APPENDIX B: **Additional BMP-Specific Design Guidance**

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APPENDIX B: **Additional Bioretention Design Guidance**

Bioretention

VOLUME MANAGEMENT

The design of a bioretention area can be broken down to a nine-step process. Table B-1 summarizes the steps, which are described in greater detail in this chapter.

Figure B-1: *Rendering showing how roadside bioretention can be retrofit into the right-of-way to intercept street runoff through curb cuts, Broadway Street, Witte Museum, San Antonio, Texas. Source: Bender Wells Clark Design*

TABLE B-1. BIORETENTION ITERATIVE DESIGN STEP PROCESS

Design step		Design component/ consideration	General guidance
1	**Determine BMP Treatment Volume (166)**	Use Appendix J	
2	**BMP Siting (166)**	Based on available space and maintenance access, incorporate into parking lot islands, medians, and perimeter; install along the roadway right-of-way; incorporate as landscaped areas throughout the property; or dedicate space for larger, centralized bioretention areas	
3	**Determine BMP Function and Configuration (167)**	Impermeable liner	If non-infiltrating (per geotechnical investigation), use an impermeable clay layer, geomembrane liner, and concrete (as described in Appendix A, Common Design Elements)
		Lateral hydraulic restriction barriers	Use concrete or geomembrane to restrict lateral flows to adjacent subgrades, foundations, or utilities.
		Underdrain (required if subsoil infiltration rate is less than 0.5 in/hour)	Schedule 40 PVC pipe with perforations (slots or holes) every 6 inches. 4-inch diameter lateral pipes should join a 6-inch collector pipe, which conveys drainage to the downstream storm network. Provide cleanout ports/observation wells for each underdrain pipe at spacing consistent with local regulations. See Appendix A, Common Design Elements.
		Internal water storage (IWS)	If using underdrain and infiltration, elevate the outlet to create a sump for additional moisture retention to promote plant survival and enhanced treatment. Top of IWS should be greater than 18 inches below soil surface.
		No underdrain	If design is fully infiltrating, ensure that subgrade compaction is minimized.

TABLE B-5. BIOSWALE ITERATIVE DESIGN STEP PROCESS (CONT.)

4	**Size the System (168)**	Temporary ponding depth	6-18 inches (6-12 inches near schools or in residential areas); average ponding depth of 9 inches is recommended
		Soil media depth	2-4 feet (deeper for better pollutant removal, hydrologic benefits, and deeper rooting depths)
		Surface area	Find surface area required to store treatment volume within temporary ponding depth, soil media depth, and gravel drainage layer depth (media porosity ≈ 0.35 and gravel porosity ≈ 0.4)
5	**Specify Soil Media (170)**	Composition and texture	85-88% sand, 8-12% fines, 2-5% plant-derived organic matter (animal wastes or by-products should not be applied)
		Permeability	1-6 in/hour infiltration rate (1-2 in/hour recommended)
		Chemical composition	Total phosphorus < 15 ppm, pH 6-8, CEC > 5 meq/100 g soil
		Drainage layer	Separate soil media from underdrain layer with 2 to 4 inches of washed sand, followed by 2 inches of choking stone (ASTM No. 8) over a 1-5-foot envelope of ASTM No. 57 stone.
6	**Design Inlet and Pretreatment (171)**	Inlet	Provide stabilized inlets (see Appendix A, Common Design Elements).
		Pretreatment	Install rock armored forebay (concentrated flow), gravel fringe and vegetated filter strip (sheet flow), or vegetated swale.
7	**Select and Design Overflow/Bypass Method (172)**	Outlet configuration	Online: All runoff is routed through system—install an elevated overflow structure or weir at the elevation of maximum ponding. Offline: Only treated volume is diverted to system—install a diversion structure or allow bypass of high flows (see Diversion Structures for details).
		Peak flow mitigation	Provide additional detention storage and size an appropriate non-clogging orifice or weir to dewater detention volume.
8	**Select Mulch and Vegetation (175)**	Mulch	Dimensional chipped hardwood or triple shredded, well-aged hard-wood mulch 3 inches deep.
		Vegetation	See Plant List (Appendix E)
9	**Design for Multi-Use Benefits (Appendix C)**	Include features to enhance habitat, aesthetics, public education, and shade.	

Step 1. Determine BMP Treatment Volume

The bioretention area must be sized to fully capture the desired or required design storm volume and filter it through the soil media. Surface storage (in the ponding area) and soil pore space (in the plant rooting zone and the underlying media and gravel drainage layers) provide capacity for the design storm volume retention. Appendix J outlines methods for determining design runoff depths associated with a range of annual treatment efficiencies. Once the design runoff depth is determined (on the basis of the desired level of treatment), a runoff volume can be determined for the contributing watershed using this depth and the methods outlined in Appendix J, San Antonio Unified Development Code, or San Antonio River Basin Regional Modeling Standards for Hydrology and Hydraulic Modeling.

Peak flow rates for the design storm should also be calculated, using the methods outlined in Appendix J so that the inlet and pretreatment can be accordingly sized and flow attenuation can be considered

Step 2. BMP Siting

Bioretention is a versatile stormwater BMP that can effectively reduce pollutants and can be integrated into site plans with various configurations and components. Stormwater treatment should be considered as an integral component and incorporated in the site design and layout from conception. Many times, determining how the bioretention area will be included in the site design is a critical and required first step. How the water is routed to the bioretention area and the available space will be key components in determining how the bioretention area is configured. The following is a list of settings where bioretention can be incorporated to meet more than one project-level or watershed-scale objective:

- Landscaped parking lot islands
- Common landscaped areas
- In parks and along open space edges
- Within rights-of-way along roads.

How the bioretention area is configured will determine the required components. Bioretention areas can serve the dual purpose of stormwater management and landscape design and can significantly enhance the aesthetics of a site. When siting bioretention, consideration must always be given to providing access for routine, intermittent, and rehabilitative maintenance activities.

Treatment Trains
Bioretention areas can be combined with other BMPs to form a treatment train that can provide enhanced water quality treatment and reductions in runoff volume and rate. For example, runoff can be collected from a roadway in a vegetated swale that then flows to a bioretention area. Both facilities can be reduced in size on the basis of demonstrated performance for meeting the stormwater runoff and addressing targeted pollutants of concern.

Step 3. Determine BMP Function and Configuration

Intended bioretention functions and configuration must be characterized early in the design process. Infiltration through native soils provides the greatest treatment potential and lowest cost. Where infiltration is limited, a high level of treatment can still be provided by filtering stormwater through an engineered soil media. The following subsections describe the necessary steps to determine if the bioretention area will safely function as an infiltration or filtration BMP.

Peak flow rates for the design storm should also be calculated, using the methods outlined in Appendix J, so that the inlet and pretreatment can be accordingly sized and flow attenuation can be considered.

Geotechnical Investigation
A licensed soil scientist or geotechnical engineer should conduct a geotechnical investigation before the BMP design. The investigator should determine the infiltration rate of the soils at the potential subgrade of the bioretention cell, the depth to the seasonally high groundwater table, presence of expansive clay minerals, and whether there is a risk of sinkhole formation. Site location with respect to aquifer recharge zones, steep slopes, water supply wells, and septic drain fields must also be assessed. For more details, see Appendix A, Common Design Elements.

Determine if Underdrains and Impermeable Liners are Needed
Underdrains will be required if a bioretention cell is lined, adjacent to a steep slope, or if the subsoil infiltration rate (as determined during the geotechnical analysis) is less than 0-0.5 inch per hour (in/hour). Use Table B-2 to determine if a bioretention area requires an impermeable liner or underdrain. For more information concerning the use of fully lined bioretention, see Planter Boxes.

TABLE B-2. DECISION TABLE FOR DETERMINING UNDERDRAIN AND IMPERMEABLE LINER REQUIREMENTS

Impermeable liners must be used if...	Underdrains must be used if...
• Site is in Edwards Aquifer Recharge Zone, Contributing Zone, or Transition Zone (Barrett 2005) • Soil contamination is expected or present • Karst geology presents risk of sinkhole formation • Runoff could unintentionally be received from a stormwater hotspot • Site is within 100 feet of a water supply well or septic drain field • Site is within 10 feet of a structure/foundation • Infiltrated water could interfere with utilities	• An impermeable liner is needed • Infiltration rate of underlying soils is less than 0-0.5 in/hour • Site is within 50 feet of a steep, sensitive slope (as determined in the geotechnical analysis—see Appendix A, Common Design Elements)

Determine if Lateral Hydraulic Restriction Barriers are Needed
When bioretention areas are near sensitive infrastructure such as pavement subgrades or buried utilities, hydraulic restriction barriers are often required to prevent lateral seepage. Hydraulic restriction barriers are often installed the full depth of excavation, but occasionally they are keyed in to greater depths to ensure vertical, deep infiltration; the geotechnical investigator should determine the required extent of hydraulic restriction barriers. Appendix A, Common Design Elements, provides specific details concerning lateral hydraulic restriction barrier design.

Design Underdrain and Internal Water Storage
The underdrain configuration greatly affects the gradient for water movement through a bioretention cell, and the hydrologic and water quality performance. Conventionally drained cells feature an underdrain that freely drains and outlets at the elevation of the subgrade. Infiltration and pollutant load reduction can be further enhanced by upturning the underdrain to create a sump (Brown and Hunt 2011a). This internal water storage (IWS) zone enhances exfiltration into underlying soils while maintaining aerobic soil conditions in the plant rooting zone. It is most convenient to upturn the underdrain in the outlet structure using a tee-connection; this allows easy access to the underdrain for inspection and maintenance. IWS can be used in conjunction with an impermeable liner, but volume calculations must account for the possibility of prolonged saturation in the lower media. Inclusion of IWS is recommended in arid and semi-arid regions (such as San Antonio) to maintain soil moisture for plant health (Li et al. 2010; Barrett et al. 2012; Houdeshel et al. 2012). To provide an aerobic root zone and to reduce mobilization of previously captured pollutants, the IWS zone should be at least 18 inches below the surface (Hunt et al. 2012). For recommended underdrain general guidance, see Appendix A, Common Design Elements.

Step 4. Size the System

The required water quality treatment volume is determined in Appendix J. Vertical dimensions should be selected on the basis of pollutants of concern and site constraints before calculating the BMP footprint. The following subsections provide guidance on sizing the surface ponding depth, media depth, and footprint of bioretention areas.

Surface Ponding Depth
Bioretention areas should have a maximum ponding depth of 12 inches but can temporarily detain runoff to a depth of 18 inches if designed for peak flow mitigation (Heasom et al. 2006; more detail concerning peak flow mitigation is provided in Step 7). Although research has demonstrated excellent performance from bioretention areas with deeper ponding depths (more than 12 inches), greater care must be taken to select vegetation that can withstand both inundation and drought, and public safety must be considered (Hunt et al. 2012). Maximum ponding depth might also be limited by vertical constraints of the site, including the elevation of existing downstream storm drain networks. For these reasons, a 9-inch average ponding depth is typically preferred. Local freeboard requirements (typically 1.0 foot for online systems and 0.5 foot for offline systems) should also be considered when selecting a ponding depth (Barrett 2005).

Soil Media Depth
Soil media depth should be optimized to meet hydrologic and water quality goals but should have a minimum depth of 2 feet (3 feet is recommended for systems with IWS; Hunt et al. 2012). The soil media provides a beneficial root zone for the chosen plant palette and adequate water storage for the water quality volume. A deeper soil media depth will provide a smaller surface area footprint by allowing more storage in the pore spaces and subsequently more evapotranspiration of stormwater by plants.

Figure B-2: *Upturned underdrains inside bioretention outlet structures create IWS in soil media to improve infiltration, water quality, and plant health, North Carolina. Source: Tetra Tech*

Table B-3 summarizes the minimum recommended media depths for targeted removal of various pollutants. Considering the target pollutant, the depth of the media in a bioretention cell should be between 2 and 4 feet. That range reflects where pollutant removal occurs, and excavations deeper than 4 feet become more expensive. The depth should accommodate the desired vegetation (shrubs or trees). If the minimum depth of 2 feet is used over restrictive underlying soils or an impermeable liner, only shallow-rooted vegetation should be planted; grassed bioretention cells can be as shallow as 2 feet. Bioretention facilities where shrubs or trees are planted could be as shallow as 3 feet unless a soil test indicates that shallower depths will support plant health. Media depths greater than 3 feet might be desired for additional pollutant removal, thermal load reduction, and hydrologic benefits, but 3 feet is typically sufficient. If large trees are to be planted in deep fill media, care should be taken to prevent overturning in high winds. Stakes and guy lines might be required to stabilize the trees during establishment.

TABLE B-3. MINIMUM BIORETENTION DEPTH TO TREAT POLLUTANTS OF CONCERN (HUNTER ET AL. 2012)

Pollutant of concern	Removal zone	Recommended depth
Sediment	Surface, top 2-8 inches	2 feet
Total nitrogen	At depth in IWS layer (>2 feet)	3 feet
Total phosphorus	Top 1-2 feet	2 feet
Pathogens	Top 1-2 feet	2 feet
Metals	Top 1-2 feet	2 feet
Oil & grease and Pathogens	Surface	2 feet
Temperature	At depth	4 feet

Size Surface Area

The footprint of the bioretention area should be calculated after the desired ponding depth and soil media depth have been selected. Bioretention areas should be sized to fully capture the treatment volume (from Appendix J) within the surface ponding zone and subsurface pore space. Available storage in the subsurface soil media and gravel drainage layer should be determined on the basis of the laboratory-measured porosity of materials that will be installed on-site; this information is typically available from suppliers or quarries. The porosity, n, of bioretention media can be estimated as 0.35, and the porosity of ASTM No. 57 gravel can be estimated as 0.40 for preliminary calculations (Brown et al. in press).

[Equation B-1-1]

$$n=\frac{V_v}{V_T}$$

where:

n = porosity (volume/volume)

V_v = volume of void space

V_T = total volume

The equivalent storage depth for a unit bioretention cross section can be calculated as follows:

[Equation B-1-2]

$$D_{eq}=\left(D_{surface}\right)+\left(n_{media}\times D_{media}\right)+\left(n_{gravel}\times D_{gravel}\right)$$

Barrett, M.E. 2005. Complying with the Edwards Aquifer Rules. Technical Guidance on Best Management Practices. RG-348. Prepared for Texas Commission on Environmental Quality, Field Operations Division, Austin, TX.

where:

D_{eq} = equivalent depth of water stored in representative cross sectional of bioretention

$D_{surface}$ = average depth of temporary surface ponding (maximum 12 inches)

n_{media} = porosity of soil media

D_{media} = depth of soil media

n_{gravel} = porosity of gravel drainage layer

D_{gravel} = depth of gravel drainage layer

If the bioretention area is being used for peak flow mitigation, the detention storage depth (volume that will bypass the soil media) cannot be included in Dsurface. More information is provided in Step 7.

The treatment volume (Vwq) is divided by the equivalent depth (Deq) to calculate the required bioretention footprint:

[Equation B-1-3]

$$A=\frac{V_{wq}}{D_{eq}}$$

where:

A = required bioretention footprint (area)

Vwq = water quality treatment volume (determined in Appendix J)

Deq = equivalent depth

Step 5. Specify Soil Media

Bioretention areas are intended to drain to below the surface in less than 24 hours, however, it is recommended that they be designed to drain in 12 hours or less as a safety factor. Typically the soil media is dewatered in less than 48 hours for plant health. If a gravel drainage layer is included beneath the bioretention area soil media, stored runoff in the drainage layer should drain in less than 72 hours. The soils must be allowed to dry out periodically to restore hydraulic capacity to receive flows from subsequent storms, maintain infiltration rates, maintain adequate soil oxygen levels for healthy soil biota and vegetation, and to provide proper soil conditions for biodegradation and retention of pollutants.

Organic matter is considered an additive to help vegetation initially establish and contributes to sorption of pollutants; however, organic materials will oxidize over time causing an increase in ponding that could adversely affect the performance of the bioretention area. Additionally, studies in Texas have demonstrated pollutant leaching when bioretention soils were amended with excessive compost (Li et al. 2010). Organic material should therefore be minimized (less than 5 percent of media volume) and consist of minimal plant-based materials. Organic amendments should not include any animal manure or by-products, which can export nutrients and pathogens.

Figure B-3: *Gravel fringe and vegetated filter strip pretreatment, Louisburg, North Carolina. Source: North Carolina State University Department of Biological and Agricultural Engineering*

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High levels of phosphorus in the media have been identified as the main cause of bioretention areas exporting nutrients (Hunt and Lord 2006). All bioretention media should be analyzed for background levels of nutrients. All soil properties should be measured by a qualified soils laboratory with AASHTO, USACE, or State accreditation.

Soil media should meet the general guidance listed in Table B-4. If the existing soils meet the criteria, it can be used as the soil media. If the existing soils do not meet the criteria, soils should be amended with the appropriate components or a substitute media must be used.

TABLE B-4. EXAMPLE OF BIORETENTION SOIL MEDIA SPECIFICATIONS (HUNTER ET AL. 2012)

Parameter	Guidance
Texture and Composition (by volume)	• Soil media should consist of a loamy sand conforming to the following general guidance: • 85 to 88% washed course sand (concrete sand passing a one-quarter-inch sieve or thoroughly washed mortar sand passing a one-eighth-inch sieve) • 8 to 12% fines passing a #270 sieve (8% fines typically yields an infiltration rate near 2 in/hour, whereas 12% fines yields an infiltration rate near 1 in/hour) • 2 to 5% organic matter
Organic Matter Material	Aged bark fines, hardwood chips, leaf litter, or similar plant-derived, composted organic material screened to 3/8 in or less. Studies have also shown newspaper mulch to be an acceptable additive (Kim et al. 2003; Davis 2007). Organic matter should not include animal manure or by-products
Infiltration Rates	0.5 to 6 in/hour (1-2 in/hour recommended for comprehensive pollutant treatment and hydrologic benefit; Hunt et al. 2012)
pH	6 to 8
Cation Exchange Capacity (CEC)	Greater than 5 milliequivalents (meq)/100 g soil
Phosphorus	Total phosphorus should not exceed 15 ppm

Step 6. Design Inlet and Pretreatment

Inlets must be designed to convey the design storm volume into the bioretention area while limiting ponding or flooding at the entrance to the bioretention area and protecting the interior of the bioretention area from damage. Take care during grading to ensure that the drainage area is properly sloped toward the bioretention area and that the inlet elevation is at least as high as the intended maximum ponding depth (for more information, see Appendix F, Critical Construction Considerations). In addition to inlet design, pretreatment is critical to remove coarse sediment and debris to prolong the functional life of the soil media. Several options are available depending on the configuration of the bioretention area and the drainage area characteristics.

Inlets
The way in which runoff is routed to the bioretention area will dictate the type of inlet. If sheet flow constitutes the source of runoff, curb cuts are typically used; design guidance for curb cuts is provided in Appendix A, Common Design Elements section. If flows are concentrated, channels or conduit can be used to convey runoff to the bioretention area.

Energy Dissipation and Pretreatment
Design of pretreatment measures will vary depending on the site layout. If sheet flow (such as parking lot runoff) is conveyed to the treatment area, the site must be graded in such a way that minimizes erosive conditions. Gravel fringes between pavement and grassed surfaces can help distribute flow and provide initial pretreatment. Gravel should consist of a 2-inch layer of ASTM No. 57 stone (underlain by filter fabric) extending 2 to 3 feet from the pavement edge, where space allows (Figure B-3). Filter strips should ideally be sodded and graded at 3:1 (horizontal:vertical) slopes or flatter. Any slopes that convey flow should be routinely inspected for rill erosion, which can contribute excessive sediment to the bioretention area and often represents the most common maintenance issue (Wardynski and Hunt 2012). Take care to prevent flow from concentrating between parking lot curb stops/blocks.

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Runoff can be routed to a bioretention area through a vegetated swale to pretreat incoming flows from impervious surfaces. Whenever concentrated flow is conveyed to the bioretention area (via channels or conduit) a rock-armored forebay should be used to dissipate energy and provide pretreatment of gross solids and sediment. Forebays should compose approximately 10 percent of the total bioretention area and should be designed to dewater between storm events to prevent vector hazards (Hunt and Lord 2006, Hunt et al 2007). Armored inlets can be used where space is limited (as shown in Figure B-4 and Figure B-5).

Bioretention areas that treat runoff from residential roofs or other cleaner (low sediment and debris yield) surfaces might not require pretreatment for trash or sediment but should include energy dissipation to the extent practicable. Energy dissipation can be provided by upturning inflow pipes so that they bubble up diffusely onto a rock apron (Figure B-6); otherwise, baffles, blocks, or cobbles can be used to still high velocities. Flow velocities should not exceed 3 feet per second (ft/sec) for grassed surfaces and 1 ft/sec for mulched surfaces.

