calculations

- **Step 1:** Calculate the time of concentration of the area draining to the filter strip using equation 4.8. This value should a minimum of 5 minutes.

  Equation 4.8
  $$T_I = \frac{1.8(1.1 - C)D^{1/2}}{S^{1/3}}$$

- **Step 2:** Find the 10-year rainfall intensity at the duration equal to the time of concentration using Appendix A.

- **Step 3:** Use Table 4-6 to find the maximum pavement length (PLmax) that can drain to the filter strip, based on intensity from Step 2 and proposed slope of the drainage area to the filter strip. Revise proposed length and area draining to the filter strip if necessary.

Table 4-6 Maximum Pavement Length in Feet (n=0.011) Allowable for a Given Pavement Slope

Drainage Area Slopes (%)	10 Year Rainfall Intensity (in/hr)*											
	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5
0.5	162	130	109	93	81	73	65	59	55	50	46	44
1.0	136	109	91	78	68	61	55	50	46	42	39	37
1.5	100	80	67	58	50	45	40	37	34	31	29	27
2.0	81	65	54	47	41	36	33	30	27	25	24	22
2.5	69	55	46	39	35	31	28	25	23	21	20	19
3.0	60	48	40	34	30	27	24	22	20	19	17	16
3.5	53	43	36	31	27	24	22	20	18	17	16	15
4.0	48	39	32	28	24	22	20	18	16	15	14	13

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

Table 4-7 Minimum Filter Strip Length (n=0.24) for a Minimum Travel Time = 3 Minutes

Filter Strip Slopes (%)	WQ Event Rainfall Intensity (in/hr)*																	
	0.4	0.5	0.6	0.7	0.8	0.9	1	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2	2.1
0.5	12	13	15	17	18	20	21	22	24	25	26	27	28	30	31	32	33	34
1.0	16	19	21	23	25	27	29	31	33	35	37	38	40	42	43	45	46	48
1.5	20	23	26	28	31	34	36	38	41	43	45	47	49	51	53	55	57	59
2.0	23	26	30	33	36	39	41	44	47	49	52	54	56	59	61	63	65	68
2.5	25	29	33	37	40	43	46	49	52	55	58	60	63	66	68	71	73	76
3.0	28	32	36	40	44	47	51	54	57	60	63	66	69	72	75	77	80	83
3.5	30	35	39	43	47	51	55	58	62	65	68	71	75	78	81	84	86	89
4.0	32	37	42	46	50	54	58	62	66	69	73	76	80	83	86	89	92	95
4.5	34	39	44	49	53	58	62	66	70	74	77	81	84	88	91	95	98	101
5.0	36	41	46	51	56	61	65	69	74	78	81	85	89	93	96	100	103	107
5.5	37	43	49	54	59	64	68	73	77	81	85	89	93	97	101	104	108	112
6.0	39	45	51	56	62	66	71	76	80	85	89	93	97	101	105	109	113	117

*Water quality rainfall event intensity should be used with a duration equal to the time of concentration for the drainage area to the filter strip.

- **Step 4:** Find the water quality rainfall event intensity at the duration equal to the time of concentration, using the time of concentration calculated in Step 1, using Appendix A.

- **Step 5:** Use Table 4-7 to find the minimum filter strip length required, based on the intensity from Step 4 and the proposed slope of the filter strip area in the direction of flow. Compare to site plan. Revise proposed length of the filter strip to meet minimum requirement if necessary.

4.5.5 Design Example

A 1 acre site is being developed by a small business (C=0.80) in Winfield. Approximately 0.20 acres 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 stip.

Figure 4-18 Site Plan of 1 Acre Small Business Site in Winfield, KS.

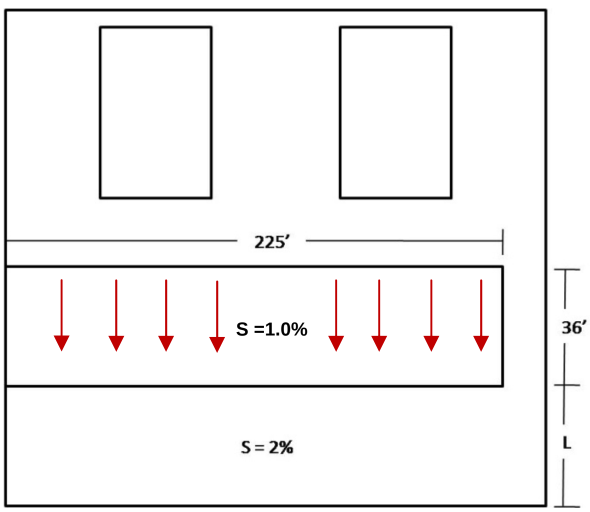

Figure 4-18 is the site plan for the proposed small business.

4.5.5.1 Time of Concentration (T_I)

T_I is found using Equation 4.8. $T_I = (1.8*(1.1-0.8)*36^{1/2})/(1.5)^{1/3} = 2.8$ min. The minimum time of concentration should be 5 minutes. Therefore, for this example, use $T_I = 5$ minutes.

4.5.5.2 Ten Year Rainfall Intensity (I_{10})

I_{10} can be found using the graph of Rainfall Intensity Curves in Appendix A. For Winfield, KS at 5 minutes, this value is 7.5 inches per hour.

4.5.5.3 Maximum Pavement Length (PL_{MAX})

PL_{MAX} can be found using Table 4-6 and finding the maximum pavement length for a drainage area slope of 1.0-percent and a rainfall intensity of 7.5 inches per hour. For this example, PL_{MAX} is 37 feet. Since proposed pavement length of 36 feet is less than PL_{MAX}, 36 feet can be used.

4.5.5.4 Water Quality Event Intensity (WQ_I)

WQ_I can be found using the Water Quality Event Curves for the 90-percent event. Using the duration of 5 minutes and the 90-percent plot, the WQ_I is 1.7 inches per hour.

4.5.5.5 Minimum Filter Length (FS_{MIN})

Use Table 4-7 to find the minimum length for a filter strip slope of 2-percent and a rainfall intensity of 1.7 inches per hour. For this example, FS_{MIN} is 59 feet. 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 feet. The result is a 60 feet filter strip at 2 -percent.

4.5.6 Submittal Requirements

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

- Drainage area map, including drainage area to filter strip area.
- Existing and proposed contour map of site (1-foot contours recommended). Compaction requirements should be stated, if required. Additional spot elevations may be helpful.
- Geotechnical investigation of site (soil borings, water table location).
- Stormwater plan/profile for site.
- Site plan view. Components clearly labeled with dimensions.
- Hydrologic 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 is recommended to confirm actual construction adheres to approved construction plans.
- Long-term inspection/maintenance plan.

4.5.7 References

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

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Section 4
Structural BMPs

4.6 Infiltration Trench

An infiltration trench is an excavated trench, typically between 3 and 12 feet deep, filled with coarse granular material, and lined with filter fabric (MARC, 2008). Infiltration trenches are used to collect stormwater runoff for temporary storage and infiltration. Infiltration trenches can be constructed for conveyance and/or infiltration purposes. Trenches used for conveyance purposes receive runoff through pipes or trenches, while trenches collecting sheet flow are used for primarily infiltration purposes. In all infiltration trenches, runoff is stored in the spaces between the gravel and infiltrates through the bottom of the trench and into the soil matrix. By doing so, the trench not only treats the WQv, but also helps preserve the natural water balance on a site by recharging groundwater and preserving baseflow (IA State, 2008). Infiltration trenches are often combined with another BMP such as a filter strip, swale, or detention basin in a treatment train. These pre-treatment BMPs are highly recommended because they limit the amount of course sediment entering the trench. Sediments can clog the trench making it ineffective (IA State, 2008). Infiltration trenches can remove suspended solids, particulates, bacteria, organics, soluble metals, and nutrients through mechanisms of filtration, absorption, and microbial decomposition (MARC, 2008). Figure 4-19 is an example of the surface view of an infiltration trench.

Sections from this manual that may need to be referenced for additional information are: Section 2; Section 4.1; Section 5.

Location characteristics (Slope, Soil Type)	Slope: < 15% Soil Type: A
Contributing drainage area	< 5 acres
Design size	3-12' deep
Detention time for WQv treatment	1-3 days
Pollutant removal efficiencies[1]	80% TSS, 60% TP and TN, 70-80% BOD
Potential for education and outreach	**Low**
Potential for use with other BMPs	**High** Pre-treatment Swale, Filter Strip
Implementation Category	Short Term: **Easy** Long Term: **Difficult** (See Section 5.4.1)
Maintenance	**Medium** Sediment/debris removal, potential clogging (See Section 5.4.1)

[1]IA State, 2008.

Figure 4-19 Surface View of an Infiltration Trench (Photo: MARC, 2008)

4.6.1 General Application

Infiltration trenches are best suited for use in residential subdivisions, small commercial lots, and parking lots. Infiltration trenches may be too space consuming for densely populated areas where underdeveloped land is scarce. They also cannot be used to treat highly contaminated runoff (MARC, 2008).

Infiltration trenches promote groundwater recharge, but the possibility for groundwater contamination must be considered where groundwater is a source of drinking water. In all circumstances, infiltration trenches should be located in areas with highly porous soils where the bedrock and/or water table are located at least four feet below the bottom of the trench (IA State, 2008). The main variable in determining trench depth is to avoid groundwater contamination.

Due to potential failure as a result of sediment clogging, infiltration trenches also need to be located at sites where upstream sediment control can be ensured (IA State, 2008).

4.6.2 Advantages and Disadvantages

Advantages	Disadvantages
Can remove up to 95-percent of suspended solids	Susceptible to clogging by sediment, necessitating frequent maintenance
Removes fine sediment, trace metals, bacteria, and oxygen-demanding substances	Risk of polluting groundwater depending on soil conditions and groundwater depth
Appropriate for small sites with porous soils	No natural components so no improvement with time
Provide groundwater recharge and preservation of stream baseflow	Cannot be used where soil infiltration rates are < 0.5 in/hr

Advantages	Disadvantages
Can be utilized below ground through pipes or channels	Restricted in karst areas (topography characterized by layers of soluble bedrock)
Fit in small spaces and can be utilized in retrofit situations	Little contribution to aesthesis and no contribution to wildlife habitat
Peak flow mitigation	

4.6.3 Design Requirements and Considerations

4.6.3.1 General Requirements

There are important site requirements to consider in the design of an infiltration trench. These include:

- Infiltration trenches should be designed to capture the WQ_V while the remaining runoff from large events bypasses the trench. The overall volume of the trench is dependent upon the water quality storm volume of runoff entering the trench from the contributing watershed (MARC, 2008). Soil infiltration rates will also be an important factor in determining trench volume.