Figure B-4 (left): *Inlet and pretreatment provided by mortared cobble forebay and energy dissipater, Los Angeles, California. Source: Tetra Tech*

Figure B-5 (center): *Inlets stabilized with mortared cobble, Los Angeles, California. Source: Tetra Tech*

Figure B-6 (right): *Upturned inlet from rooftop bubbles up diffusely onto gravel pad, Chocowinity, North Carolina. Source: Tetra Tech*

Step 7. Select and Design Overflow/Bypass Method

Two design configurations (offline or online) can be used for treating storms that are larger than the bioretention area is designed to store. If peak flow cannot be fully mitigated by the flow rate through the soil media, the outlet can be adapted to meter the rate of outflow.

Offline
An offline bioretention area (Figure B-7) can be designed such that stormwater bypasses the bioretention area once the capacity has been exceeded. A structure can also be designed that diverts into the bioretention area only the volume of stormwater for which the bioretention area is designed. For more information on diversion structures, see Appendix A, Common Design Elements.

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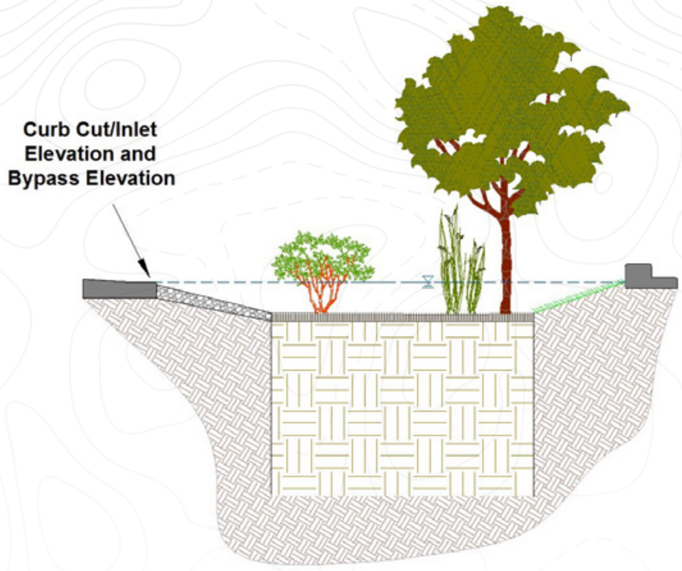

Figure B-7: *Offline bioretention area where system fills to capacity and excess flow bypasses along curbline at inlet Online*

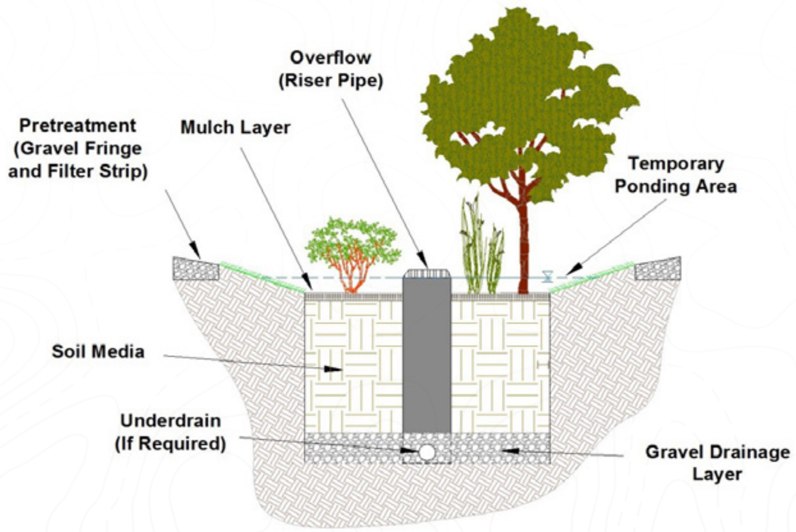

Figure B-8: *Online bioretention area with a vertical rise overflow with a variable flow outlet structure*

For online systems, all flow is routed through the bioretention area and excess runoff overflows an outlet structure. Outlet systems for online bioretention areas can be designed to provide some peak flow mitigation in addition to storing the design volume (see Designing for Peak Flow Mitigation). Appropriate energy dissipation should be incorporated in online systems such that media is not scoured during higher flow events. Two basic options can be used for outlets or overflow for online bioretention systems.

Option 1: Vertical riser

1. An elevated outlet structure (typically an above-grade concrete drop inlet for larger bioretention areas or a PVC pipe for smaller bioretention areas) that is connected to the underdrain or directly to the drainage system.
2. The vertical riser should be sized to safely convey flow greater than the water quality volume. The vertical pipe will provide access for cleaning the underdrains.
3. The inlet to the riser should be set at the specified ponding depth and capped with an appropriate non-clogging grate. Figure B-8 shows an example of an online bioretention area with a vertical riser overflow design.

Option 2: Level spreader

1. A level spreader can be used to diffuse overflows from the bioretention area and should be installed along the exit edge or outflow section of the bioretention area. The level spreader should be concrete.
2. The top surface of the level spreader should be installed at a height equal to the ponding depth, or slightly greater if in conjunction with a vertical riser, to allow runoff exceeding the capacity of the bioretention area to safely pass.
3. The level spreader can be designed as a weir to allow for varied outlet flows providing some peak flow mitigation.
4. See Appendix A, Common Design Elements for details on level spreader design.

Typically, bioretention areas constructed in the right-of-way should be designed as offline stormwater treatment facilities. Once a bioretention area constructed in the right-of-way has reached capacity, stormwater flows should bypass the system and continue flow in the existing stormwater drainage system or continue to the next BMP. If a bioretention area constructed in the right-of-way requires underdrains, a vertical riser overflow system can be incorporated as the primary overflow method in addition to the bypass.

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Designing for Peak Flow Mitigation
Bioretention areas can be designed for peak flow mitigation by providing additional storage and, if necessary, modifying the outlet structure to discharge water at a controlled rate. Some additional water can be retained in the system above the water quality treatment volume for a short period without affecting the vegetation. If additional ponding depth is provided to store the flood control volume, maximum ponding depth must not exceed 18 inches. The riser should be designed to mitigate for the required peak flow without exceeding the maximum ponding time of 24 hours. This requirement can be achieved by incorporating an orifice or a weir with its invert at the elevation of the water quality treatment volume ponding depth (Figure B-9). Orifices that could be clogged by floating mulch or debris should be protected with a trash rack, a hood, or by installing a downturned pipe (for design of non-clogging orifices, see Stormwater Wetlands). The volume of water detained above the elevation of the drawdown orifice or weir cannot be credited toward the water quality treatment volume because this excess water will drain untreated to the storm sewer network without filtering through the soil media. Alternatively, underdrain outflow can be regulated using a restrictor plate, and all runoff can be routed through the soil media.

Figure B-9: *Bioretention outlet structures designed for peak flow mitigation in Camp Pendleton, California (left) where a graduated riser pipe regulates drawdown of the detention volume; and Southwest Middle School, Gastoria, North Carolina (right) where orifices allow controlled dewatering of the detention volume and water quality treatment volume is retained below the orifice elevation. Source: Tetra Tech*

Discharge of the detention volume through orifices and weirs can be calculated using the following equations. For further guidance on hydraulic design, refer to USDA-SCS (1956) or Chow (1959).

[Equation B-1-4]

Orifice: $Q=C_d A\sqrt{2gH}$

[Equation B-1-5]

Weir: $Q=CLH^{3/2}$

where:
- Q = discharge (cubic feet per second)
- C_d = coefficient of discharge (0.6 for sharp openings, 0.8 for pipe openings)
- A = cross sectional area of orifice (square feet)
- g = acceleration due to gravity (32.2 ft/s^2)
- H = head of water acting on the structure (height of water over the centerline of the orifice or height of water over the crest of the weir; feet
- C = discharge coefficient (3.33 for broad-crested weir, 3.0 for sharp crested weir)
- L = total length of weir (perpendicular to flow; feet)

Step 8. Select Mulch and Vegetation

Both mulch and vegetation are critical design components of bioretention areas from hydrologic, water quality, and aesthetic perspectives. Much of the biological activity in bioretention areas occurs in the mulch and root zone. The following subsections provide general guidance for mulch and vegetation.

Mulch

Mulch is a critical component of the bioretention area because it provides a food source and habitat for many of the biological organisms critical to the function of the bioretention area. Much of the hydrocarbon, metals, and total suspended solids removal is believed to occur near the surface in the mulch layer (Hong et al. 2006; Hatt et al. 2008; Li and Davis 2008; Stander and Borst 2010). The bioretention area should be covered with mulch when constructed and annually replaced to maintain adequate mulch depth. Mulch is also important to sustain nutrient levels, suppress weeds, retain moisture for the vegetation, and maintain infiltrative capacity. Mulch should meet the following criteria:

1 Dimensional chipped hardwood material (Figure B-10) is preferred for its permeability of both water and air. Well-aged, triple-shredded hardwood material can also be used if dimensional chipped hardwood material is unavailable (well-aged mulch is defined as mulch that has been stockpiled or stored for at least 12 months).

2 Free of weed seeds, soil, roots, and other material that is not hardwood material.

3 Mulch depth will be 2 to 4 inches thick, with 3 inches preferred (thicker applications can inhibit proper oxygen and carbon dioxide cycling between the soil and atmosphere).

4 Grass clippings, pecan shells, or pure bark should not be used as mulch.

Figure B-10: *Triple-shredded hardwood mulch*

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Vegetation

One advantage of bioretention areas is that they can be used for the dual purpose of stormwater treatment and landscaping or be integrated into the existing landscape. Bioretention areas can be used toward meeting the 40 percent tree canopy cover goal of San Antonio's SA2020 Plan. For bioretention areas to function properly as stormwater treatment and blend into the landscape, vegetation selection is crucial. Appropriate vegetation will have the following characteristics:

1. Plant materials must be tolerant of summer drought and extreme heat, ponding fluctuations, and saturated soil conditions for 12 to 48 hours.
2. It is recommended that a minimum of three tree, three shrub, and three herbaceous groundcover species be incorporated to protect against facility failure from disease and insect infestations of a single species.
3. Vegetation with deep and extensive root systems are more tolerant of extreme hydroperiods and can effectively transpire large volumes of soil water. Planting deep-rooting vegetation directly above buried underdrains should be avoided (although interference of plant roots with underdrains is not a common maintenance issue).
4. Native plant species or noninvasive adapted cultivars that do not require chemical inputs are recommended to be used to the maximum extent practicable. Only native and noninvasive species will be selected in areas designated as natural open space.
5. Shade trees should be free of branches for the bottom 1/3 of their total height and lines of sight must be maintained when planting along roadways.
6. Tree height and placement should consider overhead utilities.
7. If turfgrass is preferred, sod should be specified that was not grown in clay soils (or washed bare root sod should be specified).
8. An example list of native plants appropriate for bioretention areas in the San Antonio region is in Appendix E.

Many options exist for vegetation arrangement and will most likely depend on the landscaping of the area around the bioretention. Size-limited landscaping could be required for bioretention areas in the right-of-way to maintain the required sight distances. Consideration should be given to water depth, bioretention configuration, desired aesthetic appearance, and potential multi-use benefits (Appendix C). An example planting plan is shown in Figure B-11 and a plant list for the San Antonio region is provided in Appendix E.

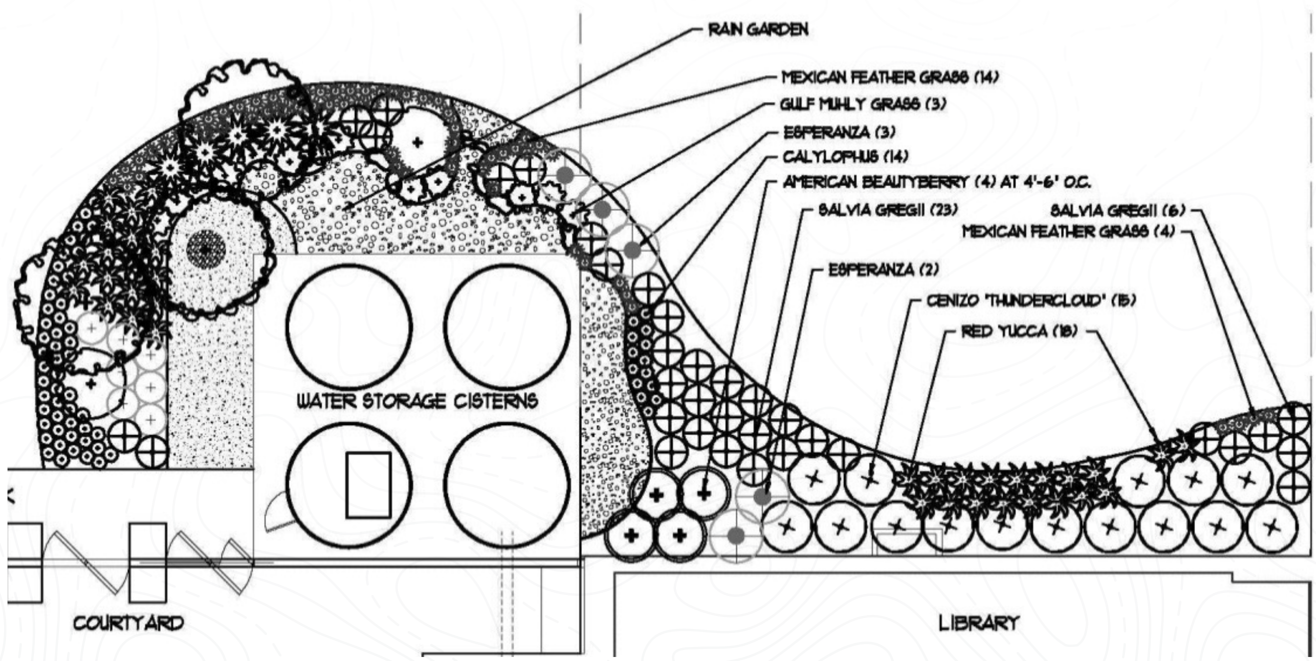

Figure B-11: *Example bioretention planting plan*

APPENDIX B: **Additional Bioretention Design Guidance**

Encroachment of turf grasses present a long-term maintenance challenge. The vegetation plan should be designed in order to delay encroachment of problem grass species in the bioretention feature and facilitate maintenance activities to prevent or remove growth of unwanted grass within the bioretention. In order to keep grass out of planting areas use all of the following methods:

1 Isolate planting areas. Do not plant Bermuda in or near planting areas. Where applicable plant less aggressive grass species.

2 In areas already bordered by Bermuda grass:

a. Install edging at least 8" below the surface.
b. Install edging at least 2" above the surface.
c. Install geotextile weed barrier at least 18" on planting side of edging.
d, Cover geotextile with a minimum of 4" of inorganic mulch.

Peak flow rates for the design storm should also be calculated, using the methods outlined in Appendix J so that the inlet and pretreatment can be accordingly sized and flow attenuation can be considered.

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APPENDIX B: **Additional Bioswale Design Guidance**

Bioswale

DESIGN

Figure B-12: *Urban at Olive. San Antonio, Texas*

The design of a bioswale is very similar to a bioretention area and can similarly be broken down to a nine-step process, as outlined in Table B-5. Unlike vegetated swales that provide limited horizontal filtration and sedimentation, bioswales are intended to filter runoff vertically through soil media; conveyance should be considered a secondary design element.

APPENDIX B: **Additional Bioswale Design Guidance**

TABLE B-5. BIOSWALE ITERATIVE DESIGN STEP PROCESS

Design step		Design component/ consideration	General guidance
1	**Determine BMP Size (pg 181)**	Use Appendix J	
2	**BMP Siting (pg 181)**	Based on available space and maintenance access, incorporate into parking lot islands, medians, and perimeter; install along the roadway right-of-way; incorporate as landscaped areas throughout the property.	
3	**Determine BMP Function and Configuration (pg 182)**	Impermeable liner	If non-infiltrating (per geotechnical investigation), use impermeable clay liner, geomembrane, or concrete (as described in Appendix A, Common Design Elements).
		Lateral hydraulic restriction barriers	Use concrete or geomembrane to restrict lateral seepage to adjacent subgrades, foundations, or utilities.
		Underdrain (required if subsoil infiltration rate is less than <0.5 in/hour)	Schedule 40 PVC pipe with perforations (slots or holes) every 6 inches. 4-inch diameter lateral pipes should join a 6 inch or larger collector pipe, depending on design, which conveys drainage to the downstream storm network. Use a 6-inch or larger pipe depending on design. Provide cleanout ports/observation wells for each underdrain pipe at spacing consistent with local regulations. See Appendix A, Common Design Elements
		Internal water storage (IWS)	If using underdrain and infiltration, elevate the outlet to create a sump for additional moisture retention to promote plant survival and treatment. Top of IWS should be greater than 18 inches below surface.
		No underdrain	If design is fully infiltrating, ensure that subgrade compaction is minimized.
4	**Size the System (pg 182)**	Temporary ponding depth	Use check dams to provide 6-18 inches surface ponding (6-12 inches near schools or in residential areas); average ponding depth of 9 inches is recommended
		Soil media depth	2-4 feet (deeper for better pollutant removal, hydrologic benefits, and deeper rooting depths)
		Slope and grade control	If necessary, use check dams to maintain maximum 2% bed slope. Install a 4-inch deep layer of ASTM No. 57 stone (underlain by filter fabric) extending 2 feet downslope from check dam to prevent erosion.
		Surface area	Accounting for slope, find surface area required to store treatment volume within temporary ponding depth, soil media depth, and gravel drainage layer depth (media porosity ≈ 0.35 and gravel porosity ≈ 0.4)
5	**Specify Soil Media (pg 184)**	Media composition and texture	85-88% sand, 8-12% fines, 2-5% plant-derived organic matter (animal wastes or by-products should not be applied)
		Media permeability	1-6 in/hour infiltration rate (1-2 in/hour recommended)
		Chemical analysis	Total phosphorus < 15 ppm, pH 6-8, CEC > 5 meq/100 g soil
		Drainage layer	Separate soil media from underdrain with 2–4 inches of washed concrete sand (ASTM C33), followed by 2 inches of choking stone (ASTM No. 8) over a 1-5 ft envelope of ASTM No. 57 stone.

TABLE B-5. BIOSWALE ITERATIVE DESIGN STEP PROCESS (CONT.)

6	**Design Inlet and Pretreatment (pg 184)**	Inlet	Provide stabilized inlets (see Appendix A, Common Design Elements)
		Pretreatment	Install rock armored forebay (concentrated flow), gravel fringe and vegetated filter strip (sheet flow), or vegetated swale
7	**Select and Design Overflow/Bypass Method (pg 184)**	Outlet configuration	Online: All runoff is routed through system—install an elevated overflow structure or weir at the elevation of maximum ponding Offline: Only treated volume is diverted to system—install a diversion structure or allow bypass of high flows
8	**Select Mulch and Vegetation (pg 184)**	Peak flow mitigation	Provide additional detention storage and size an appropriate non-clogging orifice or weir to dewater detention volume
9	**Design for Multi-Use Benefits (Appendix C)**	Include features to enhance habitat, aesthetics, public education, and shade.	

Step 1. Determine BMP Size

The bioswale must be sized to fully capture the desired or required design storm volume and filter it through the soil media. Surface storage (in the ponding area) and soil pore space (in the plant rooting zone and the underlying media and gravel drainage layers) provide capacity for the design storm volume retention. Appendix J outlines methods for determining design runoff depths associated with a range of annual treatment efficiencies. Once the design runoff depth is determined (on the basis of the desired level of treatment), a runoff volume can be determined for the contributing watershed using this depth and the methods outlined in Appendix J, San Antonio Unified Development Code, or San Antonio River Basin Regional Modeling Standards for Hydrology and Hydraulic Modeling.

Peak flow rates for the design storm should also be calculated, using the methods outlined in Appendix J so that the inlet and pretreatment can be accordingly sized and flow attenuation can be considered.

Step 2. BMP Siting

Bioswales are intended to provide the same function as a bioretention area with the same pollutant-removal capacity with a narrow width to be more easily configured into the site plan for parking lot edges and narrow rights-of-way. Bioswales are a versatile stormwater BMP because they can effectively reduce pollutants and can be integrated into the site plan with various configurations and components. Stormwater treatment should be considered as an integral component and incorporated in the site design and layout from conception. Many times, determining how the bioswale will be included in the site design is a critical and required first step. How the water is routed to the bioswale and the available space will be key components in determining how the bioswale will be configured. Access for maintenance activities must also be provided. The following is a list of settings where bioswales can be incorporated to meet more than one project-level or watershed-scale objective:

- Along the edge and between parking stalls in parking lots
- Within rights-of-way along roads

How the bioswale is configured determines the required components. Pretreatment at some level is always recommended to remove gross solids where possible and reduce flows to a non-erosive rate. Curb cuts can be required to allow stormwater to enter the bioswale, while providing some delineation in high-traffic areas. Bioswales can serve the dual purpose of stormwater management and landscape design and can significantly enhance the aesthetics of a site. Bioswales typically have multiple components including the following:

- Filter strip or grass buffer for pretreatment
- Media layer for filtration
- Ponding area for storage
- Plants for pollutant uptake and landscaping

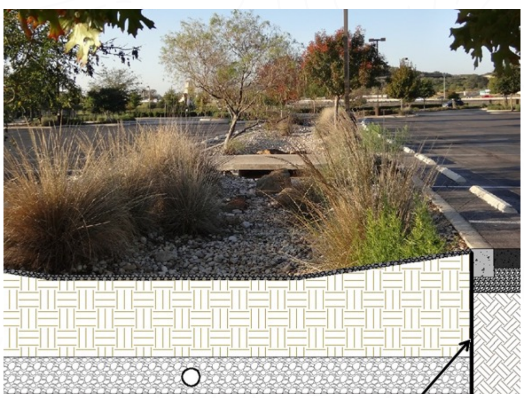
APPENDIX B: **Additional Bioswale Design Guidance**

In addition, bioswales can be combined with other BMPs to form a treatment train that can provide enhanced water quality treatment and reductions in runoff volume and rate. For example, runoff can be collected from a roadway or a parking lot **in a bioswale that then overflows to a bioretention area. Both facilities can be reduced in size on the basis of demonstrated** performance for meeting the stormwater runoff requirements and addressing targeted pollutants of concern.

Step 3. Determine BMP Function and Configuration

Bioswales can be designed as infiltrating or filtering BMPs, similar to bioretention. Because of the narrow configuration of a bioswale and its intended use along the edges of parking lots and roads, infiltration pathways will most likely need to be restricted to prevent unintended effects on roads, foundations, other infrastructure, or hotspot locations. In some conditions, lateral seepage can cause damage to surrounding structures depending on the type of soils in the area (Figure B-13). Areas that have a potential for settling under saturated conditions, as determined in the geotechnical investigation, should be protected from lateral flows. Types of clay that have a high potential for expansion when saturated should be protected from moisture in load-bearing conditions. For details on hydraulic restriction barriers, see Appendix A, Common Design Elements.

Where infiltration is allowed, IWS can be used to enhance exfiltration, pollutant removal, and soil moisture for plant health (Li et al. 2010; Brown and Hunt 2011; Barrett et al. 2012; Houdeshel et al. 2012). As with bioretention, the IWS zone should be at least 18 inches below the surface throughout the length of the bioswale. Excavating the subgrade in tiers and creating partitions between cells of the bioswale will further improve performance by providing more uniform exfiltration across the subgrade (as illustrated in Figure B-14).



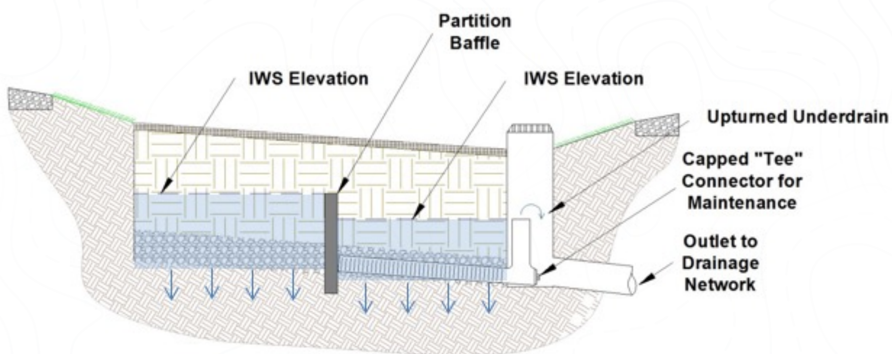


Figure B-13: (left) *Bioswale at Rim Retail Center, San Antonio, Texas showing example cross section with hydraulic restriction barrier to prevent lateral seepage to adjacent pavement subgrade.*
Figure B-14: (right) *Bioswale incorporating and a partition baffle to enhance exfiltration*

Step 4. Size the System

The bioswale water quality treatment volume can be determined by using Appendix J. The following subsections discuss sizing bioswales for water quality and conveyance.

Geometry, Temporary Ponding Depth, and Soil Media Depth
Bioswale dimensions have similar standards to bioretention areas except that they are typically long and narrow with widths between 2 and 8 feet. Bioswales have a maximum ponding depth of 18 inches, with an average depth of 9 inches. Soil media depth should be specified according to the pollutant of concern, hydrologic goals, and drainage configuration, as outlined in the bioretention section.

Slope and Grade Control
Bioswales are to be sized to capture and treat the volume produced by the design storm and, where site conditions allow, should also be sized to infiltrate the volume-reduction requirement. For the stormwater runoff requirements and calculations, see Appendix J. If the bioswale will have longitudinal slope (parallel to flow), flow velocity should generally not exceed 1 ft/sec in mulched swales and 3 ft/sec in grassed swales and the shear stress should not exceed the permissible shear stress of the bed materials, as suggested in TxDOT (2011). Guidance for calculating flow velocity is provided in the vegetated swales section. Check dams might be required to ensure retention and infiltration of the design storm volume into the soil media. The maximum bed slope of the bioswale may not exceed 2 percent to prevent erosion, but

bioswales with check dams may contain average slopes (from upslope to downslope end) of up to 5 percent (the bed slope of each section between check dams must be 2 percent or less). Check dams should be adequately embedded in the side slopes and can be constructed of concrete, metal sheet pile, or wood (Figure B-15). Earthen and stone check dams should not be used because of risk of erosion. The area downslope of check dams should be armored with at least a 4-inch-deep gravel or cobble layer extending 2 feet from the base of the check dam (as shown in Figure B-16). Gravel should consist of No. 57 stone and should be underlain by geotextile to prevent scour and erosion of underlying soil. Cobble can be mortared to prevent removal.

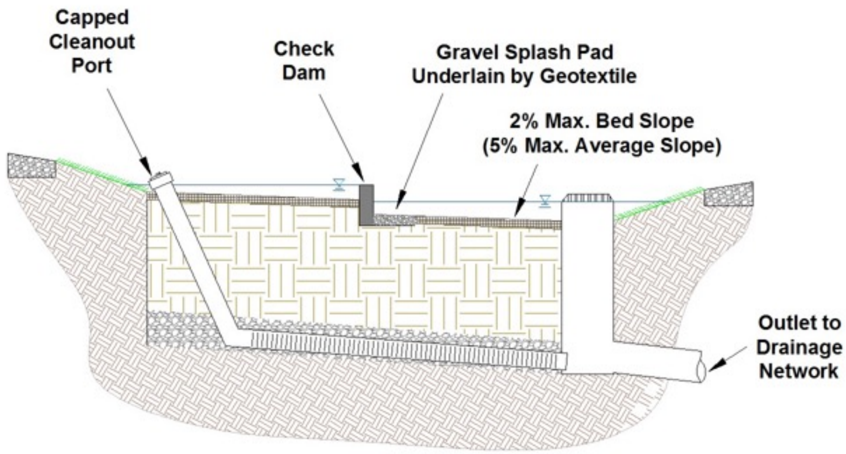


Figure B-15 (left): *Bioswale with a check dam, Los Angeles, California. Source: Tetra Tech*
Figure B-16 (right): *Example profile of a bioswale with a check dam to retain the design storm volume*

Size surface area

The footprint of the bioswale should be calculated following the methods provided in the bioretention section, but slope must be taken into account when specifying an average ponding depth. The required surface area can be calculated using the equations in the bioretention section assuming a nine-inch ponding depth. The swale configuration should be adjusted to attain the desired surface area. If the bed of the bioswale is sloped, the required number of check dams to create the desired ponding depth can be estimated using the following equations:

[Equation B-2-1]

$$N=\frac{L_{swale} x S}{h_{dam}}$$

[Equation B-2-2]

$$L_{dam}=\frac{L_{swale}}{N}$$

where:

N = number of check dams required

L_{swale} = total length of bioswale (ft)

S = longitudinal slope of bioswale (ft/ft)

h_{dam} = $(2 \times D_{surface})$ = height of check dams (ft; use a maximum height of 1-5)

L_{dam} = distance between check dams (ft)

The above equation is simplified and should be adjusted on the basis of specific site conditions and bioswale configuration. The slope of a site can vary, the number of check dams required should be calculated separately for each significant change in slope.

Step 5. Specify Soil Media

The soil media in the bioswale should meet the requirements specified in the bioretention section.

Step 6. Design Inlet and Pretreatment

Inlets must be designed to convey the design storm volume into the bioswale while limiting ponding or flooding at the entrance and protecting the interior from damage. Several options are available depending on the configuration of the bioswale. Ideally, runoff will pass over a filter strip where flow can be dispersed and gross solids removed before it enters the bioswale. That is not always possible, especially in retrofit situations were space might not be available. Methods used for designing bioretention inlets and pretreatment are applicable to bioswales. Typical inlet configurations are also described in Appendix A, Common Design Elements.

Step 7. Select and Design Overflow/Bypass Method

Bioswales, like bioretention, can be designed as either offline or online systems (for design guidance, see bioretention). Examples of offline and online bioswales are shown in Figure B-17. When flows through a bioswale could exceed the recommended maximum flow rates, regardless of whether a system is designed to be online or offline, a bypass structure is recommended to prevent erosion in the bioswale. The flow velocity in a mulched system should not exceed 1 ft/sec, and flow in a grassed system should not exceed 3 ft/sec. Flows can be greater (up to 14 ft/sec) with the use of reinforced turf matting and will depend on the matting selected. Flow rate can be calculated using methods in TxDOT (2011).

Figure B-17: *An offline bioswale (left) along the road right of way with excess flow bypass along the gutter line at the Doseum, San Antonio Texas; and an online bioswale (right) with an overflow outlet structure along a roadway and parking lot that is part of a treatment train with an adjacent detention basin, San Antonio, Texas.*

Step 8. Select Mulch and Vegetation

Both mulch and vegetation are critical design components of bioswales from hydrologic, water quality, and aesthetic perspectives. The following subsections provide general guidance for mulch and vegetation.

Mulch
Bioswales intended to be mulched must be covered with mulch when constructed and annually replaced to maintain adequate mulch depth. Mulch can help sustain nutrient levels, suppress weeds, and maintain infiltrative capacity. Mulch should meet the specifications provided in the bioretention section.

APPENDIX B: **Additional Bioswale Design Guidance**

Vegetation

One advantage of a bioswale, similar to bioretention areas, is that they can be used for the dual purpose of stormwater treatment and landscaping or be integrated into the existing landscape. For bioswales to function properly as stormwater treatment and blend into the landscaping, vegetation selection is crucial. Appropriate vegetation will have the following characteristics:

1 Plant materials must be tolerant of summer drought, ponding fluctuations, and saturated soil conditions for 10 to 48 hours.

2 It is recommended that a minimum of three tree, three shrub, and three herbaceous groundcover species be incorporated to protect against facility failure from disease and insect infestations of a single species. Plant rooting depths must not damage the underdrain, if present. Slotted or perforated underdrain pipe must be more than 5 feet from tree locations (if space allows).

3 Native plant species or adapted noninvasive cultivars that do not require chemical inputs are recommended to be used to the maximum extent practicable.

4 Shade trees should be free of branches for the bottom 1/3 of their total height and lines of sight must be maintained when planting along roadways.

5 A list of native plants appropriate for San Antonio is in Appendix E.

Endless options for vegetation arrangement are available, and the one chosen will most likely depend on the landscaping of the area around the bioswale.

Encroachment of turf grasses presents a long-term maintenance challenge. The vegetation plan should be designed in order to delay encroachment of problem grass species in the bioswale feature and facilitate maintenance activities to prevent or remove growth of unwanted grass within the bioswale. In order to keep grass out of planting areas use all of the following methods:

1 Isolate planting areas. Do not plant Bermuda in or near planting areas. Where applicable plant less aggressive grass species.

2 In areas already bordered by Bermuda grass:

   a Install edging at least 8" below the surface.
   b Install edging at least 2" above the surface.
   c Install geotextile weed barrier at least 18" on planting side of edging.
   d Cover geotextile with a minimum of 4" or inorganic mulch

BIOSWALE REFERENCES

Barrett, M.E., M. Limouzin, and D.F. Lawler. 2012. Effects of media and plant selection on biofiltration performance Journal of Environmental Engineering 139(4):462-470.

Brown, R.A. and W.F. Hunt. 2011. Underdrain configuration to enhance bioretention exfiltration to reduce pollutant loads. Journal of Environmental Engineering 137(11):1082–1091.

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Li, M.-H., C.Y. Sung, M.H. Kim, and K.-H. Chu. 2010. Bioretention for Stormwater Quality Improvements in Texas: Pilot Experiments. Texas A&M University in cooperation with Texas Department of Transportation and the Federal Highway Administration.

TxDOT (Texas Department of Transportation). 2011. Chapter 13, Section 2. Soil Erosion Control Considerations. Hydraulic Design Manual. Austin, TX.

Notes:

APPENDIX B: **Additional Permeable Pavement Design Guidance**

Permeable Pavement

Because of the wide variability in design specifications and available resources, standard design guidance for all pavement elements may not be provided herein. For more design guidance, contact a qualified professional with experience in implementing permeable pavement.

Figure B-18: *Permeable pavement parking stalls at the Oaks at University Business Park, San Antonio, Texas. Source: Bender Wells Clark Design*

PERMEABLE PAVEMENT DESIGN

The design of a permeable pavement system follows a nine-step process, as described in Table B-6.

TABLE B-6. PERMEABLE PAVEMENT ITERATIVE DESIGN STEP PROCESS

Design step		Design component/ consideration	General guidance
1	**Determine BMP Size (pg 189)**	Use Appendix J	
2	**BMP Siting (pg 189)**	Based on available space, incorporate into parking lots (solely parking stalls or parking stalls and driving lanes), parking lanes along roadways, pedestrian sidewalks and plazas, and fire access roads. If outside the Edwards Aquifer Contributing Zone or if the system will be lined with an impermeable liner, a maximum total drainage area to permeable pavement area ratio of 2:1 is recommended.	

APPENDIX B: **Additional Permeable Pavement Design Guidance**

TABLE B-6. PERMEABLE PAVEMENT ITERATIVE DESIGN STEP PROCESS (CONT.)

3	**Select Permeable Pavement Surface Course (pg 190)**	Surface course type	Pervious concrete, porous asphalt, and permeable interlocking concrete pavers (PICP) are the preferred types of permeable pavement because detailed industry standards and certified installers are available. Concrete grid pavers and plastic grid systems also have useful applications.
4	**Determine BMP Function and Configuration (pg 195)**	Impermeable liner	If non-infiltrating (per geotechnical investigation), use impermeable clay liner, geomembrane, or concrete (as described in Appendix A,Common Design Elements).
		Lateral hydraulic restriction layers	Use concrete or geomembrane to restrict lateral flows to adjacent subgrades, foundations, or utilities.
		Underdrain (required if subsoil infiltration rate < 0-0.5 in/hour)	Schedule 40 PVC pipe with perforations (slots or holes) every 6 inches. 4-inch diameter lateral pipes should join a 6-inch collector pipe, which conveys drainage to the downstream storm network. Provide orifice at underdrain outlet sized to release water quality volume over 2-5 days. See Appendix A, Common Design Elements.
		Internal water storage (IWS)	If using underdrain and infiltration is feasible, elevate the outlet to create a sump to enhance infiltration and treatment.
		No underdrain	If design is fully infiltrating, ensure that subgrade compaction is minimized.
		Observation wells	Provided capped observation wells to monitor drawdown.
		Subgrade slope and Geotextile	Subgrade slope should be 1.5% - 5%. Localized depressions should be prohibited. Baffles should be used to ensure water quality volume is retained. Geotextile should be used along perimeter of cut to prevent soil from laterally entering the aggregate voids.
5	**Design the Profile (pg 197)**	Temporary surface ponding depth (Edwards Aquifer Zones)	Surface ponding should be provided (by curb and gutter) to capture the design storm in the event that the permeable pavement surface clogs.
		Specify sand/soil filter layer	• With underdrains: min. 3-inch layer of ASTM C-33 washed sand above gravel of underdrain drainage layer. A 2-inch layer of choking stone between sand and gravel might be needed. • No underdrains: min. 12-inch subsoil (see Appendix A, Common Design Elements)
		Calculate surface area and reservoir depth	Water quality volume should be fully stored within the aggregate base layers below the surface course. Base layer should be washed ASTM No. 57 stone (washed ASTM No. 2 may be used as a subbase layer for additional storage).
		Structural design	A pavement structural analysis should be completed by a qualified and licensed professional.
6	**Design for Safe Bypass/ Conveyance of Larger Storms (pg 198)**	Large storm routing	• Poured in place systems (pervious concrete or porous asphalt): system can overflow internally or on the surface • Modular/Paver-type systems (PICP): internal overflow is required to prevent upflow and transport of bedding course.

TABLE B-6. PERMEABLE PAVEMENT ITERATIVE DESIGN STEP PROCESS (CONT.)

7	**Design Edge Restraints and Transitions (pg 198)**	Transition strip	Provide a concrete transition strip between any permeable and impermeable surfaces and around the perimeter of PICP installations
8	**Design Signage (pg 199)**	Signage should prohibit activities that cause premature clogging and indicate to pedestrians and maintenance staff that the surface is intended to be permeable	
9	**Design for Multi-Use Benefits (Appendix C)**	Provide educational signage, enhanced pavement colors, or stormwater reuse systems.	

Step 1. Determine BMP Size

Permeable pavement must be sized to fully treat the desired or required design storm volume. Aggregate pore space provides capacity for the design storm volume retention. Appendix J outlines methods for determining design runoff depths associated with a range of annual treatment efficiencies. Once the design runoff depth is determined (on the basis of the desired level of treatment), a runoff volume can be determined for the contributing watershed using this depth and the methods outlined in Appendix J, San Antonio Unified Development Code, or San Antonio River Basin Regional Modeling Standards for Hydrology and Hydraulic Modeling.

Peak flow rates for the design storm should also be calculated, using the methods outlined in Appendix J so that the inlet and pretreatment can be accordingly sized and flow attenuation can be considered.

Step 2. BMP Siting

Permeable pavement is a highly versatile stormwater BMP because it can effectively reduce pollutants and can be integrated into site plans with various configurations and components. Stormwater treatment should be considered as an integral component and incorporated in the site design and layout from conception. Many times, determining how permeable pavement will be included in the site design is a critical and required first step. How the water is routed to the permeable pavement and the available space will be key components in determining how the permeable pavement is configured.

Permeable pavement is typically designed to treat stormwater that falls on the actual pavement surface area and has been used at commercial, institutional, and residential sites in spaces that are traditionally impervious. Outside the Edwards Aquifer Contributing Zone, Transition Zone, and Recharge Zone, runoff from adjacent surfaces is allowed but must be limited to runoff from stabilized areas with very little sediment yield. A maximum total drainage area to permeable pavement area ratio of 2:1 is recommended.

Runoff from pervious surfaces or high-sediment areas should be prevented, and permeable pavement should not be installed in areas prone to flooding with sediment-laden water (e.g., floodplains) because excessive sediment can prematurely clog the pores. Overhanging trees should also be avoided to reduce the deposition of detritus on the pavement surface, which can be ground into joints and pores if not routinely removed.

Because permeable pavements are intended for use in fully stabilized catchments, pretreatment measures are generally not required. An exception is presented if runoff is contributed to the permeable pavement from adjacent rooftops; leaves and debris should be screened before discharge onto the pavement surface.

Following is a list of settings in which permeable pavement can be incorporated to meet more than one project-level or watershed-scale objective:

- Parking lots
- Parking lanes in rights-of-way along roads
- Sidewalks and pedestrian plazas
- Access roads and shoulders

In addition, permeable pavement areas can be combined with other BMPs to form a treatment train that provides enhanced water quality treatment and reductions in runoff volume and rate. For example, runoff can flow from a roadway to the permeable pavement section and overflow to a bioretention area as shown in Figure B-17. Both facilities can be reduced in size according to demonstrated performance for meeting the stormwater runoff requirements and addressing targeted pollutants of concern.

Figure B-19: Permeable pavement and bioretention treatment train

Step 3. Select Permeable Pavement Surface Course

Traditional pavement sections are typically designed by geotechnical engineers and indicated on civil construction plans. Design considerations include number of vehicle trips and turning movements, weight of vehicles, pavement strength an long term integrity. Engage civil and geotechnical engineers during site planning and prior to selecting pavement types.

Several types of permeable pavement are available: pervious concrete, porous asphalt, permeable interlocking concrete pavers, concrete grid pavers, and plastic grid systems, among others. Each type of pavement has advantages and disadvantages, so factors such as cost, pavement use (parking area, driveway, sidewalk, fire lane, and such) and maintenance requirements should be considered on a site-by-site basis. When applicable, follow manufacturers' instructions to ensure a successful implementation.