- It is best to use multiple pretreatment techniques together with infiltration trenches to eliminate potential clogging and to extend the lifespan of the trench. It is recommended that a grass filter strip be installed upslope of the infiltration trench to help remove sediments before reaching the infiltration trench.

- Trenches shall be designed to provide a detention time of 6 to 24 hours for the water quality storm (MARC, 2008).

- The contributing drainage area to any infiltration trench should be less than five acres.

- Cold weather can limit the use of trenches. Winter sanding can clog trenches and winter salting increases the potential for chloride contamination of groundwater (IA State, 2008). In areas subject to freezing temperatures, designers shall ensure that part of the trench is constructed well below the frost line. Ensure that plowed snow is not stored on top of infiltration trench. Infiltration trenches can operate effectively in colder climates if effectively operated and maintained.

- Plans shall include a geotechnical evaluation at the site (EPA, 1999).

4.6.3.2 Location and Soils

- Infiltration trenches are suitable to capture sheet flow or function as an offline device. They can be situated in medium to high-density residential areas (IA State, 2008).

- When used in an offline configuration, the WQ_V shall be diverted to the infiltration trench through the use of a flow splitter (IA State, 2008).

- Trenches shall be located at least 150 feet away from drinking water wells in order to decrease the chance for groundwater contamination. In addition, they shall be 100 feet from building foundations (Metro Council, 2001).

- The underlying soils must meet the soil screening criteria with an infiltration rate of 0.5 in/hr or greater (EPA, 1999).

- Acceptable soil texture classes are: sand, loamy sand, sandy loam and loam. These soils are in the A or B hydrologic group. Trenches shall not be constructed on soils in the C or D hydrologic group (EPA, 1999).

- Soils reports from the Soil Conservation Service shall be used to identify soil type. Sufficient soil borings shall be taken to verify site conditions.

- The seasonally high water table must be at least four feet below the bottom of the infiltration trench (IA State, 2008).

- The drainage area (5 acres or less) must be fully developed and stabilized with vegetation prior to construction in order to avoid high sediment loads (EPA, 1999).

4.6.3.3 Slope

- The drainage area slope determines runoff velocity. Locate infiltration trenches where up-gradient slopes are 5-percent or less. The down-gradient slope should be less than 15-percent to minimize slope failure and seepage (IA State, 2008).

- The slope of the surrounding area should allow runoff to enter the trench as sheetflow. Runoff can be captured by depressing the surface of the trench or by placing a berm at the down-gradient side of the trench (IA State, 2008).

4.6.3.4 Design Specifications

- If stormwater is conveyed as channel flow, maximize the length of the trench parallel to the direction of flow.

- The storage volume of the trench shall be equal to the WQv. Infiltration trenches shall be designed to fully dewater within 24 hours following a rainfall event (IA State, 2008).

- The sides and bottom of the trench shall be lined with filter fabric. In addition, a layer of nonwoven filter fabric or sand shall be placed 6-12 inches below ground surface to prevent suspended solids from clogging the majority of the storage media (MARC, 2008).

- The bottom slope of the trench should be flat in order to evenly distribute flow and encourage uniform infiltration (IA State, 2008).

- Fill the infiltration trench with clean, washed stone with a diameter of 1.5 to 3 inches (void space of 38 to 42-percent). By washing stone prior to installation, fine particles are removed from the stone that could potentially cause clogging. Top the trench with stone aggregate, pea gravel, or large stones.

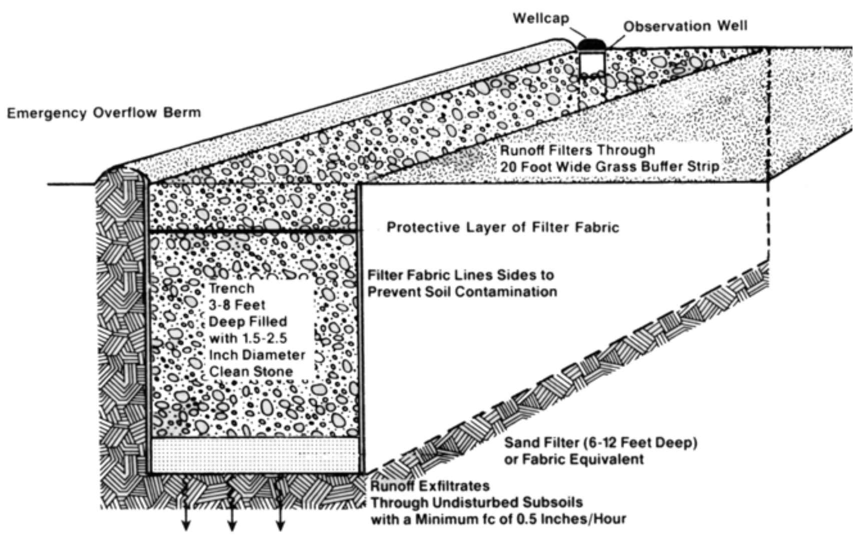
- Do not use limestone or shale as aggregate material in the trench as it may cement over time (MARC, 2008).

- An observation well must be located at the center of the trench to monitor water drainage from the system.

- The well can be a 4 to 6 inch diameter PVC pipe with a lockable cap. The well shall be either 6 inches above ground or flush with the ground (IA State, 2008).

Figure 4-20 provides a schematic of a typical infiltration trench.

Figure 4-20 Infiltration Trench Design (Source: Schueler, 1987)



*While the trench depth in this example is stated as 3-8 feet deep, overall trench depth may be a maximum of 12 feet deep. The aggregate used to fill the trench can vary between 1.5-3 inches. (IA State, 2008).

4.6.4 Design Example

This example outlines the design requirements of an infiltration trench in Dodge City, Kansas. The trench is constructed at ground surface and collects sheet flow from the neighboring drainage area. The total drainage area is 1.0 acres, 60-percent covered with an impervious parking lot. The high water table was found to be 9 feet below ground surface.

CRITERIA	SITE STATUS
Infiltration rate $\geq$ 0.5 in/hr	Infiltration rate is 0.5 in/hr, on Type A soil
Up-gradient slope < 5%	Slope is 1-percent
High levels of pollution runoff should not be infiltrated	Not industrial land use
Infiltration prohibited in karst topography	Not in karst topography
Bottom of infiltration trench must be vertically separated from the high water table by 4 feet	The high water table was found to be 9 feet below ground surface. Thus, the maximum trench depth is 5 feet.
Maximum contributing area $\leq$ 5 acres	Contributing area is 3 acres
Infiltration trenches must be located 150 feet horizontally from any water supply well.	No water supply wells within 150 feet
Setback 100 feet from structures	Trench is 100 feet from the parking lot.

Step 1: Compute Water Quality Volume

Equation 4.1: Rv = 0.05 + I (0.009)

Therefore, for this example Rv = 0.05 + 60.0 (0.009) = 0.59

Step 2: Compute WQ_V

Equation 4.2:
$$WQ_V = \frac{P_{WQ} \times R_V \times A_T}{12}$$

Therefore, for this example WQ_V = (0.79-inch)*(0.59)*(1.0ac/12-inch)
WQ_V= 0.039 acre-feet = 1698ft^3

Step 3: Find the minimum infiltration trench volume (V_{TRMIN}) based on the WQ_V and the void space of the aggregate to be used in the trench (n). For this example n equals 40-percent.

$$V_{TRMIN} = \frac{WQ_V}{n} = \frac{1698\ ft^3}{0.40} = 4{,}245\ ft^3$$

This volume should be multiplied by a factor of 1.2 to account for possible loss of volume due to sedimentation.

$$V_{TR} = V_{TRMIN} * 1.2 = 4{,}245 * 1.2 = 5{,}094\ ft^3$$

Find the minimum surface area of the trench (A_{TR}).

$$A_{TRMIN} = \frac{12 \times WQ_V}{P_{SOIL} \times t}$$

Where:

P_{SOIL} = Percolation rate of soil (inch per hour)

t = Trench Retention Time (hour)

With P_{SOIL} equals 0.5 inch per hour for type A soils, and a desired infiltration time of 18 hours.

$$A_{TRMIN} = \frac{12 \times 1698}{0.5 * 18} = 2{,}264\ ft^2$$

Find the minimum trench depth (D_{TMIN}).

$$D_{TRMIN} = \frac{V_{TR}}{A_{TR}} = \frac{5094\ ft^3}{2264\ ft^2} = 2.25\ ft$$

For this example, the trench must be between 2.25 feet and 5 feet (based on the location of the water table.)

Based on the minimum surface area, a length and width for the trench can be established. Widths should not exceed 25 feet. For this example, we will assume a trench width of 6 feet.

$$L = A_{MIN} / W = \frac{2264}{6} = 377\ ft$$

Step 4: General Infiltration Trench Design Specifications

Filter Fabric

The sides and bottom of the trench shall be lined with filter fabric and a layer of filter fabric shall be added one foot below the trench surface. Filter fabric placed one foot below trench surface will maximize pollutant removal and decrease pollutant loading in the trench bottom (IA State, 2008).