PICPs, grid pavers, and plastic grid systems are generally better suited to smaller areas because of the labor involved with installation; however, many contractors now employ mechanical placement technologies to expedite the installation of pavers making larger parking areas more feasible. PICP and block pavers can be used for driveways, entryways, walkways, or terraces to achieve a more traditional, formal appearance.

More detailed information for the various types of permeable pavement follows.

Porous Asphalt

Porous asphalt pavement consists of fine- and course-aggregate stone bound by a bituminous-based binder. The amount of fine aggregate is reduced to allow for a larger void space of typically 15 to 20 percent. Because porous asphalt is a hot-mixed pavement, binder temperature performance grade (PG) should be specified on the basis of the anticipated climate to prevent premature failure (melting and sealing) under extreme heat conditions. PG 76-22 liquid asphalt binder is recommended (CAPA n.d.; TxDOT 2004). Thickness of the asphalt depends on the traffic load but usually ranges from 3 to 7 inches. A required underlying base course, typically a washed No. 57 stone, increases storage, and adds strength because porous asphalt is designed to be a flexible pavement (Ferguson 2005). A 1- to 2-inch layer of choker course (single-sized crushed aggregate, one-half inch) is typically required to stabilize the surface. Porous asphalt with an aggregate reservoir layer is currently not approved by TCEQ to meet the TSS reduction criteria in the Edwards Aquifer rules (TCEQ 2012).

APPENDIX B: **Additional Permeable Pavement Design Guidance**

Porous asphalt can also be installed directly over existing concrete to form a permeable friction course (PFC) overlay. PFCs do not provide the volume storage capacity of porous asphalt systems with reservoir layers, but they can provide excellent water quality improvements in addition to enhanced driver safety (reduced hydroplaning, improved stopping distance, reduced spray, and improved visibility), noise reduction, and improved ride quality (Rand 2006; NCHRP 2009; Eck et al. 2012). PFC overlays have been used with great success on San Antonio and other Texas highways and have been approved to meet the TSS removal rules in the Edwards Aquifer (Rand 2006; TCEQ 2012).

The properties of porous asphalt depend on the materials used and the compaction procedures. Specified mix design should be in accordance with the National Asphalt Pavement Association (NAPA) Porous Asphalt Pavements for Stormwater Management (NAPA 2008). General guidelines are provided below.

Permeability. Typical flow rates for water through porous asphalt range from 150 to 300 in/hour (Roseen and Ballestero 2008). Those values exceed the typical permeability of subsurface soils, so the soils would be the limiting factor.

Figure B-20: *Example of porous asphalt, Mission Library, San Antonio, Texas.*

Durability: As with all BMPs, the longevity of porous asphalt (Figure B-20) is highly dependent on proper maintenance. Many porous asphalt parking lots have been in service for more than 20 years.

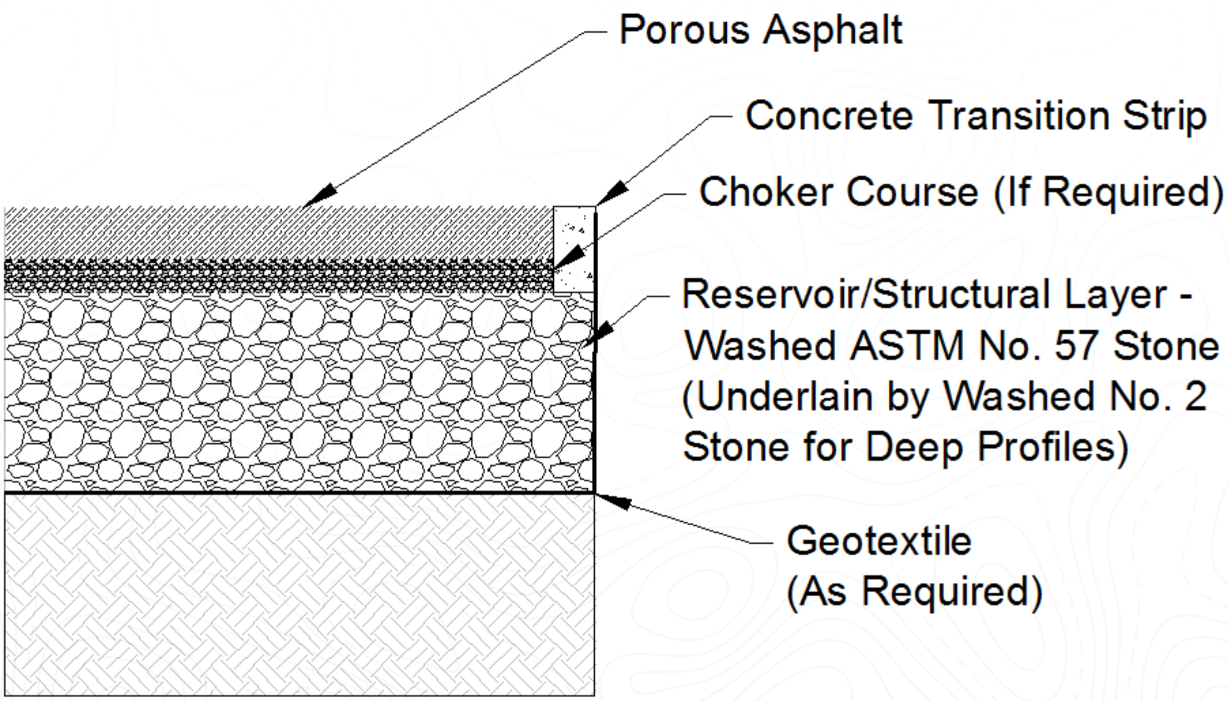


Figure B-21: *Typical pervious asphalt cross section*

APPENDIX B: **Additional Permeable Pavement Design Guidance**

Pervious concrete is a mixture of Portland cement, fly ash, washed gravel, and water. The water-to-cementitious material ratio is typically 0.35–0.45 to 1 such that the mixture displays a wet metallic sheen without the paste flowing from the aggregate (NRMCA 2004; Barrett 2005). Unlike traditional installations of concrete, permeable concrete usually contains a void content of 15 to 25 percent, which allows water to infiltrate directly through the pavement surface to the subsurface. A fine, washed gravel, less than 13 mm in size (No. 8 or 89 stone), is added to the concrete mixture to increase the void space (GCPA 2006). An admixture improves the bonding and strength of the pavements. The pavements are typically laid with a 4- to 8-inch (10 to 20 cm) thickness over a gravel reservoir (depth varies according to water volume capture requirements), typically a washed No. 57 stone. Pervious concrete is a rigid pavement and therefore does not require an aggregate base course for structural support. Pervious concrete will typically exhibit a coarser surface texture than impervious concrete but is ADA compliant. A typical pervious concrete is shown in Figure B-22.

Figure B-22: *Example of pervious concrete, Kinston, North Carolina. Source: North Carolina State University Department of Biological and Agricultural Engineering*

The properties of pervious concrete vary with design and depend on the materials used and the compaction procedures. Design mix should conform to the latest version of the American Concrete Institute's ACI 522.1-08 Specification for Pervious Concrete Pavements. General guidelines are provided below.

Permeability	Typical flow rates through well-maintained pervious concrete average greater than 1,500 in/hour (Bean et al. 2007).
Compressive Strength	Pervious concretes can develop compressive strengths in the range of 500 to 4,000 pounds per square inch (psi)—suitable for a wide range of applications.
Flexural Strength	Flexural strength of pervious concrete ranges between 150 and 550 psi. Pervious concrete does not typically incorporate rebar.
Shrinkage	Drying shrinkage of pervious concrete is faster but much less than that experienced with conventional concrete. Pervious concretes should be constructed with control joints to regulate cracking. In general, joints should be cut one-quarter of the pervious concrete thickness, be placed a maximum of 20 feet on centers (15 feet is typical) perpendicular to the curb, and should form square panels.
Abrasion resistance	Because of the rougher surface texture and open structure of pervious concrete, abrasion and raveling of aggregate particles can be a problem. Surface raveling in new pervious concrete can occur when rocks loosely bound to the surface break free under traffic loads. Such raveling is considerably reduced after the first few weeks. Raveling can be reduced by carefully covering the pervious concrete during curing to prevent the surface from drying prematurely. Polypropylene fibers and/or latex can also be added to reduce abrasion resistance (Dong et al. 2010).

Barrett, M.E. 2005. Complying with the Edwards Aquifer Rules. Technical Guidance on Best Management Practices. RG-348. Prepared for Texas Commission on Environmental Quality, Field Operations Division, Austin, TX.

APPENDIX B: **Additional Permeable Pavement Design Guidance**

Permeable Interlocking Concrete Pavements
PICPs are available in many different shapes and sizes. When lain, the blocks form patterns that create openings through which rainfall can infiltrate. Orientation of rectangular pavers is important for structural purposes—herringbone patterns tend to provide the most efficient structural design, especially where vehicle stopping and turning are expected. ASTM C936-13 specifications state that the pavers be at least 2.36 inches (60 mm) thick with a compressive strength of 55 MPa (8,000 psi) or greater. Typical installations consist of the pavers and crushed aggregate fill, a 1-5- to 3.0-inch (38 to 76 mm No. 8) fine-aggregate bedding layer, and an aggregate base-course, typically a washed No. 57 stone, storage layer (Smith 2011). If greater storage is required, a reservoir subbase layer of No. 2 stone can be included. More details on PICP can be found in Smith (2011). An example PICP profile is shown in Figure B-23.

Unlike permeable concrete and porous asphalt, PICP is not subject to time and temperature limitations in installation. Interlocking Concrete Pavement Institute (ICPI) standards should be followed during design and construction. Below are listed general specification guidelines.

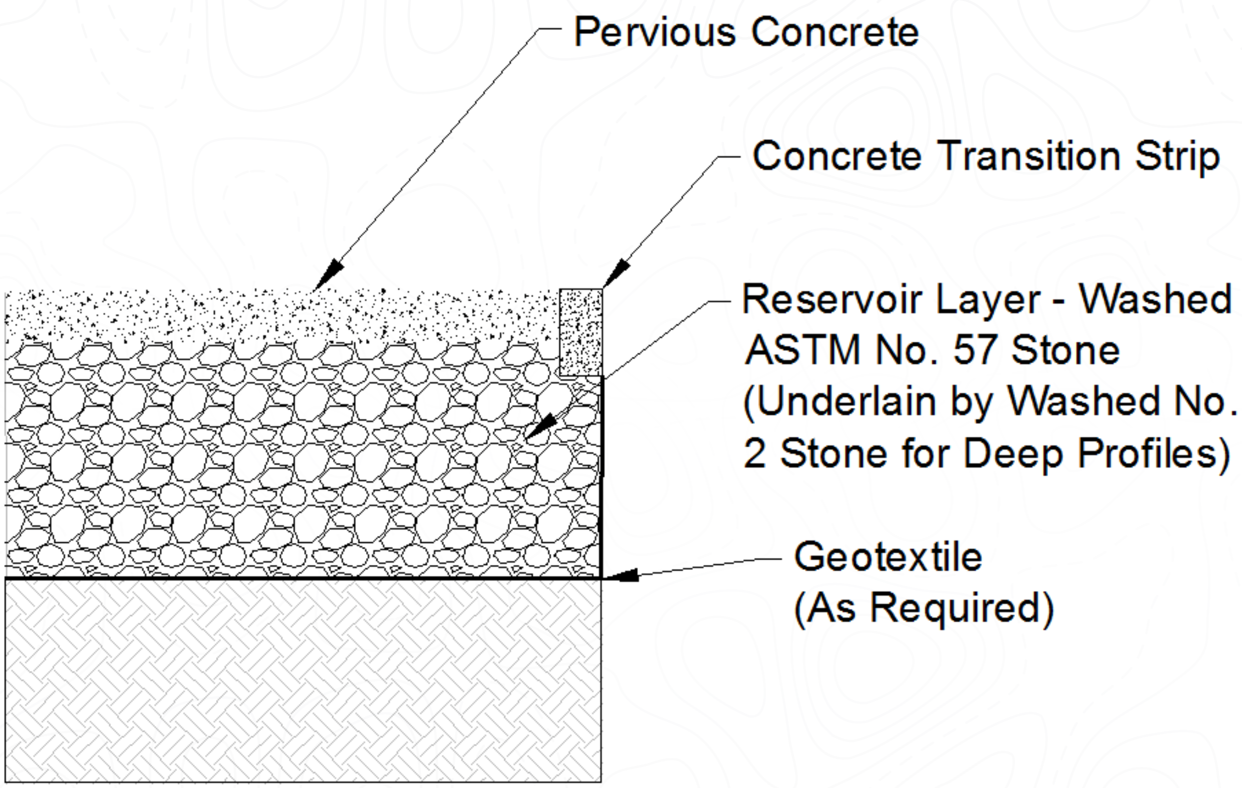
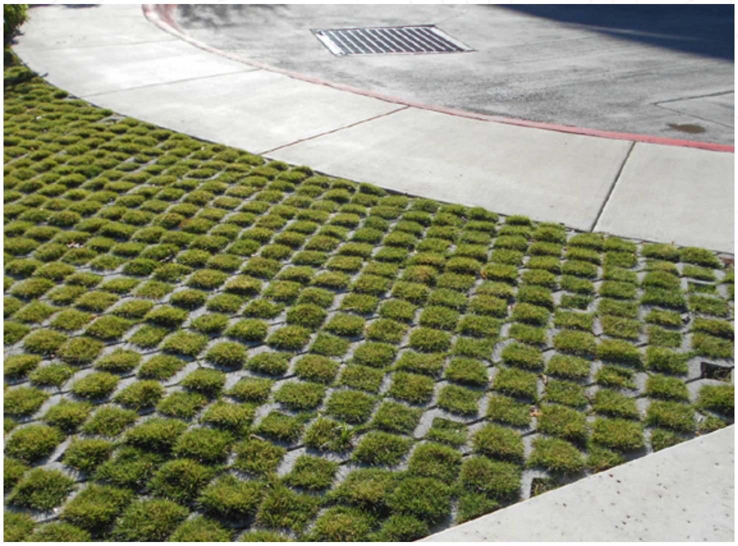


Figure B-23: *Typical PICP cross section*

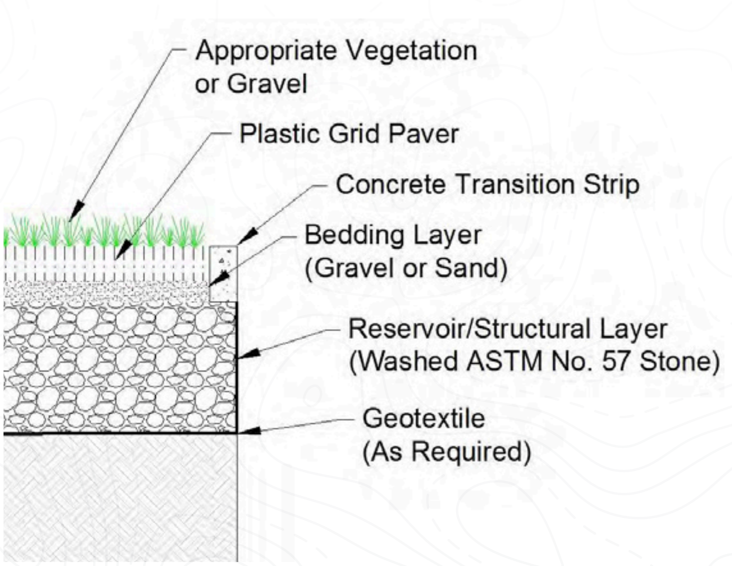
Permeability	Lifetime infiltration rates on maintained PICP surfaces range from 14 to 4,000 in/hour depending on the joint filling material (Borgwardt 2006; Bean et al. 2007).
Compressive Strength	PICP has a minimum compressive strength of 8,000 psi (55 MPa).
Durability	Regularly maintained permeable pavement systems can last more than 20 years and provide an initial high level of surface infiltration even as the surface takes in moderate amounts of sediment.



Concrete Grid Pavers
Concrete Grid Paver (CGP) systems conform to ASTM C 1319, Standard Specification for Concrete Grid Paving Units which describes paver properties and specifications. CGP units have a minimum thickness of 3.125 inches (80 mm) thick with a maximum 24 × 24 inch (60 × 60 cm) dimension. The percentage of open area ranges from 20 to 50 percent and can contain topsoil and grass, sand, or aggregate in the void space (Figure B-24). The minimum average compressive strength of CGP units can be no less than 35 MPa (5,000 psi). A typical installation consists of grid pavers with fill media, 1–1-5 inches (25 to 38 mm) of bedding sand or No. 8 gravel, gravel base course typically consisting of washed No. 57 stone, and a loosely compacted soil subgrade (ICPI 2004). If sand is used, a geotextile should be used between the sand course and the reservoir media to prevent the sand from migrating into the stone media.

Figure B-24: *Example of Concrete Grid Pavers (CGP) planted with turf grass that serves as an emergency vehicle access and landscape feature at the River House, San Antonio, Texas.*

The ICPI provides design standards for CGP design and installation, but application of CGPs is typically limited to very low traffic areas (such as emergency vehicle access roads or event overflow parking). This limitation is because of differential settling and subsequent rocking of pavers that can occur because CGPs (unlike PICP) do not interlock.



Plastic Grid Systems
Plastic grid systems, also called geocells, turf pavers, or turf reinforcing grids, consist of flexible-plastic, interlocking units that allow for infiltration through large gaps filled with gravel or topsoil planted with turf grass. Similar to PICP, a 1–2 inch sand bedding layer and gravel base course are often added to increase infiltration and storage. The empty grids are typically 90 to 98 percent open space, so void space depends on the fill media (Ferguson 2005). To date, no uniform standards exist; however, one product specification defines the typical load-bearing capacity of empty grids at approximately 13.8 MPa (2,000 psi) (Invisible Structures 2001). That value increases up to 38 MPa (5,500 psi) when filled with various materials (Invisible Structures 2001). If sand is used, a geotextile should be used between the sand course and the reservoir media to prevent the sand from migrating into the stone media. Plastic grid systems are currently not approved by TCEQ to meet the TSS reduction criteria in the Edwards Aquifer rules. A typical plastic grid system profile is shown in Figure B-25.

Figure B-25: *Typical plastic grid system cross section*

Plastic grids (Figure B-26) provide structural support but are generally limited to very low traffic areas such as emergency vehicle access lanes and event overflow parking. They are usually planted with grass. Several companies manufacture plastic grid systems.

Load bearing capacity Plastic grid systems have a load-bearing capacity up to 6,700 psi when filled (CONTECH 2011).

Durability Because plastic grid systems are typically manufactured from high-density polyethylene (HDPE), long service lives, up to 50 years, can be expected with proper maintenance.

Figure B-26: *Example of plastic grid system filled with rock that serves as a roadway for residential units, San Antonio, Texas.*

Step 4. Determine BMP Function and Configuration

The hydrologic and water quality performance of permeable pavement is largely determined by the drainage configuration. Furthermore, some areas might not warrant infiltration. The following design steps can be used to determine the drainage configuration design.

Perform Geotechnical Investigation
Once the appropriate surface course has been selected and discussed with the property owner, the in situ soils must be tested before the system can be sized. Performing soil tests during the conceptual and preliminary design phases will ensure that the proposed permeable pavement system is optimized to actual site conditions and to prevent costly change orders resulting from poorly estimated soil parameters.

A geotechnical investigation should be performed by a licensed soil scientist or geotechnical engineer. All soil testing should be performed at the depth of the initially proposed subgrade because this is the soil strata where infiltration might occur. If a detention (non-infiltrating system) is proposed, soil tests must still be performed to determine structural requirements and to identify the elevation of the seasonal high water table. For details on geotechnical analyses, see Appendix A, Common Design Elements.

Determine if Underdrains and Impermeable Liners are Needed
On the basis of the infiltration rate measured in the previous step, the drawdown time of the system at full capacity should be calculated. If the infiltration rate of the soils on which the permeable pavement area will be installed is less than 0-0.5 in/hour, underdrains will be required (as described in Table B-7). The underdrains can be embedded in the aggregate reservoir layer or in a gravel trench below the reservoir layer, as shown in Figure B-275. For information on designing an underdrain system, see Appendix A, Common Design Elements. IWS should be included in all infiltrating systems to enhance infiltration (Wardynski et al. 2013). The elevation of the upturned underdrain outlet dictates the volume of water retained in the profile, which should be greater than or equal to the water quality volume (as determined in Step 5). An example permeable pavement profile containing IWS is provided in Figure B-28.

APPENDIX B: **Additional Permeable Pavement Design Guidance**

If infiltration is disallowed, the system should be lined with a hydraulic restrictive layer. Factors prescribing an impermeable liner are provided in Table B-7. Non-infiltrating systems are also known as detention systems and can be designed similar to other detention structures. Outflow should be regulated in accordance with water quality (releasing water over the course of 2 to 5 days) and flood control requirements for detention structures (discussed in Step 6).