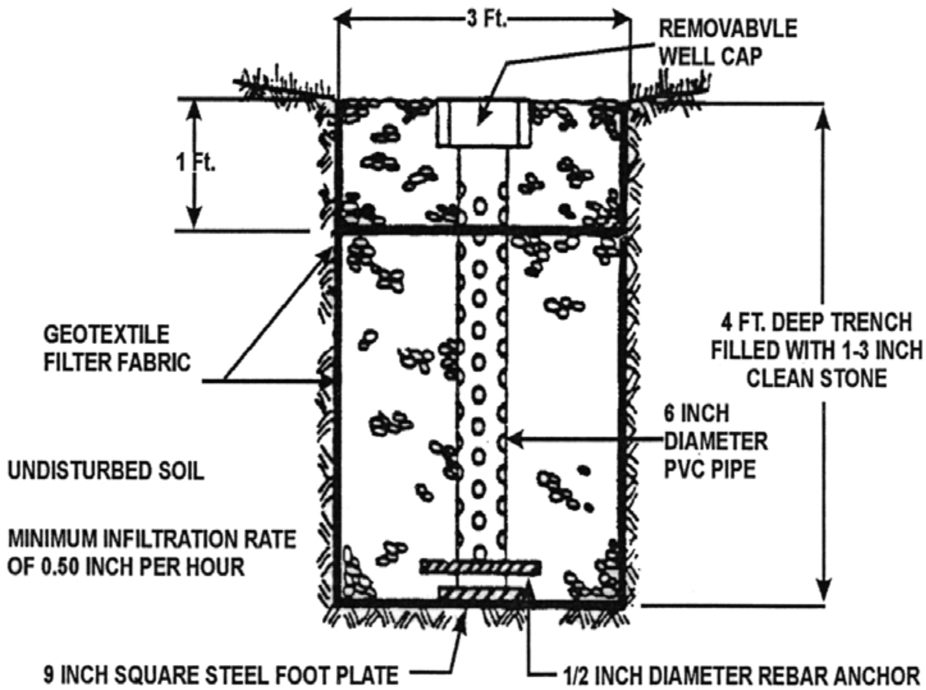
Aggregate

At the designer's discretion, a 4 to 6-inch layer of clean, washed sand or medium aggregate concrete sand can be placed in the bottom of the trench in lieu of filter fabric. The trench shall be filled with washed stone aggregate 1.5 to 3 inches in diameter. By washing aggregate/stone prior to installation, fine particles are removed from the stone that could potentially cause clogging. Limestone or shale should not be used. Pea gravel may be substituted for the top one foot of stone aggregate in the trench. Pea gravel shall be #8 to 3/8-inch (IA State, 2008).

Observation Well

An observation well should be installed. The well shall consist of a 4 to 6–inch diameter PVC tube with a screw-top lid and lockable cap. It shall be anchored to a footplate at the bottom of the trench, and shall be located at the longitudinal center of the trench (Metro Council, 2001). Refer to Figure 4-21 for an example of observation well design.

Figure 4-21 Observation Well Details



Source: Southeastern Wisconsin Regional Planning Commission, 1991.

4.6.5 Submittal Requirements

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

- Drainage area map, including drainage area to infiltration trench.
- Existing and proposed contour map of site (1-foot contours recommended). Compaction requirements should be stated, if required. Additional spot elevations may be helpful.
- Geotechnical investigation of site (soil borings, water table location). Should include a percolation test at the total trench depth.
- Stormwater plan/profile for site.
- Site plan view. Components clearly labeled with dimensions.

- Cross section detail of proposed trench with dimensions for construction. Include appropriate design calculations (refer to Design Example). Include calculations and details for diversion structures if the trench will be used for conveyance.

- Erosion and sediment control measures.

- An as-built survey is recommended to confirm actual construction adheres to approved construction plans.

- Long-term inspection/maintenance plan.

4.6.6 References

EPA Office of Water, 1999. *Stormwater Technology Fact Sheet.*

Atlanta Regional Commission (ARC). 2001. *Georgia Stormwater Management Manual.* Available atwww.georgiastormwater.com

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Section 4
Structural BMPs

4.7 Extended Dry Detention

Extended dry detention basins (EDDBs) are designed to detain the stormwater water quality volume (WQv) for 40 hours to allow particles and associated pollutants to settle (UDFCD, 2005). This attenuation of stormwater reduces the peak stormwater runoff rate for all stormwater events. Unlike extended wet detention basins, these facilities do not maintain a permanent pool between storm events (CASQA, 2003). EDDBs may develop wetland vegetation and 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). Figure 4-22 shows an example of an extended dry detention basin.

Sections from this manual that may need to be referenced for additional information are: Section 2; Section 4.1; Section 5

Location characteristics (Slope, Soil Type)	Slope: < 15% Soil Type: All
Contributing drainage area	10-50 acres (75 acres absolute maximum)[2]
Design size	Minimum: L:W 2:1-4:1, D 2 feet
Detention time for WQv treatment	40 hrs[1]
Pollutant removal efficiencies[1]	50% TSS, 10% TN, TP[3]
Potential for education and outreach	**Low** Not attractive, usually decentralized location
Potential for use with other BMPs	**Moderate** Pretreatment required for TSS removal
Implementation Category	Short Term: **Easy** Long Term: **Easy** (See Section 5.4.1)
Maintenance	**Low** Sediment/debris removal, vegetation upkeep (See Section 5.4.1)

[1]MARC, 2008, [2]IAState, 2008, [3]EPA, 2006

Figure 4-22 Extended Dry Detention Basin Located at an Industrial Location

Source: NCDENR Stormwater BMP Manual

4.7.1 General Application

EDDBs can be used to improve stormwater runoff quality and reduce peak stormwater runoff rates. By providing extra storage above the required extended detention volume, an EDDB can also be used for flood control. Twenty-four hours 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 to trap sediment from construction activities within the tributary drainage area (temporary sediment basins). The accumulated sediment should be removed after upstream land disturbances cease and the tributary area stabilized. The basin should be restored to design conditions for long term use (MARC, 2008). To enhance the removal of soluble nitrogen and phosphorus, it is recommended that a shallow permanent pool is maintained with wetland vegetation (Muthukrishnan et al. 2006).

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).

4.7.2 Advantages and Disadvantages

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

4.7.3 Design Requirements and Considerations

Extended dry detention design shall be by a registered Professional Engineer in the State of Kansas. All design calculations and construction drawings shall be sealed and signed.

4.7.3.1 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 below.

Other infiltration BMPs should be considered in areas with high quality and/or well drained soils (Pennsylvania Stormwater Manual, 2006).

- EDDBs are appropriate in areas where pollutant removal and water quality are secondary to peak volume management.

- A maintenance ramp and perimeter access shall be included in the design to facilitate access to the basin for maintenance activities (CASQA, 2003).

- Public safety shall be considered in EDDB design. Fences and landscaping can be used to impede access to the facility, but should not impede sheet flow into the system. Limit access to outfall pipes (CASQA, 2003).

- The EDDB bottom should be 1 to 2 feet above the seasonal maximum groundwater table, as groundwater may surface within the basin or contribute baseflow to the basin (UDFCD, 2005).

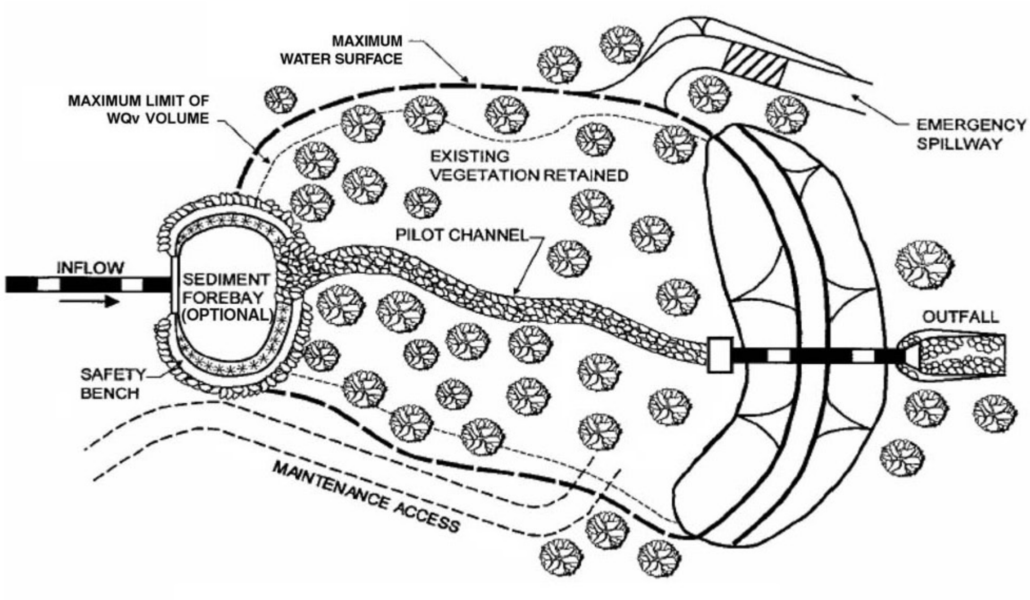
- Design EDDBs to deter large numbers of geese from gathering in the facility. Geese can add to the nutrient and fecal coli form loads entering and leaving the facility. Planting a buffer of trees, shrubs, and native ground cover around the EDDB can help discourage resident geese populations (MARC, 2008).

4.7.3.2 Basin Dimensions

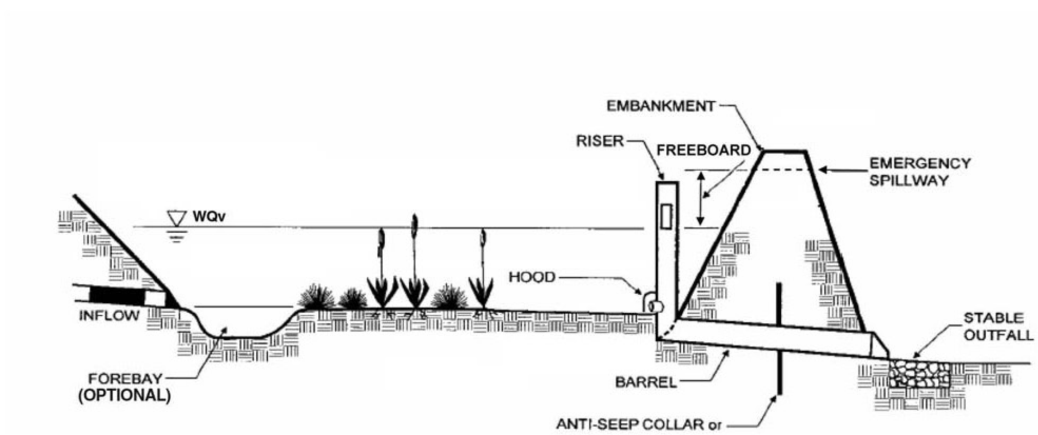
To determine the required storage volume of an EDDB, calculate the WQv based on the drainage area and add 20 percent to the result (See section 4.6.6.1). The basin shall be sized to treat this volume over 40 hours. The additional volume will promote silt and sediment deposition in the EDDB. This will 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 WQv calculated and other site characteristics. General guidelines are outlined below. Figure 4-23 shows a schematic of an extended dry detention basin.