TABLE B-7. DECISION TABLE FOR DETERMINING UNDERDRAIN & IMPERMEABLE LINER REQUIREMENTS

Impermeable liners must be used if...	Underdrains must be used if...
• Site is in Edwards Aquifer Recharge Zone or Transition Zone • Contaminated soil is expected or present • Karst geology presents risk of sinkhole formation • Runoff may unintentionally be received from a stormwater hotspot • Site is within 100 feet of a water supply well or septic drain field • Site is within 10 feet of a structure/foundation • Infiltrated water may interfere with utilities	• An impermeable liner is needed • Infiltration rate of underlying soils is less than 0-0.5 in/hour • Site is within 50 feet of a steep, sensitive slope

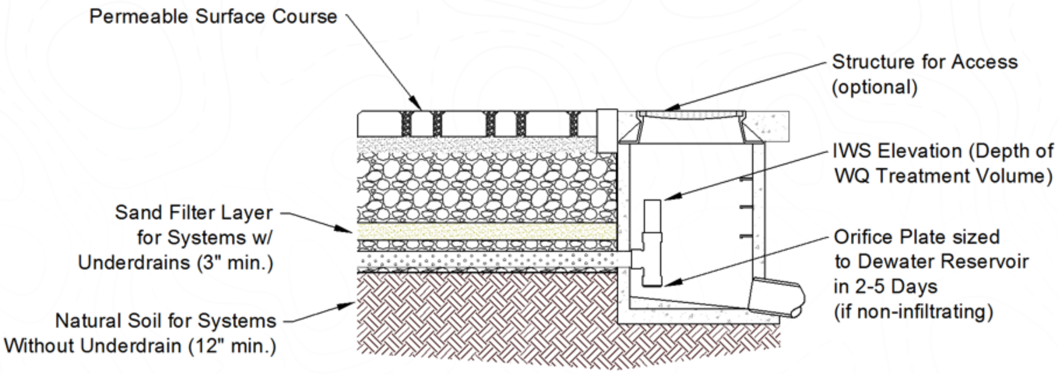


Figure B-27 (left): *Permeable pavement showing example cross section with trenched underdrain at Alamo Heights Fire Station, Alamo Heights, Texas. Source: Tetra Tech*
Figure B-28 (right): *Example permeable pavement profile featuring IWS*

Figure B-29: *Observation well installed in permeable pavement*

Observation Wells

Design drawings should specify installation of observation wells to monitor the drawdown rate of permeable pavement reservoir layers. Wells should be constructed of perforated PVC pipe (4-inch diameter or greater) and should be designed to prevent damage from vehicular traffic. If necessary, observation wells can be installed at an angle and daylight in adjacent landscape areas (as long as the well extends the full depth of the reservoir layer). Wells should be securely sealed with watertight caps. Figure B-29 provides examples of observation wells installed in permeable pavement applications.

Design Subgrade Slope and Specify Geotextile

The subgrade slope should not exceed 0.5 percent to ensure that the design volume is captured and evenly distributed and to maintain structural integrity. Baffles can be installed along the subgrade to provide grade control if necessary. In fully lined systems, a drawdown orifice should be provided in each baffle to allow dewatering between storm events.

A geotextile should be placed beneath the reservoir media and along the perimeter of the cut in all infiltrating systems. A needled, non-woven, polypropylene geotextile conforming to the general guidelines in Table B-8 should be specified. It is important to line the entire trench area, including the sides, with a geotextile before placing the aggregate. The geotextile serves an important function by inhibiting soil from migrating into the reservoir layer and reducing storage capacity.

Step 5. Design the Profile

The permeable pavement profile must be designed to capture the water quality treatment volume and filter it through soil or a sand filter layer. In fully lined (non-infiltrating systems), the treatment volume should ideally be detained for 2 to 5 days (orifice sizing equations are in the Stormwater Wetlands section). Additionally, the profile must provide structural support for the anticipated vehicular loading.

Temporary Surface Ponding Depth (Edwards Aquifer Zones)
When permeable pavement is used in the Edwards Aquifer protection zones, surface ponding must be provided (by curb and gutter) such that the design storm volume will be retained onsite if the permeable pavement surface clogs (Barrett 2005). Curb edging and driveways should be elevated such that the design water quality volume ponds on the surface and does not flow offsite. This is not typically a requirement outside the Edwards Aquifer protection zones.

Specify Sand/Soil Filter Layer
Percolating runoff through native soils is the most effective way to improve water quality. When no underdrains are required (when subsoil infiltration rates greater than 0-0.5 in/hour), a minimum of 12 inches of native soil should be provided at the subgrade to filter captured stormwater before infiltration (for soil general guidance, see Appendix A, Common Design Elements). If underdrains are used, or if subsoils are not suitable for stormwater filtration, a minimum of 3 inches of ASTM C-33 washed sand should be included above the aggregate of the underdrain drainage layer. A layer of choking stone might be needed between the sand filter layer and the gravel drainage layer, as discussed in Appendix A, Common Design Elements.

Calculate Surface Area and Reservoir Media Depth
The aggregate base course should be designed to store at a minimum the water quality treatment volume determined in Appendix J. For infiltrating systems, this volume should be retained in the profile using IWS (as described in step 4). The stone aggregate used should be washed, angular, crushed stone, 0.75 to 1 inch in diameter with a void space of about 40 percent (No. 57 stone). ASTM No. 2 stone may be used as a subbase layer below the base course for additional storage. Aggregate contaminated with soil and typical crusher run stone should not be used because those materials will clog the pores at the bottom of the pavement.

If the area of permeable pavement is known, the following equation can be used to determine the depth of storage layer (aggregate base course) needed to capture the water quality treatment volume:

[Equation B-3-1]

$$d = \frac{V}{An}$$

where:
- ***d*** = aggregate layer depth (ft)
- ***V*** = water quality volume (ft^3)
- ***A*** = surface area (square ft)
- ***n*** = porosity (use actual laboratory measured porosity of material)

Structural Design Requirements
If permeable pavement will be used in a parking lot or other setting that involves vehicles, the pavement surface must be able to support the maximum anticipated traffic load. The structural design process will vary according to the type of pavement selected, and the manufacturer's specific Recommendations should be followed. Engage civil and geotechnical engineers during this process. The thickness of the permeable pavement and reservoir layer must be sized to support structural loads and to temporarily store the design storm volume (e.g., the water quality, channel protection, and flood-

control volumes). On most new development and redevelopment sites, the structural support requirements will dictate the depth of the underlying stone reservoir.

The structural design of permeable pavements involves considering four main site elements:

- Total traffic
- Vehicle weight
- In situ soil strength
- Environmental elements
- Bedding and reservoir layer design.

The resulting structural requirements can include the thickness of the pavement, filter, and reservoir layer. Designers should note that if the underlying soils have a low California Bearing Ratio (less than 4 percent), the soil might need to be compacted to at least 95 percent of the Standard Proctor Density, which generally rules out their use for infiltration.

Designers should determine structural design requirements by consulting transportation design guidance sources, such as the most current edition of the following:

- AASHTO Guide for Design of Pavement Structures.
- AASHTO Supplement to the Guide for Design of Pavement Structures.
- ASCE Design Standard 68-18 Permeable Interlocking Concrete Pavement.

Step 6. Design for Safe Bypass/Conveyance of Large Storms

Permeable pavement systems, as with any other stormwater BMP, must be designed to safely route runoff in excess of the intended design flow. The method of large-storm routing is largely site specific and depends on the type of permeable surface course. When poured in place, surface courses are used (pervious concrete or porous asphalt), volume in excess of the system storage capacity can be allowed to bubble up through the profile and run off the site safely as surface flow. Catch basins or slot drains could be installed around the perimeter of the permeable pavement to drain any overflow; inlets can be specified slightly above the elevation of the finished surface to allow some surface ponding, if allowable.

Modular paving systems (PICP, CGP, or plastic grid systems) should not be designed to overflow in this manner, however, because upflowing water could dislodge and carry away aggregate from the bedding course. When surface overflow is not a feasible or preferred option, the system can be designed to (1) completely store the 25-year storm volume in the aggregate reservoir and exfiltrate into underlying soils, (2) convey larger storms safely through the system using underdrains (equipped with orifices, if required), or (3) use other internal controls to allow bypass of larger storms. Large storm routing can be designed to satisfy detention requirements, per local requirements.

Step 7. Design Edge Restraints and Transitions

Providing separation between permeable pavements and adjacent impermeable surfaces serves multiple purposes, including the following:

1. Clearly identifying for maintenance personnel the transition between permeable and impermeable surfaces
2. Restraining modular (block) pavers and porous asphalt to prevent lateral shifting or unraveling of edges
3. Creating a hydraulic restriction layer to prevent lateral seepage of runoff below adjacent pavements and structures
4. Delineating parking zones with clean, aesthetically pleasing lines.

Restraints for flexible pavements are typically composed of standard concrete curbs (elevated or at grade, depending on application) or specially designed monolithic concrete walls. At intersections between permeable and impermeable surfaces, a hydraulic restriction layer (typically a geomembrane) is installed along the entire length of the cut and at least 2 feet laterally along the subgrade and under the impermeable surface. Figure B-30 shows an example of an edge restraint.

APPENDIX B: **Additional Permeable Pavement Design Guidance**

Figure B-30: A 1-foot concrete transition strip is used as an edge restraint between PICP and impermeable asphalt, Floresville, Texas.

Step 8. Design Signage

It is good practice to specify signage on engineering plans; signage educates the public and identifies permeable pavements to maintenance personnel. Prohibited practices, such as stockpiling soils or mulch, should be clearly displayed to protect permeable pavements from premature clogging. Signage will also prevent poured in place permeable pavements from being mistaken as impermeable and then paved over during repair.

PERMEABLE PAVEMENT REFERENCES

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APPENDIX B: **Additional Planter Box Design Guidance**

Planter Boxes

Figure B-31: *Planter box inlet configuration, San Diego, California. Source: Tetra Tech*

PLANTER BOX DESIGN

Planter boxes provide similar function to a bioretention area but can be used to provide treatment where infiltration is not possible because of geotechnical limitations or vertical constraints. The design process is similar to that of lined (non-infiltrating) bioretention cells with a few noted exceptions in Table B-8.

TABLE B-8. PLANTER BOX ITERATIVE DESIGN PROCESS

Design step		Design component/ consideration	General guidance
1	**Determine BMP Treatment Volume (pg 202)**	Use Appendix J	
2	**BMP Siting (pg 203)**	Based on available space, incorporate along the perimeter of buildings, along the roadway right-of-way, or near the outlet of a green roof or cistern.	
3	**Determine BMP Function and Configuration (pg 203)**	Impermeable liner	Planter boxes are typically contained in a concrete vault (as described in Appendix A, Common Design Elements)
		Underdrain (required)	Schedule 40 PVC pipe with perforations (slots or holes) every 6 inches. 4-inch diameter lateral pipes should join a 6-inch collector pipe, which conveys drainage to the downstream storm network. Provide cleanout ports/ observation wells for each underdrain pipe at spacing consistent with local regulations. See Appendix A, Common Design Elements
		Internal water storage (IWS)	With careful plant selection, the outlet can be slightly elevated to create a sump for additional moisture retention to promote plant survival and enhanced treatment. Top of IWS should be more than 18 inches below surface.

TABLE B-8. PLANTER BOX ITERATIVE DESIGN PROCESS (CONT.)

4	**Size the System (pg 203)**	Temporary ponding depth	6-18 inches (6-12 inches near schools or in residential areas); average ponding depth of 9 inches is recommended
		Soil media depth	2-4 feet (deeper for better pollutant removal, hydrologic benefits, and deeper rooting depths)
		Surface area	Find surface area required to store treatment volume in temporary ponding depth, soil media depth, and gravel drainage layer depth (media porosity ≈ 0.35 and gravel porosity ≈ 0.4).
5	**Specify Soil Media (pg 203)**	Composition and texture	85-88% sand, 8-12% fines, 2-5% plant-derived organic matter (animal wastes or by-products should never be applied)
		Permeability	1-6 in/hour infiltration rate (1-2 in/hour recommended)
		Chemical composition	Total phosphorus < 15 ppm, pH 6-8, CEC > 5 meq/100 g soil
		Drainage layer	Separate soil media from underdrain with 2 to 4 inches of washed sand, followed by 2 inches of choking stone (ASTM No. 8) over a 1-5 foot envelope of ASTM No. 57 stone.
6	**Design Inlet and Pretreatment (pg 203)**	Inlet	Provide stabilized inlets (see Appendix A, Common Design Elements).
		Pretreatment	Minimal pretreatment is required if receiving rooftop runoff; however, pretreatment recommendations provided in bioretention section should be followed if receiving surface runoff from paved areas.
7	**Select and Design Overflow/ Bypass Method (pg 203)**	Outlet configuration	Online: All runoff is routed through system—install an elevated overflow structure or weir at the elevation of maximum ponding Offline: Only treated volume is diverted to system—install a diversion structure or allow by-pass of high flows
		Peak flow mitigation	Provide additional detention storage and size an appropriate non-clogging orifice or weir to dewater detention volume
8	**Select Mulch and Vegetation (pg 203)**	Mulch	Dimensional chipped hardwood or triple shredded, well-aged hardwood mulch 3 inches deep.
		Vegetation	See Plant List (Appendix E).
9	**Design for Multi-Use Benefits (Appendix C)**	Include features to enhance habitat, aesthetics, public education, and shade.	

APPENDIX B: **Additional Planter Box Design Guidance**

Step 1. Determine BMP Treatment Volume

The planter box must be sized to fully capture the desired or required design storm volume and filter it through the soil media. Surface storage (in the ponding area) and soil pore space (in the plant rooting zone and the underlying media and gravel drainage layers) provide capacity for the design storm volume retention. The volume of water that must be treated is equal to the design storm volume and can be calculated using the information in Appendix J. Once the design runoff depth is determined (according to the desired level of treatment), a runoff volume can be determined for the contributing watershed using this depth and the methods outlined in Appendix J, San Antonio Unified Development Code, or San Antonio River Basin Regional Modeling Standards for Hydrology and Hydraulic Modeling.

Peak flow rates for the design storm should also be calculated, using the methods outlined in Appendix J, so that the inlet and pretreatment can be accordingly sized and flow attenuation can be considered.

Step 2. BMP Siting

Planter boxes, like bioretention areas, can be incorporated into the site design with various configurations and components. Unlike bioretention areas, planter boxes, because they are completely contained, can be included close to buildings and other structural foundations without affecting structural stability as long as underdrain outflow and overflow are routed in a safe direction. Planter boxes can be perched above grade on structures and/or be placed in series along a grade (tiered systems) to take advantage of vertical structures.

Step 3. Determine BMP Function and Configuration

Planter boxes have the same drainage requirements as bioretention but are typically hydraulically isolated from subsoils so underdrains are always required. IWS is not generally incorporated into planter box design unless a very shallow reservoir is provided. Care must be taken to select plants that can withstand saturated root zones if IWS is selected as a design option.

Step 4. Size the System

Planter boxes have the same sizing standards as a bioretention area.

Step 5. Specify Soil Media

Planter boxes must meet same soil media standards as a bioretention area.

Step 6. Design Inlet and Pretreatment

Inlets for a planter box must meet the same standards as inlets for bioretention area. Planter boxes can incorporate filter strips, forebays, and curb cuts if located along the right-of-way. Because of the ability to install planter boxes adjacent to structural foundations, a planter box inlet can also incorporate a downspout from an adjacent building. Pipe flow and downspouts can be stabilized using similar strategies for a curb cut using sod, if the flow rate is less than 3 cubic feet per second (cfs), stone, splash block, or other erosion protection material for higher flows. Alternatively, downspouts can be upturned to bubble up into the planter box in a diffuse manner.

Step 7. Select and Design Overflow/Bypass Method

Planter boxes can be designed as offline or online systems. Planter boxes designed in the right-of-way should be designed as offline systems. Because underdrains will be required for planter boxes, the overflow system will typically include a vertical riser in both online or offline systems. The vertical riser should be designed as described in the bioretention section.

Step 8. Select Mulch and Vegetation

The mulch and vegetation will be the same for a planter box as a bioretention area. Some consideration should be taken as to the location of the planter box when selecting the vegetation. Shade-tolerant plants should be selected if the planter box will be shaded by surrounding structures. Planter boxes in the right-of-way should be vegetated with low shrubs to comply with sight distance requirements.

Notes:

APPENDIX B: **Additional Green Roof Design Guidance**

Green Roofs

Figure B-32: Example of an intensive green roof at James Madison High School Agriscience Building, San Antonio, Texas. Source: Bender Wells Clark Design

GREEN ROOF DESIGN

Green roof design is largely dependent on structural constraints of the subject and desired goals. Table B-9 summarizes the nine basic design steps, which are described in more detail below. Additional design guidance can be found in Tolderlund (2010) and New York City Department of Environmental Protection and New York City Department of Buildings (2012).

TABLE B-9. GREEN ROOF ITERATIVE DESIGN STEP PROCESS

Design step		Design component/ consideration	General guidance
1	**Determine Green Roof Type (pg 206)**	Extensive	Shallow growing media (4–6 inches), small, drought-tolerant vegetation, no irrigation needed.
		Intensive	Growing media more than 6 inches, regular irrigation required deeper rooted vegetation. Contact qualified professional with experience designing intensive green roofs.
2	**Determine Green Roof Size (pg 206)**	Use Appendix J	
3	**Determine Structural Capacity of Roof (pg 207)**	Underlying roof deck and building structure	Evaluate proposed or existing building and roof structure to determine additional dead and live load capacity available to accommodate green roof installation.

APPENDIX B: **Additional Green Roof Design Guidance**

TABLE B-9. GREEN ROOF ITERATIVE DESIGN STEP PROCESS (CONT.)

4	**Specify Impermeable Liner and Root Barrier (pg 207)**	Roof liner	Select waterproof liner. Conventional roof waterproofing tar is typically sufficient but can be supplemented with waterproof geomembranes if desired.
		Root barrier	Select root barrier. Geomembranes used as waterproof liners can sometimes double as root barriers.
5	**Specify Drainage Layer (pg 207)**	Aggregate	Minimum 2 inches of clean washed No. 8 stone or alternative lightweight, high-porosity, inorganic or synthetic aggregate. Geotextile fabric should be installed between the media and the aggregate.
		Manufactured	Select drainage layer specified for green roof applications that incorporates minimum 0.75 inch of retention storage of rainfall. Geotextile fabric should be installed between the media and the drainage layer.
6	**Design Outlet Components (pg 207)**	Roof drains	Provide roof drains or scuppers consistent with local building code requirements. Surround outlets with minimum 12 inches of high-porosity drainage material (washed ASTM No. 57 stone or comparable).
7	**Specify Soil Media (pg 208)**	Depth	Minimum 4-inch depth (intensive green roofs)
		Content	Media should consist of a well-drained, high-porosity mix of primarily lightweight aggregate (preferred media is site specific, but expanded mineral materials are typically specified for intensive green roofs). pH = 6-5–8.0, CEC greater than 10 meq/100 g.
8	**Select Vegetation (pg 208)**	Low growing, drought-tolerant species	See Plant List (Appendix E).
9	**Design for Multi-Use Benefits (Appendix C)**	Site specific	Include features to enhance recreational opportunities, habitat, aesthetics, and energy savings.

Figure B-33: *Example of an extensive green roof, East Lansing, Michigan. Source: Tetra Tech*

Step 1. Determine Green Roof Type

Green roofs can be categorized into one of two basic types according to design goals, structural constraints, and funding: extensive and intensive. The following subsections describe each type of green roof.

Extensive Green Roofs

Green roofs with less than 6 inches of media and shallow-rooting, xeric vegetation are considered extensive (Figure B-33). These roofs require little or no irrigation and contribute lighter loads to rooftops than intensive green roofs. Vegetation is typically composed of small succulents like stonecrops *(Sedum spp.)* or other desert plants that can withstand extreme temperature and moisture fluctuations. In the semi-arid environment, extensive green roofs typically require drip irrigation during plant establishment and dryer summer months. Irrigation should be achieved using air conditioner condensate or harvested rainwater. If sufficient water is not available from these sources, deeper media with higher water holding capacity can be specified or an alternative BMP should be selected. Various manufactured systems are available on the market with modular trays and built-in drainage layers to simplify design and installation.

Intensive Green Roofs
When a green roof has more than 6 inches of media and features deeper-rooting plants, it is considered intensive (Figure B-32). Intensive green roofs can be installed where structural support can handle the extreme weight of deep, saturated soils and vegetation. Often intended to function as small rooftop parks or gardens, intensive green roofs can provide many amenities; however, park-like landscaping on a rooftop might require irrigation, so take care to select water-efficient plants, especially if limited air conditioner condensate is available (Bexar Regional Watershed Management will not support BMPs that require permanent irrigation systems). **Because of the wide variability in intensive green roof layout, media type and depth, irrigation demand and landscaping, it is not appropriate to explore the design process in this manual. For more design guidance, contact a qualified professional with experience in implementing intensive green roofs.**

Step 2. Determine Green Roof Size

Green roofs typically treat only direct rainfall, except for certain situations where runoff is generated from adjacent roof areas or where air conditioner condensate is captured. Design volume and flow rates can be determined using the methods in Appendix J.