- Basin depths shall be between 2 to 5 feet as a shallow basin with large surface area performs better than a deep basin with the same volume (Nashville, 2000).
- Side slopes should range from 20:1 to 4:1.
- The flow length to width ratio shall be at least 2:1 (Muthukrishnan et al. 2006), but 3:1 minimum is recommended. The width should gradually increase from the inlet area and then retract near the outlet area to ensure adequate detention time.
- Dams that are greater than 10 feet in height but do not fall into state or federal requirement categories shall be designed in accordance with the latest edition of SCS Technical Release No. 60, *Earth Dams and Reservoirs*, as Class C structures (KCMetro APWA, 2006).
- When flood storage for the 1 percent storm is included the EDDB design must provide protection for facility embankments. Each dam should be protected with an emergency spillway unless the principal spillway is large enough to pass the peak flow of the 1-percent storm without breaching the dam (NRCS, 2000).

Figure 4-23 Schematic of an Extended Dry Detention Basin



Plan



Source: Maryland DOE, 2000

4.7.3.3 Basin Configuration

The inlet of the basin shall 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 runoff entry into the main basin and reduces overall maintenance. It is more cost-effective to remove sediments and trash from a small, easily accessed forebay than the large basin. The outlet should be designed to release the captured runoff over the 40 hour 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, 2008).
- **Forebay**
- The forebay shall be a 4 to 6 foot deep cell delineated by a barrier and shall be sized to contain at least 10 percent of the WQv.
- The minimum length to width ratio shall be greater than 2:1 to prevent short-circuiting (Muthukrishnan et al. 2006).

Outlet

- Locate basin outlet as far away from basin inlet as possible to prevent water from short-circuiting the facility (Nashville, 2000).
- Outflow structures shall be protected by well screen, trash racks, grates, stone filters, or other approved devices to ensure that the outlet works will remain functional and not experience blockage or clogging (KC Metro APWA, 2006).
- No single outlet orifice shall be less than 4 inches in diameter (smaller orifices are more susceptible to clogging). If the calculated orifice diameter necessary to achieve a 40-hour drawdown is less than 4 inches, a perforated riser, orifice plate, or v-notch weir shall be used instead of a single orifice outlet (MARC, 2008).
- Keep perforations larger than 1 inch when using orifice plates or perforated risers. Smaller orifice sizes may be used if the weir plate is placed in a riser manhole in a sump-like condition (MARC, 2008) or is protected by a well screen.

4.7.3.4 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 initiate slower velocities. Vegetation surrounding the outlet may serve as a buffer for the BMP to reduce runoff impacts on downstream areas. Information about the establishment and maintenance of native vegetation is outlined in Section 5 of this manual.

4.7.4 Design Calculations

A short summary of the design calculations is presented below. A detailed design example is presented in Section 4.7.5.

- **Step 1** Determine WQ_V based on drainage area and regional precipitation information according to Equations 4.1 and 4.2.

Equation 4.1 $$R_v = 0.05 + 0.009(\mathrm{I})$$

Equation 4.2 $$WQ_V = \frac{P_{WQ} \times R_V \times A_T}{12}$$

Where:
To obtain basin design volume, V_{DESIGN}, multiply WQ_V by 1.2 to account for sedimentation (approximately 20-percent of the WQ_V)

- **Step 2** Determine the outlet type (single orifice, perforated riser, or v-notch weir), outlet loads, and required outlet dimensions. If the diameter calculated for a single orifice is less than 4 inches, use a perforated riser or v-notch weir outlet to prevent clogging. Use equation 4.16 to determine the outflow rate. Equations specific to the outlet type are presented in Appendix G.

- **Step 3** Size trash racks according to outlet type and size. Equations specific to the outlet type are presented in Appendix G.

Equation 4.16 Water Quality Outflow Rate

$$Q_{WQ} = \frac{43{,}560\, ft^2 \times WQ_V}{40 hrs \times 3{,}600 s}$$

Where:

Q_{WQ} = Average water quality outflow rate (cfs)
WQ_V = Water quality volume (acre feet)

- **Step 4** Design the forebay based on WQ_V and minimum depth requirements. The forebay volume should be greater than 10-percent of the WQ_V (Equation 4.22). The forebay surface area is calculated using Equation 4.23.

Equation 4.22 Forebay Volume

$$V_{FB} > 0.1\, WQ_V$$

Equation 4.23 Forebay Surface Area

$$A_{FB}(ac) = \frac{V_{FB}(ac - ft)}{Z_{FB}(ft)}$$

- **Step 5** Determine basin shape, basin side slopes, and dam embankment side slopes.

- **Step 6** Install appropriate vegetation using methodology provided by local native vegetation experts.

4.7.5 Design Example

A single-family housing development (65-percent impervious) is being built on previously undeveloped land in Hays. The developer is required to design and build an EDDB to accept runoff from the 50 acre tributary drainage area and provide and outlet device that will release the WQv within 40 hours of the WQ event. The majority of soil in the development has high-clay content. The land slopes are less than 10-percent across the development area. Refer to Appendix G for example calculations.

4.7.5.1 Basin Water Quality Volume (Step 1)

Determine the tributary area to the EDDB (A_T)

The tributary area, A_T, is 50 acres. Due to the fact that the percent imperviousness is already known, we can utilize the Short-Cut Hydrology Method.

Calculate the Rv based on equation 4.1

The tributary area is 65-percent impervious. Thus, $R_v = 0.05 + 0.009(65) = 0.635$.

Calculate the WQv based on equation 4.2

For Hays, KS, the water quality event is 0.9 inches. WQ_V = (0.9 inch*0.635*50 acre)/12 inches = 2.38 acre feet.

$V_{DESIGN} = (1.2)*WQ_V = 2.86$ acre feet

4.7.5.2 Water Quality Outlet (Step 2)

For this example, we will use a single orifice for an outlet structure. Equations associated with this outlet structure are presented in Appendix G. If the orifice diameter required to drain the excess to the permanent pool is less than 4 inches, a perforated riser or v-notch weir should be used (MARC, 2008).

Water quality depth (Z_{WQ})

Set the depth above the WQ_V outlet (Z_{WQ}) based on facility dimensions for surface area and desired depth.

$Z_{WQ} = 3$ feet

Average WQ_V head (H_{WQ})

The average head is half of the depth above the WQ_V outlet.

For this example, $H_{WQ} = 0.5(3 \text{ ft}) = 1.5$ feet.

Outflow rate (Q_{WQ})

Calculate the average outflow rate that results from the WQ_V exiting the system over 40 hours using Equation 4.16.

For this example, Q_{WQ} = ((2.38 acre feet)*0.3025 =0.72 cubic feet per second (cfs)

Orifice discharge coefficient (C_O)

Set orifice coefficient (Co) depending on orifice plate shape. For this example C_O = 0.62.

Orifice Diameter (D_O)

Calculate the diameter using the Equation G.1 and assuming a C_O = 0.62.

$D_O = 24*((0.72\ \text{cfs})/(0.62*\pi*(2*32.2\ (\text{ft}^2/\text{s})*1.5\ \text{ft})^{1/2})^{1/2}) = 4.7$ inches. Due to the fact that this diameter is greater than 4 inches, a single orifice outlet will provide adequate drawdown configurations.

4.7.5.3 Flood Control

If designing the EDDB for flood control, follow local agency guidelines for detention basins.

4.7.5.4 Trash Racks (Step 3)

The trash racks protect outlet structures from damage resulting from trash and debris. Calculations are based on the outlet type used. Reference Appendix G for outlet type specific equations.

Outlet Area (A_{OT})

Calculate the water quality outlet area based on the orifice diameter using equation G.9.

For this example, $A_{OT} = (\pi/4)*(4.7\ \text{inches})^2 = 17.0\ \text{inches}^2$.

Open Area (A_T)

Calculate the required trash rack open area from the total outlet area based on outlet structure type for equations see Appendix G.

For this example, we used a single orifice outlet and thus will use equation G.10.

$A_T = (17.0\ \text{inches}^2)*77e^{-0.124*4.7} = 732$ square inches.

4.7.5.5 Basin Shape

- The flow path through the facility shall be made as long as possible to increase stormwater runoff residence time in the basin (UDFCD, 2005).
- A pilot channel can be constructed through the main part of the facility to convey low flows from the forebay to the bottom stage. A minimum 4 inch depth is required if concrete lined sides are used and 8 inches if buried riprap sides are used. At a minimum, provide conveyance capacity equal to twice the release capacity at the upstream forebay outlet (UDFCD, 2005).
- The top stage is defined as the basin bottom adjacent to the pilot channel on either side. It shall be at least 1 foot deep (Dts) with its bottom sloped 1 percent to 2 percent toward the pilot channel (Sts) (UDFCD, 2005).

- The bottom stage is defined as the deep portion of the EDDB around the outlet structure. This part of the basin shall be 1.25 to 3.0 feet deeper than the top stage. The bottom stage shall store 10 percent to 25 percent of the WQ_V that is stored below the top stage.

4.7.5.6 Forebay (Step 4)

Forebay volume (V_{FB})
The forebay volume should be greater than 10-percent of the WQ_V (Equation 4.22).

For this example, V_{FB} must be greater than 0.1*(2.38 acre feet) = 0.2 acre feet.

Forebay depth (Z_{FB})
The forebay depth should be at least 4 feet deep.

Minimum forebay surface area (A_{FB})
For this example using equation 4.23, A_{FB} = 0.2/4 = 0.1 acre.

4.7.5.7 Basin Side Slopes (Step 5)

The basin side slopes should be at least 4:1 (H:V) to ensure public safety and maintenance access. Stabilize side slopes with native vegetation.

4.7.5.8 Dam Embankment Side Slopes (Step 5)

- Dam embankment side slopes should not exceed 3:1 (H:V) for public safety.
- Embankment soils should be compacted to at least 95 percent of their maximum density according to ASTM D 698-70 (Modified Proctor).

4.7.5.9 Vegetation (Step 6)

To facilitate stabilization and biological filtration, the basin berms and side slopes should be planted with native vegetation.

To determine the appropriate native species, gather the following information about the EDDB site:

- Soil types (soil tests, soil maps in Appendix B)
- Annual precipitation with dates for wet/dry season (Maps in Appendix A)
- Ecoregion and corresponding vegetation (Map and table in Appendix C)
- Previous land use

Provide the soil type, precipitation, previous land use, and ecoregion information to a native vegetation expert for planting suggestions (vegetation types, seeding rates, establishment procedures, maintenance procedures). Use the “typical vegetation” listed in Appendix C as a guideline to check final list. Native vegetation contacts and links are listed in Appendix C.

4.7.5.10 Inlet Protection
Dissipate flow energy at basin's inflow point(s) to limit erosion and promote particle sedimentation.

4.7.5.11 Access
For maintenance purposes, there must be an all-weather access to the bottom and forebay (UDFCD, 2005). Slopes should not exceed 3:1.

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

- Drainage area map, including drainage area to detention basin.
- Existing and proposed contour map of site (1-foot contours recommended). Compaction requirements should be stated, if required. Additional spot elevations may be helpful.
- Geotechnical investigation of site (soil borings, water table location).
- Stormwater plan/profile for site.
- Detention basin plan view. Components clearly labeled with dimensions.
- Hydrologic calculations (refer to Design Example).
- Detail of control structure (orifice/weir) with dimensions for construction. Include appropriate design calculations (refer to Design Example).
- Velocity downstream of control structure. Appropriate armoring should be specified.
- Vegetation plan with schedule for installation and initial maintenance. Appropriate erosion control measures should be included.
- An as-built survey of the detention basin is recommended to confirm actual construction adheres to approved construction plans.
- Long-term inspection/maintenance plan.
- Other requirements as required by local jurisdiction for flood storage beyond water quality event.

4.7.7 References
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Section 4 Structural BMPs

4.8 Extended Wet Detention

Extended wet detention basins (EWDBs) are designed to reduce pollutants from stormwater runoff via removal mechanisms in the permanent pool and decrease peak runoff rates with an extended storage capacity (UDFCD, 2005, IDEQ , 2005). The primary removal mechanism is settling as stormwater runoff resides in this pool, but pollutant uptake, particularly of nutrients, also occurs to some degree through biological and chemical activity in the pond (CASQA, 2003). In addition, a temporary detention volume is provided above this permanent pool to capture the water quality volume (WQv) and enhance sedimentation (UDFCD, 2005). EWDBs differ from traditional wet ponds in that the WQv is split between the permanent pool and the extended detention volume that is provided above the pool (IAState, 2008). The influent water mixes with the permanent pool water as it rises above the permanent pool level. The temporary detention volume above the permanent pool provides additional time for sedimentation. The surcharge captured volume above the permanent pool is then released over 40 hours (UDFCD, 2005). EWDBs have similar levels of pollutant removal as a traditional wet detention basin, but require less land area (Iowa Stormwater Manual, 2008). EWDBs are similar in function to constructed wetlands but differ primarily in that they have a greater average depth (CASQA, 2003). EWDBs can be very effective in removing pollutants, and, under the proper conditions, can satisfy multiple objectives, including water quality improvement, flooding and erosion protection, creation of wildlife and aquatic habitats, and recreational and aesthetic provision (UDFCD, 2005). Figure 4-24 is a photograph of an extended detention basin.

Sections from this manual that may need to be referenced for additional information are: Section 2; Section 4.1; Section 5

Location characteristics (Slope, Soil Type)	Slope: < 10%[1] Soil Type: All
Contributing drainage area	Site specific; requires water budget calculations – rule of thumb is at least 10 acres per acre of permanent pool surface area[1]
Design size	Tributary area from 2 to 1,000 acres
Detention time for WQv treatment	40 hrs
Pollutant removal efficiencies[1]	80% TSS, 65% TP 35-65% TN[2]
Potential for education and outreach	**High** Lot level private gardens can be part of your NPDES outreach activities
Potential for use with other BMPs	**Moderate** Pretreatment required for TSS removal
Implementation Category	Short Term: **Easy** Long Term: **Easy** (See Section 5.4.1)
Maintenance	**High** Sediment/debris removal, vegetation upkeep (See Section 5.4.1); Permanent pool depth inspection/maintenance

[1]Mid-America Regional Council, 2008, [2]IAState, 2008

Figure 4-24 Extended Wet Detention Basin with Landscaping and Recreational Components

Source: MARC, 2008

4.8.1 General Application

EWDBs can be used to improve stormwater runoff quality and reduce peak stormwater runoff rates and peak stages from roads, parking lots, residential neighborhoods, commercial areas, and industrial sites. An EWDB can also be designed to provide flood control benefits. An EWDB is more applicable to treat larger tributary areas than other BMPs, and can be utilized as a second BMP in a treatment train. An EWDB may be used for a smaller site if the drainage area is sufficient for sustaining a permanent pool. An EWDB works well in conjunction with other BMPs such as upstream onsite source controls and downstream filter basins or wetland channels (UDFCD, 2005). See Section 2 for applicability of an EWDB in a treatment train.

4.8.2 Advantages and Disadvantages

Advantages	Disadvantages
Moderate pollutant removal[1]	Low volume reduction (+/- 10%)
80% TSS[2], 65% TP[2] 35-65% TN[3]	
Peak flow mitigation[1]	Potential outflow impacts on downstream quality
Potential for use as a flood control mechanism[1]	Increases in surface water temperature
Opportunity for recreational and open space facilities[1]	May attract unwanted wildlife such as geese
Widespread application can reduce channel degradation caused by high sediment and runoff loads[1]	Can be a source of odor if not properly maintained, which includes maintaining the permanent pool depth
Wildlife habitat	

[1]MARC, 2008, [2]IDEQ, 2005, [3]Iowa, 2008

4.8.3 Design Requirements and Considerations

Extended dry detention design shall be by a registered Professional Engineer in the State of Kansas. All design calculations and construction drawings shall be sealed and signed.

4.8.3.1 Site Requirements

EWDBs are very applicable for the management of runoff from large drainage areas. EWDB facilities should be designed as off-line entities outside of stream corridors and buffer areas (MARC, 2008). Due to their ability to serve larger drainage areas, EWDBs can be designed for recreational and wildlife preservation purposes in mind. Guidelines for determining the appropriate location of an EWDB are outlined below.

- EWDBs shall have between 2 and 1,000 acres tributary to the facility (KC Metro APWA, 2006).

- Do not locate EWDBs on fill sites or on or near steep slopes. Depending on soils, bottom modifications can include compaction, incorporating clay into the soil, or an artificial liner (Nashville, 2006).

- A maintenance ramp and perimeter access should be included in the design to facilitate access to the basin for maintenance activities (CASQA, 2003).

- The maximum water surface that the facility is designed for shall be a minimum distance of 20 feet from property lines and building structures or per agency specification. A greater distance may be necessary when the detention facility might compromise foundations or slope stability (KC Metro APWA, 2006).

- Public safety shall be considered in EWDB design. Fences and landscaping can be used to impede access to the facility. The facility shall be contoured so as to eliminate any drop-offs or other hazards.

- When possible, terraces or benches shall be used to transition into the permanent pool. In some cases there is not sufficient room for grading of this type and the pond may require a perimeter fence (Nashville, 2006).

4.8.3.2 Basin Dimensions

Basin geometry is a function of the WQv calculated and other site characteristics. To determine the required storage volume of an EWDB, calculate the WQv based on the drainage area and add 20 percent to the result. This will provide the basin size necessary to treat this volume over 40 hours. The additional volume will account for silt and sediment deposition in the EWDB. This volume allows a flow through velocity that is less than the settling velocity of pollutants (Muthukrishnan et al. 2006). General guidelines are outlined below.

- Side slopes above the littoral bench (see Figure 4-25) shall be 4:1 (H:V) or flatter unless retaining walls are used. Side slopes below the littoral bench can be as steep as 3:1 to maximize permanent pool volumes where needed (Nashville, 2006).

- To maintain a permanent pool, the tributary area to the EWDB should be at least 5.5 acres for each acre-foot of permanent pool volume and at least 10.3 acres for each acre of permanent pool surface area. Table 4-8 presents threshold tributary areas for different Rational C values. These are general guidelines. Water budget calculations are recommended for most designs.

- Design of the permanent pool volume should allow for 14 days hydraulic residence time to allow for particulate settling and nutrient uptake. This is accomplished by sizing the pool using regional precipitation data and characteristics of the tributary area to the EWDB. These considerations are illustrated in the design example at the end of this section.

- The EWDB shall be designed to detain the WQv above the permanent pool and shall release the WQ_v over a 40 hour period. Additional flood control volume can also be provided above the permanent pool (UDFCD, 2005). Refer to local stormwater detention for design specifications if flood control is to be incorporated into the design of the EWDB.

- Dams that are greater than 10 feet in height but do not fall into state or federal requirement categories shall be designed in accordance with the latest edition of SCS Technical Release No. 60, *Earth Dams and Reservoirs*, as Class C structures (KC Metro APWA, 2006).

Table 4-8 Threshold Tributary Areas to EWDB (MARC, 2008)

	Rational Runoff Coefficient							
	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1
Minimum Tributary Area per Acre-Foot of Volume	18.4	13.8	11	9.2	7.9	6.9	6.1	5.5
Minimum Tributary Area per Acre of Surface Area	34.2	25.7	20.5	17.1	14.7	12.8	11.4	10.3

Reproduced from MARC, 2008

4.8.3.3 Basin Configuration

The inlet of the basin shall be designed to minimize runoff velocities to prevent sediment re-suspension. Runoff will flow through the inlet and into a forebay. The forebay exists to collect sedimentation prior to runoff entry into the main basin, therefore reducing overall maintenance. It is more cost-effective to remove sediments and trash from a small, easily accessed forebay than the large basin. The permanent pool depth should be designed to limit sedimentation and vegetation encroachment into the open water surface. The outlet should be designed to release the captured runoff over the 40 hour detention time without erosion. It is recommended to install a trash rack at the outlet to aid in maintenance. Figure 4-25 offers guidance for basin configurations.

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, 2008).