Step 3. Determine Structural Capacity of Roof

Green roof design primarily depends on the excess load that can be applied to a rooftop. A qualified structural engineer should be consulted to determine the structural capacity of the roof in question to support additional dead and live load resulting from green roof installation. For new construction, the building designer might consider the additional roof load in selecting building structural components. In either scenario the dead and live roof loads from the green roof installation will depend on the specific green roof components and must be evaluated case by case. In general, extensive green roofs can be expected to exert a dead load (fully saturated) of 15 lb/square foot to 55 lb/square foot. Loading by intensive green roofs will widely vary based on soil depth and other components (Tolderlund 2010).

Step 4. Specify Impermeable Liner and Root Barrier

As with all roofs, a watertight barrier must be provided to prevent rainwater from infiltrating the underlying structure. Watertight tar surfaces (conventionally used for roof sealing) are usually sufficient impermeable liners, but additional plastic or rubber membranes can be placed over the tar for added protection. The liner should be resistant to heat, desiccation, and ultraviolet radiation. A root barrier should be specified and placed directly above the impermeable liner or alternatively above an optional insulation layer that can be placed directly on the liner.

Figure B-34: *Light-colored gravel delineates the no-planting zone for maintenance personnel, Raleigh, North Carolina. Source: City of Raleigh*

Step 5. Specify Drainage Layer

A drainage layer, also known as a drainage net or sheet drain, is necessary to convey excess rainwater to the roof drains. This layer will also maintain an aerobic root zone for plant health. Geotextile should be placed between the media and the drainage layer to prevent migration of media and act as a root barrier. Geotextiles containing chemicals that prevent root penetration can be used so root systems do not infiltrate and clog the drainage layer.

Step 6. Design Outlet Components

As with all roofs, components must be incorporated into the roof structure to allow free drainage of excess runoff from the rooftop and away from the building. For extensive green roof applications, drainage components can include internal roof drains or roof scuppers along roof perimeters. These components should be designed in accordance with local building codes. To ensure adequate conveyance of roof runoff from the drainage layer to the outlets, green roofs should be set back a minimum of 12 inches from roof drains. The area surrounding the roof drains should be filled with clean washed No. 57 stone or alternative high-porosity material. Placing light-colored stone buffer around the roof drains also delineates a no-plant zone for maintenance staff (Figure B-34). The no-plant zone should remain free of vegetation to prevent drain clogging.

Step 7. Specify Soil Media

Green roofs can be designed as flow-through systems or can be designed to detain a specific design volume of water (as determined by a qualified structural engineer). Sizing methodology presented in Bioretention can be used to design the system to capture a specific design volume. Soil media for green roofs should have the following characteristics:

- Well drained and aerated
- High porosity
- High nutrient holding capacity (cation exchange capacity)
- Permanent (non-biodegrading)
- Lightweight
- Windproof
- Stable (must support plants).

Several media types are available from green roof component suppliers, but generally expanded lightweight aggregates are preferred (e.g., expanded slate, expanded shale, expanded clay, terra cotta). For extensive green roofs, a minimum of 4 inches of media should be provided. Specifications should be included on design plans. Intensive green roofs should also employ lightweight aggregate media, but structural capacity generally allows a wider range of soil materials. Green roof media installation can be challenging and may require the use of a crane, auger, conveyor, or pneumatic delivery system.

Step 8. Select Vegetation

Green roof vegetation should consist of low-growing, highly drought-tolerant, biodiverse species that are adapted to survive in the harsh environment of a rooftop. Appropriate vegetation should be selected based on the specific site conditions and recommendations by local horticulturalists and green roof manufacturers.

GREEN ROOF REFERENCES

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APPENDIX B: **Additional Sand Filter Design Guidance**

Sand Filters

Figure B-35: *Surface sand filter at Parman Library, San Antonio, Texas. Source: Bender Wells Clark Design*

SAND FILTER DESIGN

Sand filters have many of the same design elements as bioretention but are typically not planted. Table B-12 lists the steps involved in sand filter design.

TABLE B-10. SAND FILTER ITERATIVE DESIGN STEP PROCESS

Design step		Design component/ consideration	General guidance
1	**Determine BMP Size (pg 210)**	Use Appendix J	
2	**Determine BMP Configuration (pg 211)**	Sand filter type	Based on available space and required access for maintenance, determine location and type of sand filter: • Surface sand filters: installed in shallow depressions on surface. Require pretreatment by vegetated swales, filter strip, or forebay. • Subsurface sand filters: can be installed along the edges of roads and parking lots to conserve space. Must include a sedimentation chamber for pretreatment.

TABLE B-10. SAND FILTER ITERATIVE DESIGN STEP PROCESS (CONT.)

3	**Determine BMP Function (pg 211)**	Impermeable liner	If non-infiltrating (per geotechnical investigation), use one of the following (as described in Common Design Elements): • Impermeable clay liner • Geomembrane liner • Concrete.
		Lateral hydraulic restriction barrier	Use concrete or geomembrane to restrict lateral seepage to adjacent subgrades, foundations, or utilities.
		Underdrain	Schedule 40 PVC pipe with perforations (slots or holes) every 6 inches. 4-inch diameter lateral pipes should join a 6-inch collector pipe, which conveys drainage to the downstream storm network. Provide cleanout ports/ observation wells for each underdrain pipe at spacing consistent with local regulations. See Appendix A, Common Design Elements.
		internal water storage (IWS)	If using underdrain and infiltration, elevate the outlet to create a sump for additional moisture retention to promote plant survival and treatment. Top of IWS should be more than 10 inches below the surface.
		No underdrain	If design is fully infiltrating, ensure that subgrade compaction is minimized.
4	**Size the System (pg 212)**	Temporary ponding depth	No deeper than 8 feet (shallower depth should be used in residential areas or near schools and parks)
		Soil media depth	1-5 feet (deeper for better pollutant removal, hydrologic benefits, and deeper rooting depths)
5	**Specify Soil Media (pg 212)**	Gradation	Washed concrete sand (ASTM C-33) free of fines, stones, and other debris
		Chemical composition	Total phosphorus < 15 ppm
		Gravel drainage layer	Separate sand media from underdrain with 2 inches of choking stone (ASTM No. 8) or geotextile over a 1-5-foot envelope of ASTM No. 57 stone
6	**Design Inlet and Pretreatment (pg 213)**	Inlet	Provide stabilized inlets (see Curb Cuts and Energy Dissipation)
		Pretreatment	Install rock armored forebay (concentrated flow), gravel fringe and vegetated filter strip (sheet flow), or vegetated swale
7	**Select and Design Overflow/ Bypass Method 213**	Outlet configuration	Online: All runoff is routed through system—install an elevated overflow structure or weir at the elevation of maximum ponding Offline: Only treated volume is diverted to system—install a diversion structure or allow bypass of high flows
8	**Design for Multi-Use Benefits (Appendix C)**	Include features to enhance aesthetics and public education.	

Step 1. Determine BMP Size

The sand filter must be sized to fully capture the desired or required design storm volume and filter it through the soil media. The sand filter should be oversized by 20 percent to accommodate the sediment accumulation in the surface of the sand filter, which reduces design volume (according to Barrett 2005). Surface storage (in the ponding area) and soil pore space provide capacity for the design storm volume retention. Appendix J outlines methods for determining design runoff depths associated with a range of annual treatment efficiencies. Once the design runoff depth is determined (according to the desired level of treatment), a runoff volume can be determined for the contributing watershed using this depth and the methods outlined in Appendix J, San Antonio Unified Development Code, or San Antonio River Basin Regional Modeling Standards for Hydrology and Hydraulic Modeling.

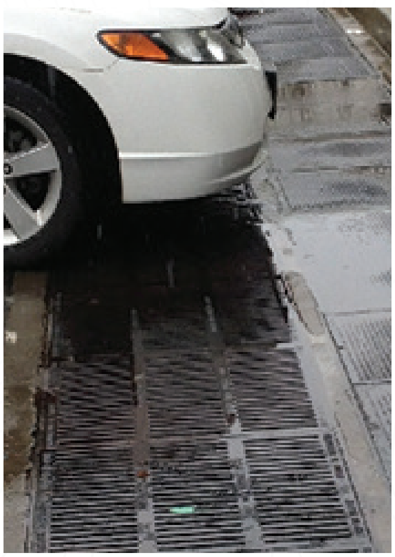
Peak flow rates for the design storm should also be calculated, using the methods outlined in Appendix J so that the inlet and pretreatment can be accordingly sized and flow attenuation can be considered.

Step 2. Determine BMP Configuration

Sand filters require less space than many BMPs and are typically used in parking lots or other highly impervious areas. Two basic configurations are available for sand filters: surface sand filters with a vegetated filter strip as a pretreatment element, or subsurface sand filters with a sedimentation/grit chamber. The aboveground option requires more space to incorporate the pretreatment filter, and it provides more pathogen reduction from the surface's exposure to sunlight.

Surface: Surface sand filters require some method of pretreatment, such as a filter strip or swale, to remove large solids and reduce the velocity of stormwater entering the BMP. Surface sand filters can be integrated into the site plan as recreation facilities or open space as shown in Figure B-35. Access should always be provided for routine, intermittent, and rehabilitative maintenance activities.

Subsurface: Subsurface sand filters require very little space and are easily incorporated belowground into the edge of parking lots and roadways. Subsurface sand filters require a pretreatment sedimentation chamber (typically 1-5-feet-wide) to allow large solids to settle. An example of a subsurface sand filter with a sedimentation chamber is shown in Figure B-36.



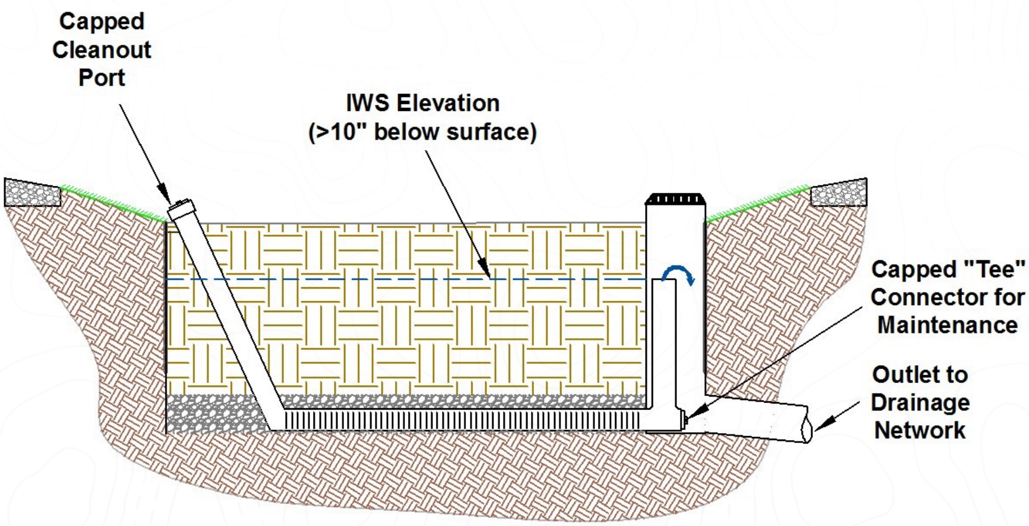


Figure B-36 (left). *Subsurface sand filter Raleigh, North Carolina. Source: Tetra Tech*
Figure B-37 (right).*Conceptual schematic of an infiltrating surface sand filter with IWS*

Step 3. Determine BMP Function

Sand filters should be designed as infiltrating practices whenever practicable. Geotechnical testing and drainage requirements are the same as for bioretention. Additionally, IWS can be used in infiltrating sand filters to increase residence time and improve volume reduction if subsoil infiltration rate is sufficiently high (e.g. infiltration rates of greater than 0-0.5 in/hour). Because plant survival is not a consideration in sand filters, the IWS elevation (underdrain outlet elevation) can be specified at 10 inches below the media surface. The IWS layer should not extend within 10 inches of the media surface because this is where the majority of sediment (and associated constituents) is captured; prolonged saturation of deposited sediments could cause previously captured pollutants to desorb/dissolve (Hunt et al. 2012). An example of a sand filter with IWS is shown in Figure B-37.

Step 4. Size the System

Vertical components of sand filters are similar to bioretention, except that there are no constraints imposed by vegetation. The following subsections describe sand filter sizing.

Surface Ponding Depth
The ponding depth of sand filters is not limited as with some BMPs because the effect on vegetation is not a concern. Depth is determined by the ability of the sand filter to completely drain within 48 hours and, therefore, is a function of the surface area and infiltration rate of the sand media. Ponding depth should not exceed 8 feet as a safety precaution, and it should be shallower near residential areas, parks, and schools. When surface sand filters feature deep ponding depths, safety precautions consistent with conventional ponds (shallow water safety shelves, fencing, etc.) should be specified in the design.

Media Depth
Sand media depth should be a minimum of 1-5 feet for sediment removal. For pollutant-specific media depths, see the bioretention section.

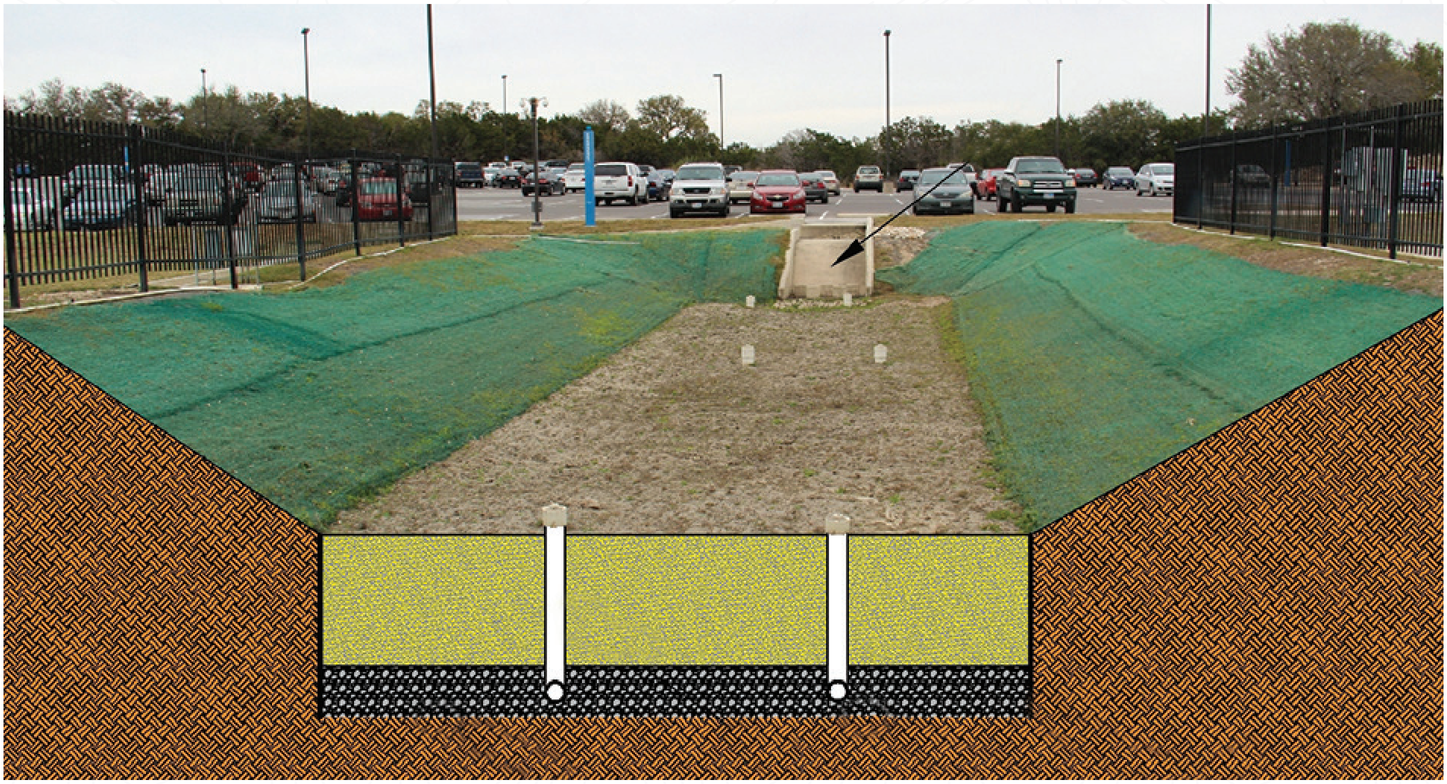


Figure B-38. *Rendering showing sand filter geometry and profile, University of Texas at San Antonio, San Antonio, Texas. Source: Tetra Tech*

Surface Area

The footprint of the sand filter should be sized using the equations provided in the bioretention section. Porosity of sand filter sand can be assumed equal to 0.4 for preliminary calculations, but actual laboratory-measured porosity should be used for final calculations. Although the footprint of sand filters can be smaller than bioretention because of deeper allowable surface ponding depths, smaller sand filters will require more frequent rehabilitative maintenance.

Step 5. Specify Soil Media

The soil media in the sand filter should be highly permeable; free of fines, stones, and other debris; and should meet the criteria listed in Table B-10. Media should be separated from gravel drainage layer using 2 inches of ASTM No. 8 choking stone or geotextile, as described in Common Design Elements.

Step 6. Design Inlet and Pretreatment

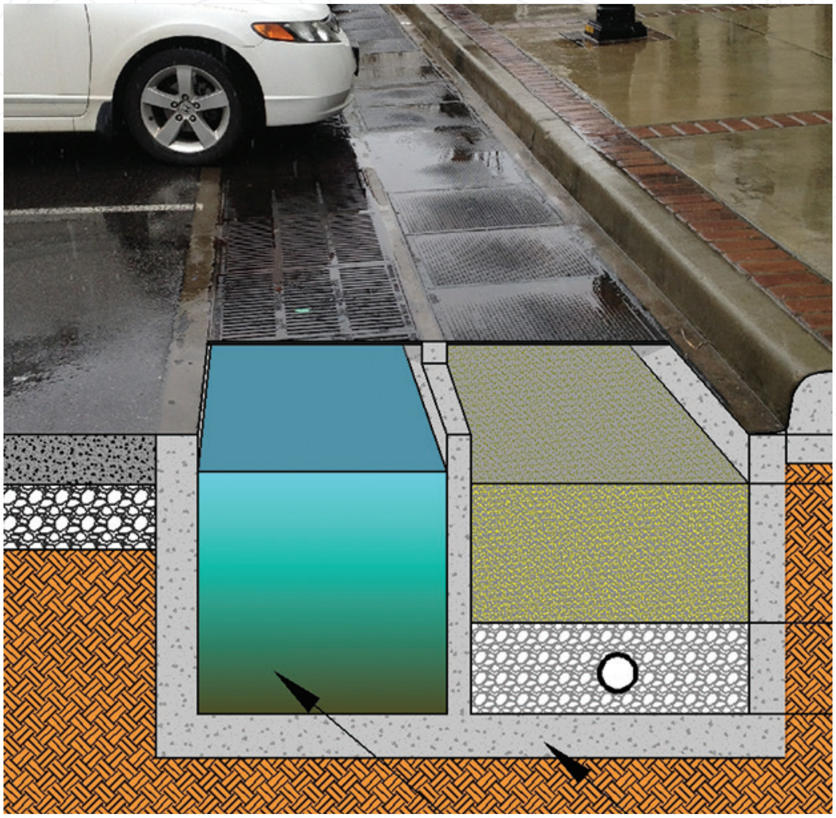


__Figure B-39.__ Rendering showing subsurface sand filter with diffusive flow inlet and slot weirs between sedimentation chamber and sand filter chamber, Raleigh, North Carolina. Source: Tetra Tech

Erosive velocities and high sediment loads can be detrimental to sand filters. Both aboveground and belowground sand filters require some type of pretreatment before stormwater contacts the filter media. Aboveground sand filters should be constructed with a flow diversion, where possible, to divert volumes that exceed the water quality volume away from the sand filter. Side slopes of above ground sand filters should be similar to a bioretention area. Below ground sand filters are typically installed in vaults and may be vertical. For more detail on diversion structures, see Appendix A, Common Design Elements. Vegetated filters can also be used with aboveground sand filters where space is available.

Flows entering sand filters should be diffused by passing over a level spreader before contacting the filter media to reduce flows, minimize filter media erosion, and distribute the flow over a larger surface area (see Vegetated Filter Strips for level spreader design details). Flows entering a subsurface sand filter should enter the sedimentation chamber and can be either concentrated or diffuse, depending on the inlet type. Concentrated flow, such as the flow for the end of a stormwater pipe, should enter the sedimentation chamber and flow into the media chamber over a level spreader to diffuse the flow before contacting the filter media as shown in Figure B-38. Diffuse flow passing into the sediment chamber over a level lip, such as the edge of a parking lot, should still flow over a level spreader before contacting the filter media. Figure B-397 shows a belowground sand filter with a diffuse flow inlet in a parking lot. It is important to distribute the flow across the surface area of the sand filter as much as possible to prevent the inflow from concentrating in one area, causing increased maintenance.

The sedimentation chamber should be dewatered between storm events to prevent vector issues; this can be done by installing a perforated riser pipe surrounded by a gravel envelope in a trash rack (see Appendix A, Common Design Elements). Sedimentation should typically be designed to hold 50% of the design water quality volume and have a depth of 2 feet to 3 feet to minimize scour of sediment deposition (Knox County 2008; Claytor and Scheuler 1996). Detailed pretreatment sizing guidance can be found in Claytor and Schueler (1996).

Step 7. Select and Design Overflow/Bypass Method

Sand filters can be designed as online or offline systems, but offline configurations are typically preferred to preserve the functional life of the filter media. Details for designing diversion structures for offline systems are provided in Common Design Elements. An alternative overflow should be incorporated for all configurations as a contingency for when the filter media clogs; doing so will prevent damage to the BMP and surrounding areas. Overflow options are described in Bioretention.

APPENDIX B: **Additional Sand Filter Design Guidance**

SAND FILTER REFERENCES

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APPENDIX B: **Additional Stormwater Wetland Design Guidance**

Stormwater Wetlands

Figure B-40: Stormwater Wetland, Lenoir, North Carolina. Source: Tetra Tech

STORMWATER WETLAND DESIGN STEPS

The design of a constructed stormwater wetland can be broken down to a nine-step process. Table B-11 summarizes the steps, which are described in greater detail.

TABLE B-11. STORMWATER WETLAND ITERATIVE DESIGN STEP PROCESS

Design step		Design component/ consideration	General guidance
1	**Determine BMP Treatment Volume and Flow Rates (pg 217)**	Use Appendix J. The design volume should be oversized by 20% to account for sediment accumulation over time.	
2	**Perform Feasibility Water Balance (pg 217)**	Evapotranspiration, infiltration	Estimate rate of water loss during drought to ensure that water is maintained in deep pools (shallow water zones do not need to remain wet year round)
3	**BMP Siting and Configuration (pg 217)**	BMP size	Incorporate into lowest areas of site
4	**Determine Geotechnical Requirements and Specify Liner (pg 217)**	Geotechnical investigation and impermeable liners	See Appendix A, Common Design Elements
5	**Design Inlet and Pretreatment (pg 218)**	Sediment forebay	Forebay should be 18–36 inches deep, 10% of the temporary ponding surface area, and should be lined with riprap for energy dissipation.

TABLE B-11. STORMWATER WETLAND ITERATIVE DESIGN STEP PROCESS (CONT.)

6	**Design Wetland Flow Path, Zones and Footprint (pg 218)**	Maximum flow path	The minimum length to width (L:W) ratio should be 2:1, but L:W should be maximized by creating a sinuous flow path and placing the outlet as far from the inlet as possible.
		Wetland zones	**Deep Pools:** 15%–20% of wetland surface area (including forebay), 18–36 inches deep. Transition: 10%–15% of wetland surface area, transition between deep pool and shallow water, 12–18 inches deep, maximum slope of 1-5:1. **Shallow Water:** 40% of wetland surface area, 3–6 inches deep, flat or 6:1 slope (at least 6-foot radius around all deep pools to provide safety shelf). Shallow water depths (less than 6 inches) provide optimum conditions for plant survival and should be verified during construction inspection. **Temporary Inundation:** 30%–40% of wetland surface area, up to 12 inches deep, 3:1 slopes. **Detention Storage/Upland:** Additional ponding depth can be provided for peak flow mitigation, as needed, but depth should generally not exceed 4 feet above the permanent pool elevation.
7	**Select and Design Outlet/ Bypass Method (pg 219)**	Outlet configuration	**Online**: All runoff is routed through the wetland basin—install an elevated riser structure or weir with an orifice at the permanent pool elevation and an overflow at the maximum temporary ponding elevation (if additional peak flow mitigation is required, a second orifice can be placed at the temporary ponding elevation and the overflow can be elevated to detain the necessary runoff). **Offline**: Runoff in excess of the design water quality volume bypasses the wetland basin—design a diversion structure per the guidance in Common Design Elements.
		Design drawdown orifice	Non-clogging orifices should feature a downturned pipe that extends 6 to 12 inches below the permanent pool elevation in an area of open water (deep pool) and allows drawdown of temporary ponding in 2 to 5 days.
		Maintenance and emergency dewatering design	A protected inlet should be provided near the base of the outlet structure with a tamper-proof manual valve (intake should be sized one standard pipe size larger than needed to dewater the basin in 24 hours).
		Outfall pipe and emergency overflow	The outlet pipe should incorporate measures to prevent lateral seepage and should discharge to an adequately stabilized area; an emergency spillway should be provided to safely bypass extreme flood flows.
8	**Specify Soil Media (pg 221)**	Wetland vegetation substrate	At least 1 to 4 inches of low-phosphorus, organic topsoil over the impermeable layer is typically required for plant establishment.
9	**Select Vegetation (pg 222)**	Wetland vegetation by zone	See Plant List (Appendix E).
10	**Design for Multi-Use Benefits (Appendix C)**	Site specific	Include features to enhance habitat, aesthetics, recreation, and public education as desired.

APPENDIX B: **Additional Stormwater Wetland Design Guidance**

Step 1. Determine BMP Treatment Volume and Flow Rates

The methods for determining wetland size are outlined in Appendix J. The wetland should be oversized by 20 percent to accommodate the sediment accumulation in the wetland, which reduces design volume (according to Barrett 2005).

Peak flow rates should also be calculated using methods outlined in the San Antonio Unified Development Code or San Antonio River Basin Regional Modeling Standards for Hydrology and Hydraulic Modeling, so that the inlet, pretreatment, outlet, bypass and other hydraulic features can be accordingly sized and flow attenuation considered.

Step 2. Perform Feasibility Water Balance

A stormwater wetland's function relies on the wetland retaining an adequate supply of water between storm events to ensure plant vigor and to maintain habitat for mosquito-eating fish (Hunt et al. 2005).

Wetlands should have enough water supplied from groundwater, runoff, or baseflow so that the permanent pools will not draw down by more than 2 feet after a 30-day drought. Where seasonally low groundwater elevations intersect with the wetland features (see step 4) groundwater resources might be sufficient to supply enough water to ensure plant survival. In areas where an impermeable liner is incorporated into the wetland design, a water balance evaluation should be conducted to determine if the necessary water will be retained in the deep pools. In doing this, the designer should consider precipitation, evapotranspiration, runoff, infiltration (if unlined), and any other inputs or outputs of water from the system. Note that the water balance should be performed only for the deep pools because wetland plants established in shallow water zones are well-adapted to periods of drought. Guidance on one method for conducting a water balance is in Hunt et al. (2007) and Knox County (2008).

Step 3. BMP Siting and Configuration

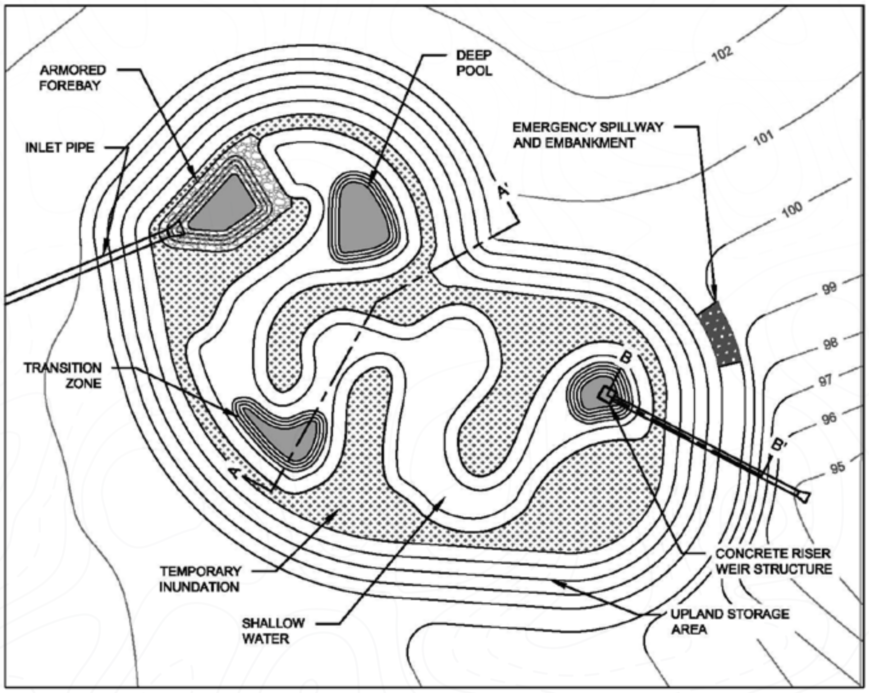

Figure B-41. *Example wetland configuration*

Constructed stormwater wetlands are typically constructed in the lowest area of a site such that runoff can be conveyed by gravity flow and so that excavation is minimized. The stormwater wetland location should provide adequate elevation difference, typically 3 feet or more, to discharge water to the existing stormwater network without the need for pumps. Constructed wetlands can be incorporated along the perimeter of a site by designing a long, linear footprint, or it can serve as an attractive amenity in common areas of developments. If the entire design volume cannot be stored in one location or if utility conflicts are apparent, wetland pockets can be distributed between several locations and connected with vegetated channels or buried conduit. For an example wetland configurations, see Figure B-41.

Step 4. Determine Geotechnical Requirements and Specify Liner

Unlike many other stormwater BMPs, stormwater wetlands are not intended to infiltrate runoff. As such, the subsoil conditions must be investigated to determine in situ infiltration rates, depth to seasonal high groundwater table, and underlying geology (including proximity to Edwards Aquifer Recharge, Contributing, and Transition zones). For details regarding geotechnical investigations, see Appendix A, Common Design Elements.

APPENDIX B: **Additional Stormwater Wetland Design Guidance**

If the site features a high groundwater table and is in an area where infiltration is permitted, hydraulic restriction layers might not be needed—in these situations the high groundwater table will help maintain a permanent pool in the stormwater wetland. If the groundwater table is deeper than the proposed permanent pool elevation or the site is in an area with sensitive subsurface resources, adequate hydraulic restriction layers should be specified to prevent infiltration. For details on designing hydraulic restriction barriers, see Appendix A, Common Design Elements.

Step 5. Design Inlet and Pretreatment

A rock-lined forebay stills incoming runoff and allows larger particles to settle as illustrated in Figure B-42.

Figure B-42. *Rock-lined forebay visible in a newly planted stormwater wetland, Wilmington, North Carolina. Source: Tetra Tech*

Step 6. Design Wetland Flow Path, Zones and Footprint

Designing the internal wetland features, zones, and footprint is an iterative process. The design must balance storage volume requirements with existing site grading and desired flow length ratios.

The flow length through the wetland should be maximized to improve residence time and treatment. This can be done by incorporating a sinuous flow path or by using berms to form racetrack style configurations (see Figure B-43). The L:W ratio (as measured from inlet to outlet and using the average width of the basin) should be 2:1, minimum, but 3:1 is preferred. The width of the flow path will be determined by the flow length and the desired shallow water area.

The wetland area should be divided into four zones, as specified in Table B-12. Although deep pools are important for maintenance of water and wildlife (including mosquito-eating predators) during dry periods, the shallow water zone is also critical for plant survival. One of the most common causes of wetland plant die-off is designing the shallow water zone too deep—depths greater than 6 inches will reduce plant survival rates and encourage the encroachment of invasive plant monocultures which can, in turn, harbor mosquito habitat. (Hunt et al. 2005).

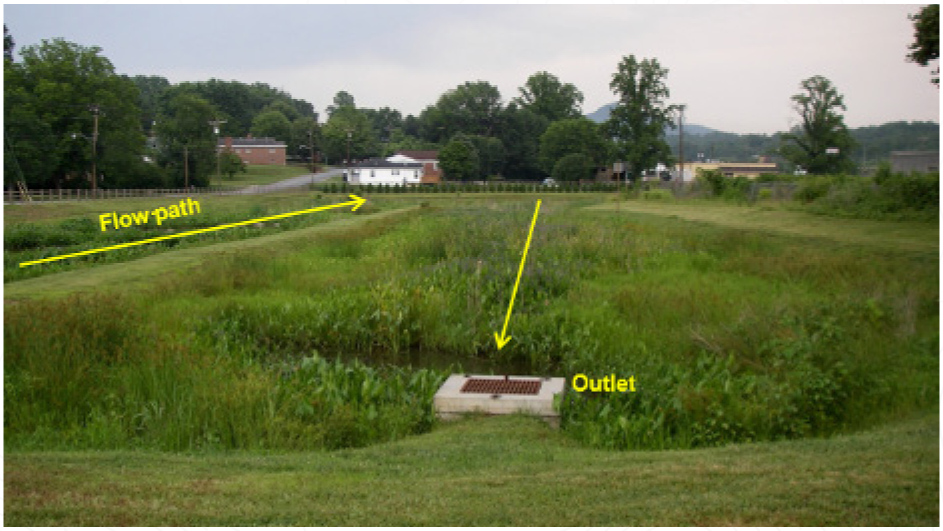

Figure B-43. *An earthen berm elongates the flow path in a racetrack-style stormwater wetland where the inlet and outlet are located in close proximity, Lenoir, North Carolina. Source: Tetra Tech*

Hunt, W.F., C.S. Apperson, and W.G. Lord. 2005. Mosquito Control for Stormwater Facilities. Urban Waterways.

TABLE B-12`. WETLAND ZONES

Deep Pools: 15%–20% of wetland surface area (including forebay), 18–36 inches deep
Transition: 10%–15% of wetland surface area, transition between deep pool and shallow water, 12–18 inches deep, maximum slope of 1-5:1.
Shallow Water: 40% of wetland surface area, 3–6 inches deep, flat or 6:1 slope (at least 6-foot radius around all deep pools to provide safety shelf). Shallow water depths (less than 6 inches) provide optimum conditions for plant survival and should be verified during construction inspection.
Temporary Inundation: 30%–40% of wetland surface area, up to 12 inches deep, 3:1 slopes
Detention Storage/Upland: Additional ponding depth can be provided for peak flow mitigation, as needed, but depth should generally not exceed 4 feet above the permanent pool elevation

The wetland footprint must be configured so that the wetland contains in its temporary and permanently ponded areas a storage volume equal to or greater than the treatment volume detailed in step 1. Determine the storage volume by using algorithms available in computer aided design software, which is typically used to develop the wetland grading plan. Alternatively, use the equation below to evaluate the storage volume for a proposed wetland configuration.

[Equation B-7-1]

$$V=(2DP)+(0.375SW)+(1.25TZ)+[TP(DP+SW+TZ)]+[TI(\frac{1}{2TP})]$$

where:

V = treatment volume contained in the stormwater wetland

DP = area of wetland dedicated to deep pool zone (sq ft)

SW = area of wetland dedicated to shallow water zone (sq ft)

TZ = area of wetland dedicated to transition zone (sq ft)

TP = temporary ponding depth of wetland (ft)

TI = area of wetland dedicated to temporary inundation zone

Step 7. Select and Design Outlet/Bypass Method

As with other BMPs, stormwater wetlands can be designed as online or offline systems. Regardless of the configuration, mechanisms are required to draw down water in the wetland basin between storm events and for maintenance. The following sections discuss the outlet design.

Online versus Offline Configuration

The outlet or bypass configuration will depend on the drainage area size, available space for onsite detention, and design goals. If a wetland is designed as an offline system, a diversion structure should be installed to route the design volume into the basin (according to the guidance provided in Appendix A, Common Design Elements). Offline wetlands can be smaller than online wetlands, which makes them ideal for retrofit scenarios because they need not provide capacity (volumetrically and hydraulically) for routing higher flows.

If an online system is desired, all runoff from the catchment is routed through the basin and out a multistage outlet structure with capacity to allow high-volume flows to safely overflow. The outlet structure should be placed near the edge of the wetland for easy maintenance access. If additional peak flow mitigation is desired, a secondary orifice or weir can be installed at the elevation of temporary ponding and the overflow can be elevated to allow larger storms to bypass. The maximum detention depth should be 4 feet above the permanent pool to reduce effects on wetland vegetation. An example outlet structure schematic is shown in Figure B-44.

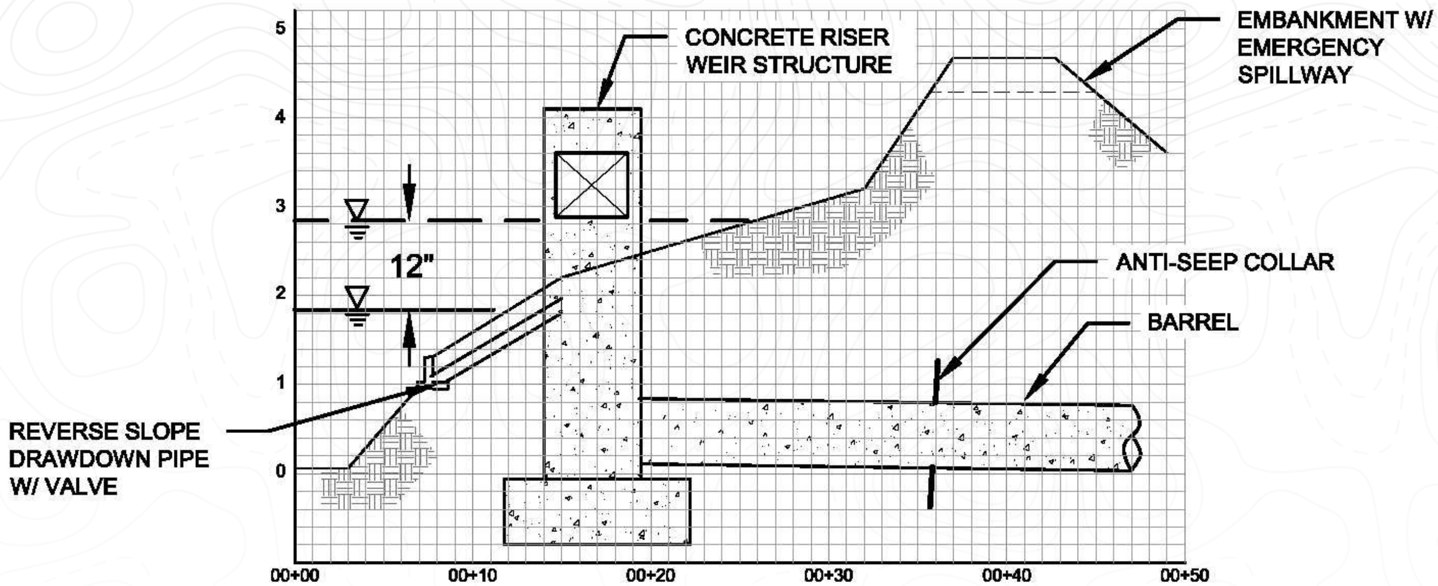

Figure B-44. Wetland outlet structure schematic

Design drawdown Orifice

A non-clogging orifice should be designed to draw down the water quality design volume in 2 to 5 days. Longer residence times are preferred to maximize treatment efficiency. The orifice should be equipped with a trash rack or a downturned intake pipe that extends 6 to 12 inches below the surface of a nearby area of open water. Submerging the intake pipe will reduce the risk of blockage caused by floating debris. A capped tee-connection can be installed on the end of the pipe for easy cleaning (when the cap is removed, a ramrod can be used to dislodge any debris that has accumulated around the submerged intake). Additional guidance on trash racks and non-clogging orifices is provided in Common Design Elements and Barrett (2005).

Additionally, installing an adjustable orifice can help with plant establishment (an example shown in Figure B-45 uses an orifice in a metal plate that can be rotated on a flange fitting to adjust orifice elevation); lowering the orifice to maintain shallower permanent pool depths for several weeks after planting will improve plant survival rates. After plants are established, the adjustable orifice can be elevated to capture the intended water quality design volume. Alternatively, the outlet structure can include a flashboard riser that uses removable boards to control the stage of water in the wetland.