Forebay

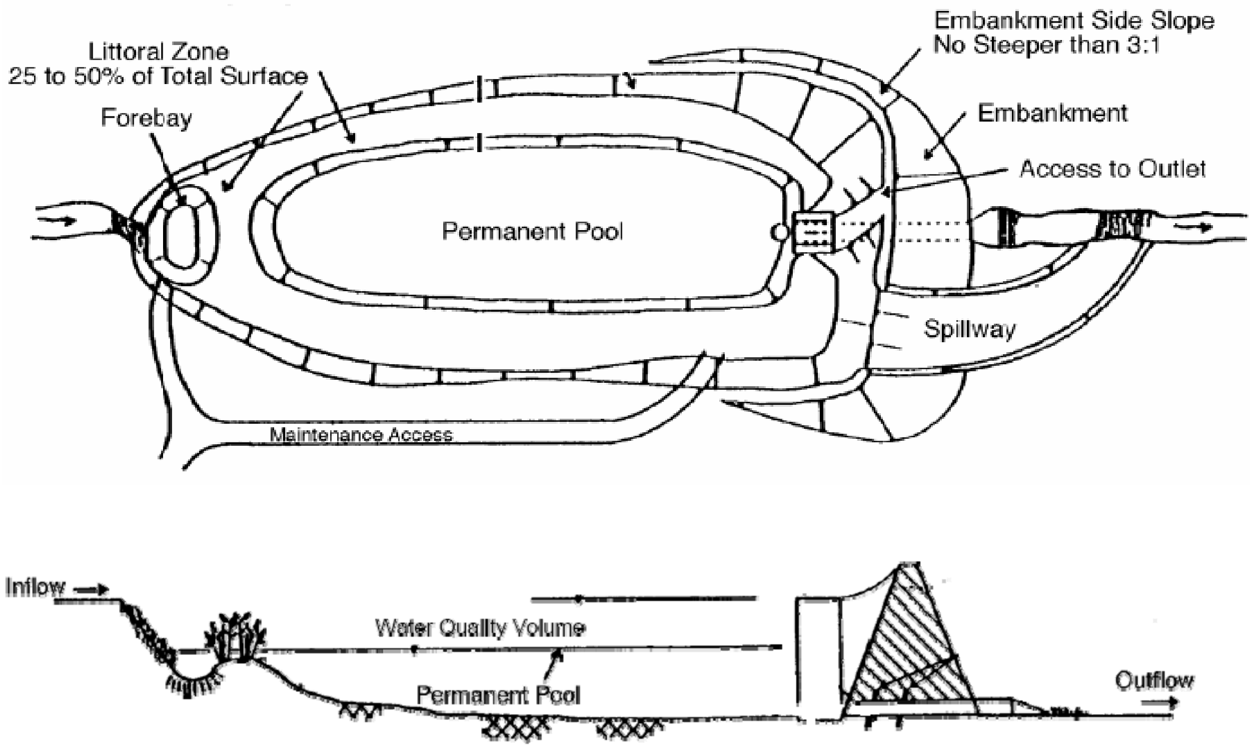
- The forebay shall be a 4 to 6 feet deep cell delineated by a barrier and shall be sized to contain at least 10 percent of the WQv.

- The minimum length to width ratio shall be a minimum of 2:1 (3:1 recommended) to prevent short-circuiting (Muthukrishnan et al. 2006).

Permanent Pool

- The permanent pool shall include a littoral bench, or shelf, around the pool's perimeter which serves as both a safety feature and a planting surface for wetland vegetation.

Figure 4-25 Cross Sectional View of Extended Wet Detention Basin



Source: MARC, 2008

- The littoral bench shall extend inward at least 10 feet from the perimeter of the permanent pool and shall be between 6 inches to 12 inches below the permanent pool surface (CASQA, 2003, UDFCD, 2005).

- The slope of the littoral bench shall not exceed 6:1. The bench shall be planted with native wetland vegetation to promote biological uptake of nutrients and dissolved pollutants and reduce the formation of algal mats. To maximize biological uptake but prevent plants from encroaching on the open water surface, the vegetated littoral bench shall comprise 25 percent to 50 percent of the permanent pool surface area (Nashville, 2006).

- Permanent pool depths optimally range from 4 feet to 6 feet, and shall be no greater than 12 feet (CASQA, 2003). The minimum depth of 4 feet shall be provided in addition to an estimated depth of sediment accumulation from 5 years of EWDB service. Permanent pool depth should be verified annually. If EWDB is used as a siltation basin prior to a BMP, bottom elevation within the EWDB may need to be modified to attain the required permanent pool depth. This can be verified by requiring an as-built survey of the basin post construction.

- If the facility is to contain fish, at least one-quarter of the area of the permanent pool must have a minimum depth of 10 feet plus a sedimentation allowance (KC Metro APWA, 2006).

- In very dry climates, an impermeable liner may be required to maintain an adequate permanent pool level (CASQA, 2003).

Outlet

- The outlet shall be designed to discharge the WQv over a period of 40 hours (UDFCD, 2005).

- Locate basin outlet as far away from basin inlet(s) as possible to prevent water from short-circuiting the facility. The flow path(s) should have a minimum length of two times the facility width, as measured across the center of the facility in the smallest dimension at the permanent pool elevation (Nashville, 2006).

- No single outlet orifice shall be less than 4 inches in diameter (smaller orifices are more susceptible to clogging).

- If the calculated orifice diameter necessary to achieve a 40-hour drawdown is less than 4 inches, a perforated riser, orifice plate, or v-notch weir shall be used instead of a single orifice outlet. Keep perforations larger than 1 inch when using orifice plates or perforated risers. Smaller orifice sizes may be used if the weir plate is placed in a riser manhole in a sump-like condition or protected by a well screen.

- A reverse-slope pipe can be used to prevent outlet clogging from debris. A reverse-slope pipe draws from below the permanent pool extending in a reverse angle up to the riser and establishes the water elevation of the permanent pool. Because these outlets draw water from below the level of the permanent pool, they are less likely to be clogged by floating debris (CASQA, 2003).

- The facility shall have a separate drain pipe with a manual valve that can completely drain the pond for maintenance purposes. To allow for possible sediment accumulation, the submerged end of the pipe shall be protected, and the drain pipe shall be sized to drain the pond within 24 hours (CASQA, 2003).

4.8.3.4 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 initiate slower velocities. Vegetation surrounding the outlet may serve as a buffer for the BMP to reduce runoff impacts on downstream areas. Information about the establishment and maintenance of native vegetation is outlined in section 5 of this manual.

4.8.3.5 Potential Treatment Train Options

These basins work well in conjunction with BMPs that are designed primarily for sediment reduction. EWDBs are also effective when combined with BMP's that effectively reduce runoff volumes. EWDBs can be used as a flood mitigation facility. EWDBs can also be used for recreation, open space, or wildlife habitat if wetlands or shallow pools are incorporated into the design (UDFDC, 2008).

4.8.4 Design Calculations

A short summary of the design calculations is presented below. A detailed design example is outlined in 4.8.6.

- **Step 1** Determine WQ_V based on drainage area and regional precipitation information according to Equations 4.1 and 4.2.

Equation 4.1 $$R_v = 0.05 + 0.009(\mathrm{I})$$

Equation 4.2 $$WQ_V = \frac{P_{WQ} \times R_V \times A_T}{12}$$

Where:
To obtain basin design volume, V_{DESIGN}, multiply WQ_V by 1.2 to account for sedimentation (approximately 20-percent of the WQ_V)

- **Step 2** Size the permanent pool volume based on the 14 day retention time requirement and desired sedimentation rates. First calculate the rational runoff coefficient according to Equation 4.5. Compare the volumes calculated in Equation 4.24 and Equation 4.26. The larger of the two is the permanent pool volume, which is then multiplied by 1.2 to account for sedimentation.

Equation 4.5 Rational Runoff Coefficient

$$C = 0.3 + 0.6 * \left(\frac{I}{100}\right)$$

Where:
C = Rational Runoff Coefficient
I = Percent impervious area (%)

Equation 4.24 *14 day permanent pool volume (acre feet)*

$$V_{P1}(ac-ft) = \frac{C * A_T * R_{14}}{12}$$

Where:
V_{P1} = 14 day permanent pool volume (acre feet)
C = Rational Runoff Coefficient
A_T = Tributary area (acre)
R_{14} = 14-day wet season rainfall depth (inch) (Table 4-9)

Equation 4.25 Impervious tributary area (acre)

$$A_{T,I} = A_T * \frac{I}{100}$$

Where:

$A_{T,I}$	=	Impervious tributary area (acre)
A_T	=	Tributary area (acre)
I	=	Percent impervious area (%)

Equation 4.26 Sedimentation permanent pool volume

$$V_{P2} = \frac{V_{B/R} * S_d * A_{T,I}}{12}$$

Where:

V_{P2}	=	Sedimentation permanent pool volume (acre feet)
$V_{B/R}$	=	Runoff volume ratio from Figure 4-26.
S_d	=	Mean storm depth (inch) (Table 4-9)
$A_{T,I}$	=	Impervious tributary area (acre)

Equation 4.27 Surface area of permanent pool

$$A_P = \frac{V_P}{Z_d}$$

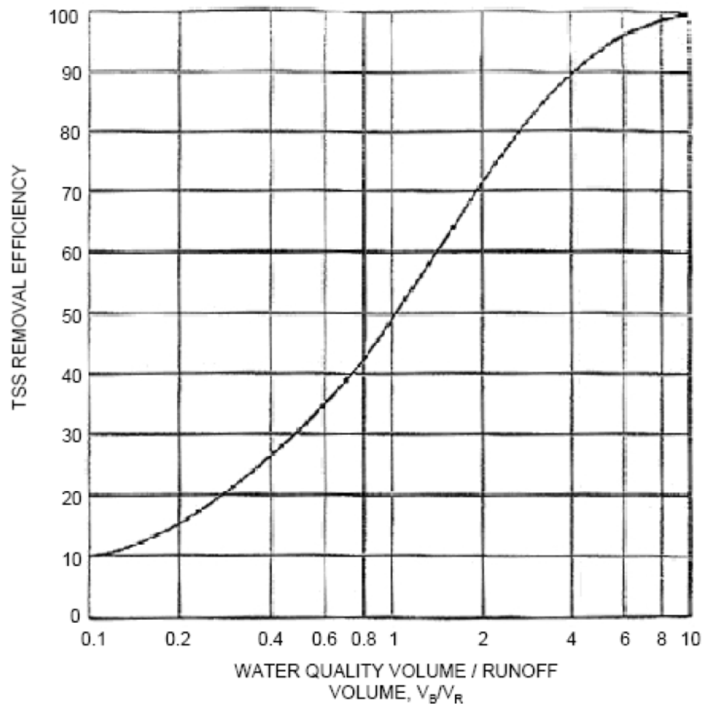
Where:

A_P	=	Surface area of permanent pool (acre)
V_P	=	Permanent pool volume that accounts for 20 percent sedimentation (acre feet)
Z_d	=	Average permanent pool depth (feet)

Table 4-9 Fourteen Day Wet Season Rainfall Depth (R_{14}) and Mean Storm Depth (S_d) for Phase II Kansas Cities

City	County	KS Region	S_d	R_{14}
Dodge City	Ford	West	0.29	0.80
Garden City	Finney	West	0.34	0.74
Hays	Ellis	West	0.36	1.34
Great Bend	Barton	Central	0.40	1.38
Manhattan	Riley	Central	0.44	1.78
Newton	Harvey	Central	0.46	1.86
Salina	Saline	Central	0.40	1.23
Arkansas City	Cowley	Central	0.44	1.61
Hutchinson	Reno	Central	0.46	1.86
Winfield	Cowley	Central	0.44	1.61
Coffeyville	Montgomery	East	0.57	1.97
Lawrence	Douglas	East	0.46	1.98
Ottawa	Franklin	East	0.46	1.99
Emporia	Lyon	East	0.46	1.99

Figure 4-26 Relationship Between TSS Removal and the Ratio of WQ_V to Runoff Volume ($V_{B/R}$)



Source: FHWA, 1989

- **Step 3** Determine the outlet type (single orifice, perforated riser, or v-notch weir), outlet loads, and required outlet dimensions. If the diameter calculated for a single orifice is less than 4 inches, use a perforated riser or v-notch weir outlet to prevent clogging. Use equation 4.16 to determine the outflow rate. Equations for each outlet type are presented in Appendix G.

- **Step 4** Size trash racks according to outlet type and size. These calculations will vary depending on outlet structure type (See Appendix G).

- **Step 5** Design the forebay based on WQ_V and minimum depth requirements. The forebay volume should be greater than 10-percent of the WQ_V (Equation 4.22) and the forebay surface area is calculated using Equation 4.23.

Equation 4.22 *Forebay Volume*

$$V_{FB} > 0.1\, WQ_V$$

Where:
V_{FB} = Forebay volume (acre feet)
WQ_V = Water quality volume (acre feet)

Equation 4.23 *Forebay Surface Area*

$$A_{FB} = \frac{V_{FB}}{Z_{FB}}$$

Where:

A_{FB} = Forebay surface area (acre)
V_{FB} = Forebay volume (acre feet)
Z_{FB} = Forebay depth (feet)

- **Step 6** Calculate the littoral bench dimensions according to Equations 4.34 and 4.35.

Equation 4.34 *Littoral Bench Surface Area*

$$0.25\,A_P \le A_{LB,MIN/MAX} \le 0.5\,A_P$$

Where:

A_P = Permanent pool surface area (acre)
A_{LB} = Littoral bench surface area (acre)

Equation 4.35 Littoral bench width

$$W_{LB,\,MIN/\,MAX} = \frac{1}{2}\sqrt{\frac{4}{\Pi} A_{LB,\,MIN/\,MAX}(43560 ft^2 / ac)}$$

Where:

W_{LB} = Littoral bench width (feet)
A_{LB} = Littoral bench surface area (acre)

- **Step 7** Determine basin shape, basin side slopes, and dam embankment side slopes.

- **Step 8** Install appropriate vegetation using methodology provided by local native vegetation experts.

4.8.5 Design Example

A commercial shopping area is being built on previously undeveloped land in Lawrence. The designer would like to build an extended wet detention basin to treat the runoff from a tributary area of 50 acres, including the facility roofs and parking lots (85-percent impervious). The majority of soil in the development is type D and has high-clay content. The land slope is less than 5-percent across the development. Example calculation spreadsheet can be found in Appendix G.

4.8.5.1 Basin Water Quality Volume (Step 1)

Determine the tributary area to the EWDB (A_T)

The tributary area, A_T, is 50 acres. Due to the fact that A_T = 50 acres and the percent imperviousness is already known, we shall utilize the Short-Cut Hydrology Method.

Calculate the Rv based on equation 4.1

The tributary area is 85-percent impervious. Thus, $R_v = 0.05 + 0.009(85) = 0.815$

Calculate the WQv based on equation 4.2
For Lawrence, the water quality event is 1.18 inches. The WQ_V is
WQ_V = (1.18 in)*(0.815)*(50 ac.)/12 in = 4.0 ac-ft

4.8.5.2 Permanent Pool Volume (Step 2)

Fourteen day Volume (V_{P1})

This method calculates the volume required in the permanent pool to detain water for the minimum 14 days. This allows time for algae uptake of phosphorus and sedimentation where phosphorus may be concentrated.

Enter the 14-day wet season rainfall R_{14} from Table 4-9 or Appendix A.

For Lawrence, the R_{14} is 1.98 in.

Determine the Rational Runoff Coefficient (C) for the tributary area based on equation 4.5.

For this site, C = 0.3 + 0.6(85/100) = 0.81

Calculate the permanent pool volume (V_{P1}) from equation 4.24.

*For this example, V_{P1} = (0.81*50*1.98)/12 = 6.7 ac-ft.*

Sedimentation Volume (V_{P2})

This method calculates the volume required to settle out the suspended solids in the permanent pool.

Select the WQ_V to runoff volume ratio ($V_{B/R}$) from Figure 4-26 based on the desired TSS removal efficiency. This ratio must be greater than 4 (MARC, 2008).

For this example, choose a $V_{B/R}$ = 4 to meet minimum requirements.

Determine the mean storm depth (S_d) for your region from Table 4-9 or Appendix A.

For Lawrence, S_d = 0.46 in.

Calculate the total impervious tributary area ($A_{T,I}$) in acres based on equation 4.25.

*For Lawrence, $A_{T,I}$ = 50*0.85 = 42.5 ac.*

Calculate the permanent pool volume (V_{P2}) using equation 4.26.

*For this example, V_{P2} = (4*0.46*42.5)/12 = 6.5 ac-ft*

CDM

Permanent Pool Volume (V_{P2})

Choose the volume that is largest between V_{P1} and V_{P2}. This value is the design volume (V_P) for the permanent pool. Add 20-percent to account for sedimentation (multiply V_P by 1.2).

In this example, $V_p = 1.2*6.7 = 8.0$ acre feet.

Set the desired average permanent pool depth (Z_d) which should be between 4 and 6 feet for non-fish pond. The estimated depth of sediment accumulation over a 5 year period must also be accounted for when specifying total depth during design.

For this pond, the depth will be set at the minimum 4 feet due to the fact that the tributary area is quite small and the pool should maintain a shallow depth to initiate sedimentation and filtration processes.

Calculate the required permanent pool surface area (A_P) using equation 4.27.

The A_P = (8.0 ac-ft)/(4 ft) = 2.0 ac

4.8.5.3 Outlet (Step 3)

There are three possible outlet types to use with detention basins. They include single orifice, perforated riser or plate, and V-notch weir. For this example, we will use a perforated riser. If the orifice diameter required to drain the excess to the permanent pool is less than 4 inches, a perforated riser or v-notch weir should be used (MARC, 2008). Refer to Appendix G for equations associated with these calculations.

Water quality depth (Z_{WQ})

Set the depth above the WQ_V outlet (Z_{WQ}) based on facility dimensions for surface area and desired depth.

$Z_{WQ} = 3$ ft

Maximum outlet area (A_O) per row of perforations

Calculate the recommended maximum outlet area per row of perforations (A_O) based on the WQ_V and the depth at the basin outlet. A Manning's value (n) of 0.013 was used for this calculation; this will vary by agency. Use equation G.2.

*For this example, A_O = (4.0 ac-ft)/(0.013*3^2 + 0.22*3-0.1) = 5.9 in^2*

Outlet pipe diameter (D_1)

Assume a single column of perforations and calculate the diameter of a single circular perforation (D_1) for each row based on A_O. Use equation G.3.

$D_1 = ((4*5.9\ in^s)/\pi))^{-1/2} = 2.8$ in

Column Number (n_C)
The optimal number of columns of perforations is 1. However, if $D_1 > 2$ inches, then design for more than one column. Keep this number as low as possible.

For this example, D_1 is greater than 2 inches, thus we will design for two columns of perforations.

Perforation diameter (D_{perf})
The circular perforation diameter is found using equation G.4.

For this example, $D_{perf} = ((4*5.9)/(\pi*2))^{1/2} = 1.9$ inches.

Horizontal column spacing (S_c)
When $n_C > 1$, the center to center column spacing of perforations, S_c, is 4 inches.

Perforation rows (n_F)
The number of rows is determined using equation G.5 assuming 4 inch center to center vertical spacing between perforations.

In this example, $n_f = (3*12/4) = 9$

4.8.5.4 Trash Racks (Step 4)
A trash rack protects outlet structures from damage resulting from trash and debris (4-27 and 4-28 the end of this section). This calculation is based on the outlet type used. For equations see Appendix G.

Outlet Area (A_{OT})
Calculate the outlet area based on the outlet area per perforation row (A_O) and the number of rows (n_F) and number of columns (n_C). Use equation G.11.

For this example, $A_{OT} = (5.9\ in^2)*(9\ rows)*(2\ columns) = 106\ in^2$.

Open Area (A_T)
Calculate the required trash rack open area from the A_{OT} depending on outlet structure type.

For this example, we used a perforated riser and thus will use equation G.12.

$A_T = (106\ in^2/2)*77*e^{(-0.124*2.8)} = 2911$ square inches

4.8.5.5 Forebay (Step 5)
Forebay volume (V_{FB})
The forebay volume should be greater than 10-percent of the WQ_V. Use Equation 4.22.

For this example, V_{FB} must be greater than 0.1(4.0 ac-ft) = 0.4 ac-ft.*

Forebay depth (Z_{FB})
The forebay depth should be at least 4 feet deep.

Minimum forebay surface area (A_{FB})
Use Equation 4.23 to calculate the minimum surface area of the forebay.

For this example, $A_{FB} = 0.40/4 = 0.10$ ac.

4.8.5.6 Littoral Bench (Step 6)

Littoral bench surface area (A_{LB})
The littoral bench surface area should be between 25-50-percent of the total permanent pool surface area (A_P from Step 2) using *equation 4.34.*

For this example, $A_{LB,MIN} = 0.25(1.98) = 0.5$ *acres*

$A_{LB,MAX} = 0.5(1.98) = 1.0$ *ac.*

Littoral bench width (W_{LB})
The minimum and maximum widths can be estimated using equation 4.35.

$W_{LB,MIN} = (1/2)*((4/\pi)*0.5*43560)^{1/2} = 83.4$ *ft* $W_{LB,MAX} = (1/2)*((4/\pi)*1*43560)^{1/2} = 118$ *ft.*

The bench width, W_{LB} should be within this range of values. For this example, we will choose the average value of 100 feet.

Bench depth (Z_{LB})
The littoral bench depth should be between 6 to 12 inches below the permanent pool surface.

4.8.5.7 Basin Side Slopes (Step 7)

The basin side slopes should be at least 3:1 (H:V) to ensure public safety and maintenance access. Stabilize side slopes with native vegetation.

4.8.5.8 Dam Embankment Side Slopes (Step 7)

- Dam embankment side slopes should be at least 3:1 (H:V) for public safety.
- Embankment soils should be compacted to at least 95 percent of their maximum density according to ASTM D 698-70 (Modified Proctor).
- Embankment slopes should be planted with turf forming grasses.

4.8.5.9 Vegetation (Step 8)

To facilitate stabilization and biological filtration, the basin berms, side slopes, and the littoral bench should be planted with native vegetation.

To determine the appropriate native species, gather the following information about the EWDB site:

- Soil types (soil tests, soil maps in Appendix B)
- Annual precipitation with dates for wet/dry season (Maps in Appendix A)
- Ecoregion and corresponding vegetation (Map and table in Appendix C)
- Previous land use

Provide the soil type, precipitation, previous land use, and ecoregion information to a native vegetation expert for planting suggestions (vegetation types, seeding rates, establishment procedures, maintenance procedures). Use the "typical vegetation" listed in Appendix C as a guideline to check final list. Native vegetation contacts and links are listed in Appendix C.

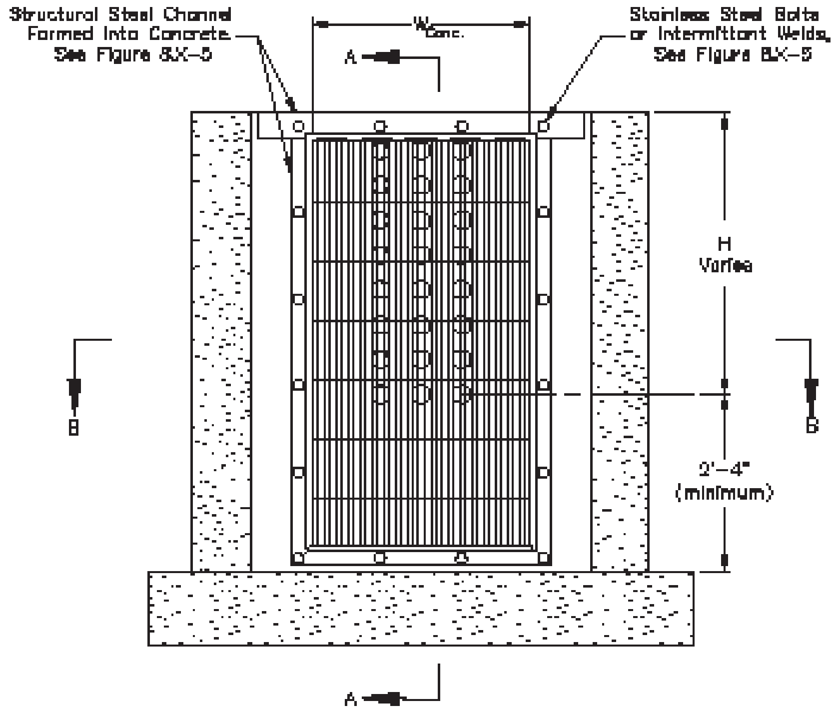
4.8.5.10 Inlet Protection

Dissipate flow energy at basin's inflow point(s) to limit erosion and promote particle sedimentation.

4.8.5.11 Access

For maintenance purposes, there must be an all-weather access to the bottom, forebay, and littoral bench (UDFCD, 2005). Slopes should not exceed 3:1.

Figure 4-27 WQv Outlet Trash Rack Design (UDFCD, 2005)



Elevation

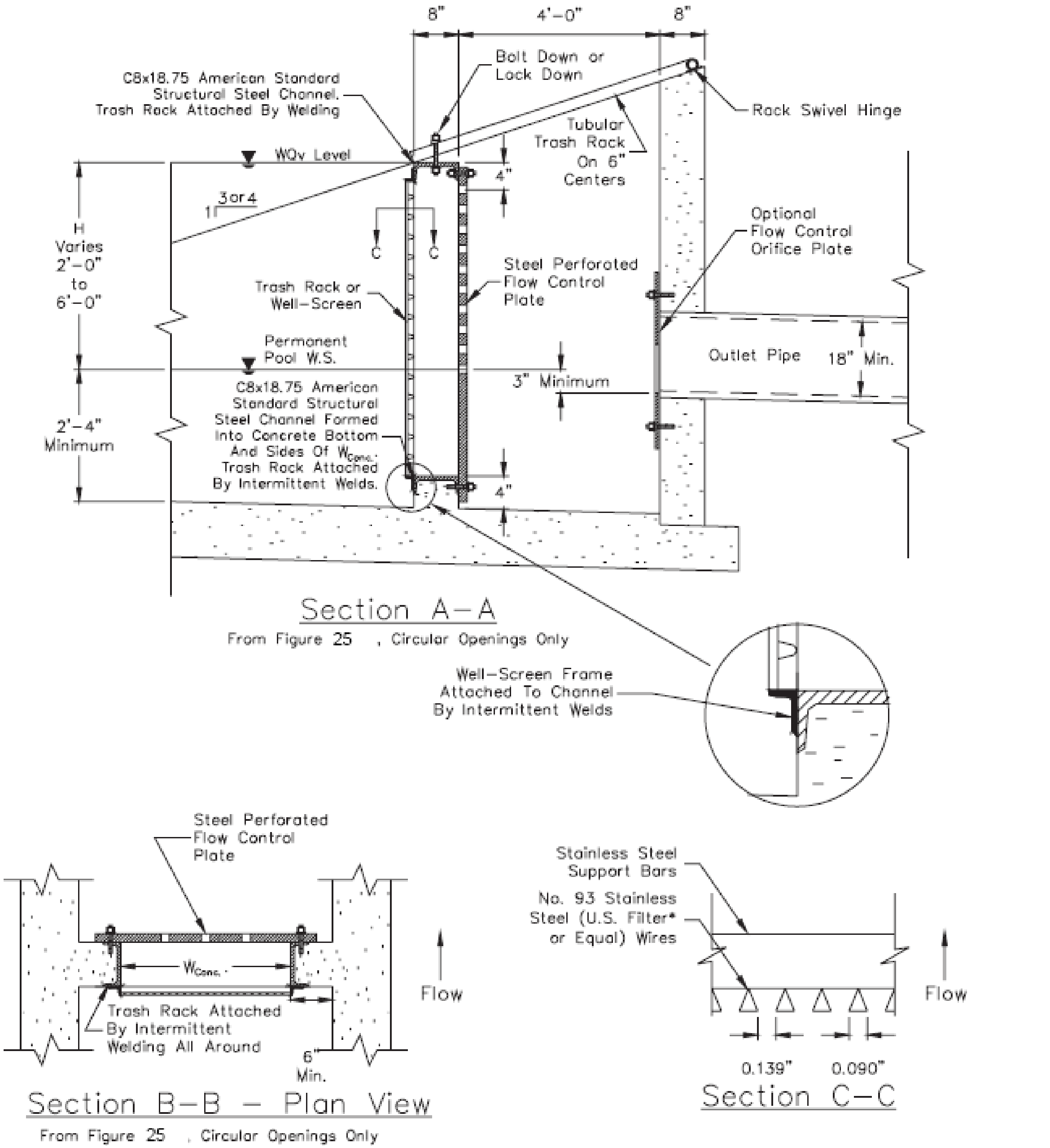
WQv Trash Racks:

1. Well–screen trash racks shall be stainless steel and shall be attached by intermittent welds along the edge of the mounting frame.
2. Bar grate trash racks shall be aluminum and shall be bolted using stainless steel hardware.
3. Trash Rack widths are for specified trash rack material. Finer well–screen or mesh size than specified is acceptable, however, trash rack dimensions need to be adjusted for materials having a different open area/gross area ratio (R value)
4. Structural design of trash rack shall be based on full hydrostatic head with zero head downstream of the rack.

Overflow Trash Racks:

1. All trash racks shall be mounted using stainless steel hardware and provided with hinged and lockable or boltable access panels.
2. Trash racks shall be stainless steel, aluminum, or steel. Steel trash racks shall be hot dip galvanized and may be hot powder painted after galvanizing.
3. Trash Racks shall be designed such that the diagonal dimension of each opening is smaller than the diameter of the outlet pipe.
4. Structural design of trash rack shall be based on full hydrostatic head with zero head downstream of the rack.

Figure 4-28 Alternative WQv Outlet Trash Rack Design (UDFCD, 2005)



Limits for this Standardized Design:

1. All outlet plate openings are circular.
2. Maximum diameter of opening = 2 inches.

*U.S. Filter, St. Paul, Minnesota, USA

R Value = (net open area)/(gross rack area) = 0.60

4.8.6 Submittal Requirements

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

- Drainage area map, including drainage area to detention basin.
- Existing and proposed contour map of site (1-foot contours recommended). Compaction requirements should be stated, if required. Additional spot elevations may be helpful.
- Geotechnical investigation of site (soil borings, water table location).
- Stormwater plan/profile for site.
- Detention basin plan view. Components clearly labeled with dimensions.
- Hydrologic calculations (refer to Design Example).
- Detail of control structure (orifice/weir) with dimensions for construction. Include appropriate design calculations (refer to Design Example).
- Velocity downstream of control structure. Appropriate armoring should be specified.
- Vegetation plan with schedule for installation and initial maintenance. Appropriate erosion control measures should be included.
- An as-built survey of the detention basin is recommended to confirm actual construction adheres to approved construction plans. An as-built survey should be required if the detention basin area was also used as a sedimentation basin during the project.
- Long-term inspection/maintenance plan. Permanent pool depth should be inspected annually by survey, with maintenance performed as needed.

4.8.7 References

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