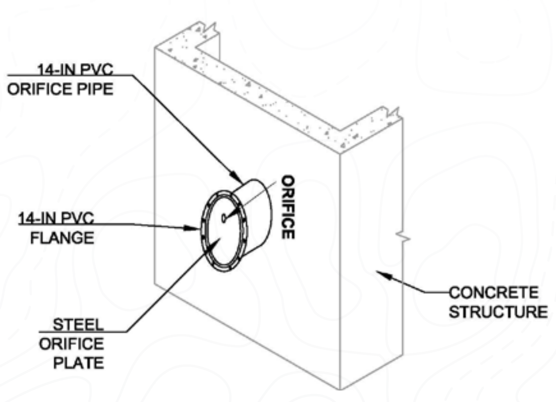

Figure B-45. Example schematic of an adjustable orifice plate

The drawdown orifice should be sized to draw down the temporary ponding depth using the following orifice equation:

[Equation B-7-2]

$$Q=C_d\pi\left(\frac{d^2}{48}\right)\sqrt{2gH}$$

Barrett, M.E. 2005. Complying with the Edwards Aquifer Rules. Technical Guidance on Best Management Practices. RG-348. Prepared for Texas Commission on Environmental Quality, Field Operations Division, Austin, TX.

where:

Q = Discharge (cfs) computed by dividing the storage volume above the permanent pool by the desired drawdown period
C_d = Coefficient of Discharge (0.60 for sharp edged orifice without projections)
π = pi (3.14)
d = orifice diameter (in)
g = acceleration of gravity (32.2 ft/sec^2)
H = driving head (ft) measured from the center of the drawdown orifice to water surface.

Note: Use $H = \frac{H_o}{3}$ as an approximation of the driving head throughout the drawdown period.

where:

H_o = Driving head (ft) measured from the center of the drawdown orifice to the crest of the overflow/bypass weir.

Design Maintenance/Emergency Dewatering Intake

A manually operated intake valve should be provided at the lowest possible stage of the wetland to allow drawdown for maintenance. The intake should be protected with gravel or a trash rack, or both, to minimize clogging and be sized one standard pipe size larger than would be needed to dewater the entire wetland basin within 24 hours. The valve should have locking features to prevent unauthorized dewatering. Figure B-46 shows an example of a maintenance dewatering intake. A flashboard riser can also be installed for rapid dewatering.

Figure B-46. *Maintenance dewatering intake design that could be used in a stormwater wetland, Raleigh, North Carolina. Source: Tetra Tech*

Outfall Pipe and Emergency Spillway

The outfall should be adequately stabilized with energy-dissipation devices to prevent scour of downstream sediment (for energy dissipation configurations, see Common Design Elements). A pipe collar or other engineering solution should also be installed to prevent seepage of water through the soil along the edge of the pipe. This piping can cause dangerous failure of embankments and drain the wetland's permanent pool. Additionally, an emergency spillway should be provided to allow 1 foot of freeboard during the 25-year event and should safely convey flows up to and including the 1 percent average recurrence interval event (Barrett 2005).

Step 8. Specify Soil Media

A 1- to 4-inch layer of topsoil must be provided for plant establishment because stormwater wetlands are typically lined with hydraulic restriction layers. Depth of soil will depend on specified plantings and underlying soil characteristics—consult a plant specialist as needed. Native soils excavated in construction can be used, but a soil test should confirm that the soils contain adequate nutrients for plant survivability (subsoils tend to be relatively infertile, so topsoil should be separately stockpiled for this purpose). Soils should not contain excessive levels of phosphorus (greater than 15 ppm) because this nutrient tends to dissociate from the soil under saturated conditions.

APPENDIX B: **Additional Stormwater Wetland Design Guidance**

Step 9. Select Vegetation

Although wetlands are typically wet, most native wetland plants are well adapted to surviving long periods of drought. Emergent plant survival rates dramatically decrease when normal water depth exceeds 6 inches. It is recommended that a diverse selection of native flowering, emergent species are planted throughout the shallow water zone (3 to 6 inch depth) around the wetland. This will provide the optimum habitat for mosquito predators, such as dragonflies, and reduce plant die-off. At least three species, preferably more, should be planted in each wetland zone.

Although trees and shrubs can provide habitat, shade, and aesthetic benefits, take care to immediately remove woody vegetation from embankments to prevent geotechnical failures. See Plant List (Appendix E).

STORMWATER WETLAND REFERENCES

Barrett, M.E. 2005. Complying with the Edwards Aquifer Rules. Technical Guidance on Best Management Practices. RG-348. Prepared for Texas Commission on Environmental Quality, Field Operations Division, Austin, TX.

Hunt, W.F., C.S. Apperson, and W.G. Lord. 2005. Mosquito Control for Stormwater Facilities. Urban Waterways. http://www.bae.ncsu.edu/stormwater/PublicationFiles/Mosquitoes2005.pdf

Hunt, W., M. Burchell, J. Wright and K. Bass. 2007. Stormwater Wetland Design Update: Zones, Vegetation, Soil and Outlet Guidance. Urban Waterways. North Carolina State Cooperative Extension Service, Raleigh, NC. http://www.bae.ncsu.edu/stormwater/PublicationFiles/WetlandDesignUpdate2007.pdf

Knox County. 2008. 3.1.8 Water Balance Method. Knox County Tennessee Stormwater Management Manual. Volume 2 (Technical Guidance). http://www.knoxcounty.org/stormwater/pdfs/vol2/3-1-8 Water Balance Calculations.pdf.

APPENDIX B: **Additional Extended Detention Basin Design Guidance**

Extended Detention Basins

Figure B-47: Extended Detention Basin in Grant Ranch, Colorado. Source: Urban Drainage Flood Control District

EXTENDED DETENTION BASIN DESIGN STEPS

The design of an extended detention basin can be broken down into the following process summarized in Table B-17 and described in greater detail below the table.

TABLE B-13. EXTENDED DETENTION BASIN ITERATIVE DESIGN STEP PROCESS

Design Step		Design component/ consideration	General guidance
1	**Determine BMP Treatment Volume (pg 226)**	The design volume should be oversized by 20% to account for sediment accumulation over time.	
2	**Determine Flood Control Volume (pg 226)**	Based on the appropriate local requirements*	

TABLE B-13. EXTENDED DETENTION BASIN ITERATIVE DESIGN STEP PROCESS (CONT.)

3	**BMP Siting and Configuration (pg 226)**	BMP location	Locate down gradient of disturbed/developed areas, in an area that will collect the most runoff from the site's impervious surfaces; avoid steep slopes; limit tree removal, which will destabilize soils and may contribute pollutants to influent.
		BMP size and shape	To maximize sedimentation processes, the basin length (measured along flow path from inlet to outlet) to width ratio should be between 2:1 and 3:1. Side slopes should be no steeper than 3:1 and should be 4:1 or flatter for improved safety, maintenance, and aesthetics.
4	**Determine Geotechnical Requirements and Specify Liner if Necessary (pg 226)**	Geotechnical investigation and impermeable liners	Consider location and potential benefit and concerns of infiltration. See Appendix A, Common Design Elements.
5	**Design Inlet and Pretreatment (pg 227)**	Sediment forebay	The forebay volume should be sized to 10% of water quality volume and be 2 to 5 feet deep. It should incorporate a sediment depth marker, for measuring accumulation of sediment, as well as energy dissipation at the inlet in order prevent erosion or resuspension of sediment.
		Other forms of pretreatment	Vegetative Filters and Grassy swales can be incorporated upstream or downstream of EDB. See Vegetated Swales.
6	**Design Basin Flow Path and Zones (pg 228)**	Maximum flow path	The minimum length to width (L:W) ratio must be at least 2:1, but L:W should be maximized by creating a sinuous flow path and placing the outlet as far from the inlet as possible. Baffles may also be considered.
		Basin zones	**Trickle or Low Flow Channel:** To convey flow from the forebay to the micropool, a concrete lined low flow channel or trickle channel is required; a slope between 0.4% - 1% and 9 inches deep is recommended; channel can be "V" shaped or a concrete channel with curbs. **Micropool:** 10% of treatment volume or 5% of the surface area of the water quality pool; the micropool should be a permanent pool located near the outfall; slopes should be 3:1 with a minimum surface area of 10 square feet; micropools should not have a low flow pilot channel. **Detention Storage:** Additional ponding depth can be provided for peak flow mitigation; the design storage volume should be based on the appropriate local requirements.

TABLE B-13. EXTENDED DETENTION BASIN ITERATIVE DESIGN STEP PROCESS (CONT.)

7	**Select and Design Outlet/Bypass Method (pg 229)**	Outlet configuration	The outlet should be designed as a riser with orifices to discharge the water quality volume over a 48 hour period. The basin must include the low-flow outlet to slowly release water, additional outlets to release peak flows of larger design storms, and trash rack to prevent clogging of both out-lets. See local ordinances for flood discharge requirements.
		Design drawdown orifice	Non-clogging orifices sized to allow for complete drawdown of the water quality volume in 48 hours and no more than 50% of the water quality volume should drain from the facility within the first 24 hours. The lowest orifice is to be placed at the de-sign elevation for the micropool.
		Maintenance	A protected inlet should be provided near the base of the outlet structure with a tamper-proof manual valve (intake should be sized one standard pipe size larger than needed to dewater the basin in 24 hours).
		Outfall pipe and emergency overflow	The hydraulic design of the outfall structure should consider tailwater effects from downstream waterways; an emergency spillway should be sized to safely pass the flow based on the appropriate local requirements for the flood control detention volume.
		Trash rack	Trash racks should be designed so as to prevent clogging of the smallest outlet opening. This may require design of a grate or screen for the water quality orifices in addition to a larger rack for the flood control openings. Trash racks must be large enough so that partial clogging will not adversely restrict flows reaching the control outlet. Spacing of the rack bars should be wide enough to avoid interference, but close enough to provide the level of clogging and safety protection required.
8	**Select Vegetation (pg 232)**	Vegetation by zone	Basin bottom, berms, and side slopes should be planted with native or meadow grasses. See Plant List (Appendix E). A minimum 25-foot vegetative buffer should extend away from the top of the pond slope in all directions – woody vegetation should not be planted in this zone, but existing trees should remain.
9	**Design for Multi-Use Benefits (Appendix C)**	Site specific	Include features to enhance habitat for beneficial pollinators, aesthetics, recreation, and public education as desired.

**Detention requirements based on local ordinances for design flood events and freeboard requirements.*

APPENDIX B: **Additional Extended Detention Basin Design Guidance**

Step 1. Determine BMP Treatment Volume

The extended detention basin should be sized to fully capture the desired or required design storm volume. Consult with the appropriate entities for the required regulatory guidance on a design storm. Once the design storm is known, Equation 1 from Appendix J can be used to calculate the water quality volume.

The forebay and primary basin should be oversized by 20 percent to accommodate sediment accumulation, which reduces design volume (Barrett, 2005). Peak flow rates for the design storm should also be calculated so that the inlet and outlets may be appropriately sized and flow attenuation designed.

Step 2. Determine Flood Control Volume

Total volume should be estimated based on the storm of interest. Peak flow rates should be calculated using methods outlined in local ordinances, such as the San Antonio Unified Development Code or San Antonio River Basin Regional Modeling Standards for Hydrology and Hydraulic Modeling, so that the inlet, pretreatment, outlet, bypass and other hydraulic features can be accordingly sized and flow attenuation considered.

Step 3. BMP Siting and Configuration

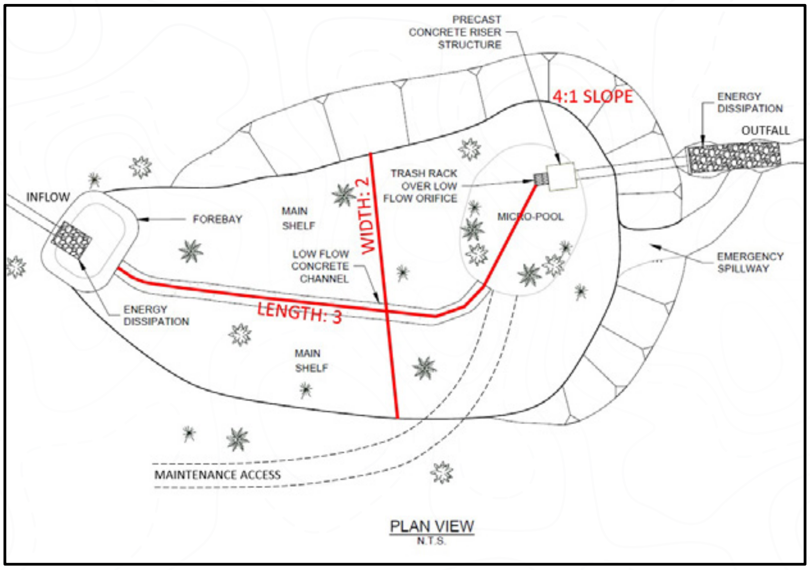

Figure B-48. *Example, Extended Detention Basin Configuration*

To identify the appropriate location of EDBs, the designer should consider flow patterns and try to place in the lowest area of a site such that runoff can be conveyed naturally by gravity flow and to minimize excavation. Locating the EDB near steep slopes should be avoided. It is recommended to locate the basin where tree removal can be limited to prevent destabilization of soil and potential increase in sediment loads.

For an example EDB configuration, see Figure B-48. The basin length to width ratio, which is measured along flow path from inlet to outlet, should optimally be between 2:1 and 3:1 to maximize sedimentation processes. Side slopes should be no steeper than 3:1 and should be 4:1 or flatter for improved safety, maintenance, and aesthetics.

Step 4. Determine Geotechnical Requirements and Specify Liner if Necessary

The primary pollutant removal pathway for EDBs is not infiltration of runoff. However, in areas where soils are appropriate and groundwater water quality hot spots are not present, infiltration can be an effective method to treat pollutants. The subsoil conditions must be investigated to determine in situ infiltration rates, soil composition, depth to seasonal high groundwater table, and underlying geology (including proximity to bedrock or Edwards Aquifer Recharge, Contributing, and Transition zones). For details regarding geotechnical investigations, see Appendix A, Common Design Elements. If clay, bedrock, or other impermeable layers are present in the subsoil, then infiltration should not be anticipated in the EDB design.

The lowest elevation in the EDB should be at least 2 feet above the high groundwater table and one foot above bedrock. Adequate clearance from the water table is essential in order to keep the basin bottom maintainable and dry. Permanently wet bottoms can become breeding grounds for mosquitos. If there is a risk for contamination of groundwater below the facility or if the site is in an area with sensitive subsurface resources, adequate hydraulic restriction layers should be specified to prevent infiltration. For details on designing hydraulic restriction barriers, see Appendix A, Common Design Elements.

EXAMPLE STEP 4: DETERMINE THE MAXIMUM DEPTH OF THE EDB

The location selected for the EDB has the following geotechnical characteristics:

Depth to Seasonally High Water Table = 20 feet
Depth to Bedrock = 100 feet
Depth to Clay = 10 feet

An EDB with a designed bottom depth 15 feet below the existing ground surface would have:
-5 feet water table clearance (20 ft – 15 ft)
-85 feet bedrock clearance (100 ft – 15 ft)
-Interference with the clay layer at bottom 5 feet (10 ft – 15 ft)

Conclusion: The basin will be dry between rain events with no groundwater interference. Bedrock will not restrict infiltration or construction activity. Presence of a clay layer will likely prevent water from infiltrating.

Note: A depth of 15 feet is only suitable if there is adequate surface area available.

Use a trial and error approach to determine the surface area required to hold 177,600 cubic feet (calculated in Step 2) in a total depth of 15 feet. This is because the volume of the basin will depend on the bottom shape and side slopes of the basin.

To estimate the volume of a proposed basin using the end-area method: Divide the basin into stages by drawing contours no more than 5 vertical feet apart. Determine the area under each contour. Next determine the volume of each stage using the following formula:

V = ½ H (A1 + A2)

where:
V = Volume of each slice
A1 = the area of the contour at the bottom of the stage
A2 = the area of the contour at the top of the stage
H = the vertical distance between A1 and A2.

Finally, sum the volume of each stage to find the total volume of the proposed basin.

Step 5. Design Inlet and Pretreatment

A rock-lined forebay reduces the velocity of incoming runoff and allows larger particles to settle. Figure B-49.

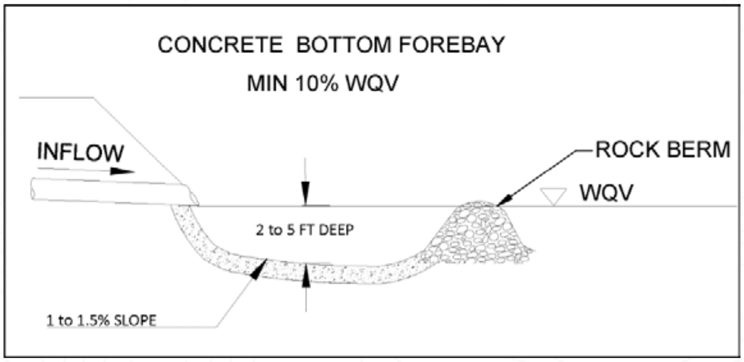

Figure B-49. *Example, Sedimentation forebay*

EXAMPLE STEP 5: FOREBAY SIZING

From Step 1, the water quality volume is 25,864 cubic feet (cf)

Required forebay volume:

25,864 cf x 10% = 2,586 cf

2,586 cf x 120% = 3,103 cf. (Oversize by 20% to allow for sediment accumulation.) The desired depth is 3 feet. If we assume the forebay will have nearly vertical sides, then the approximate surface area of the forebay will be:

3,103 cf / 3 ft = 1,034 sf

APPENDIX B: **Additional Extended Detention Basin Design Guidance**

Figure B-50. Concrete trickle channel

Step 6. Design Basin Flow Path and Zones

Flow Path
As mentioned in Step 3, the basin length to width, which is dependent upon the EDB flow path, should optimally be between 2:1 and 3:1. Use of berms, baffles, or swales may be incorporated to increase the effective length.

Low Flow or Trickle Channel
The low flow or trickle channel will convey water from the forebay to the micropool, outlet, or other enhanced EDB feature. A longitudinal slope of .04 to 1% should be maintained for the low flow channel.

A concrete channel is recommended. A flat bottomed channel shape is recommended in order to facilitate maintenance by standard equipment. Riprap and soil riprap are not recommended as they can be damaged during sediment removal. Erosion protection may be needed at the downstream end of the low flow channel.

Basin Storage
Additional storage for flood control based on the appropriate local requirements should be provided. Capacity above the water quality volume may be designed using the calculated flow and volume requirements from local regulations. Using volumes and flows calculated, additional capacity can be added on to the water quality volume. The spillway for the EDB should be set based on the designed flood control volume surface elevation.

Basin Design
Side slopes for the embankments should be at a maximum of 3:1 side slopes, but optimally should be 4:1 or flatter. The entire basin should be designed with consideration towards maintenance access. A 10 foot access driveway at a maximum 5% slope to the pond and into the pond would allow for maintenance trucks and excavators to perform necessary maintenance activities. The access points should be designed with consideration to the geotechnical investigation. If reinforced access is needed, this will need to be incorporated into the overall pond design.

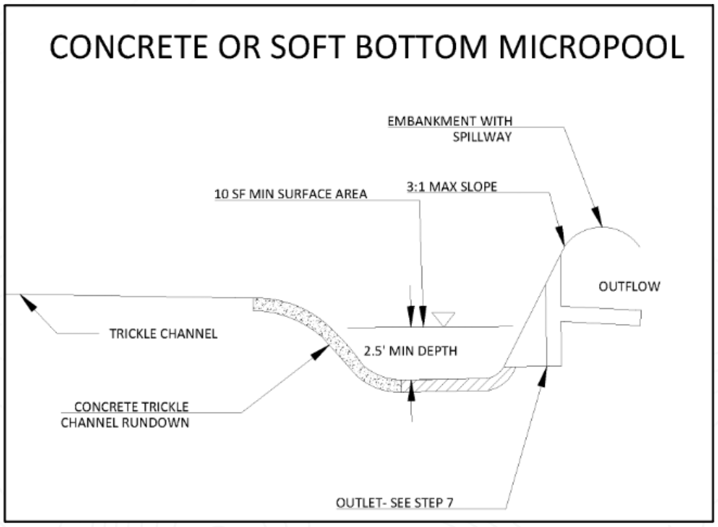

Figure B-51. Micropool configuration

Micropool
The EDB design should include a small permanently ponded micropool. The micropool should be located directly in front of the outlet structure in the embankment of the EDB and requires side slopes of vertical walls or stabilized slopes of 3:1 (H:V). The micropool should be at least 2-5 feet in depth with a minimum surface area of 10 square feet. The bottom should be concrete unless a baseflow is present or anticipated or if groundwater is anticipated.

Step 7. Select and Design Outlet/Bypass Method

For an EDB, the outlet structure is sized to control all design storms (based upon hydrologic routing calculations) and should consist of a riser with orifices designed to convey the design volume over a 48-hour period. Other control structures must be discussed with SARA. Small outlets that will be subject to clogging or are difficult to maintain are not acceptable.

Outlet and Trash Rack
The outflow structure should be sized to allow for complete drawdown of the water quality volume within at least 48 hours. Since sediment will tend to accumulate around the lowest stage outlet, the inside of the outlet structure for an EDB should be depressed below the ground level to minimize clogging due to sedimentation. Depressing the outlet bottom to a depth below the ground surface at least equal to the diameter of the outlet is recommended. No more than 50% of the water quality volume should drain from the facility within the first 24 hours. Most EDB outlet structures include an orifice plate for water quality discharge and a larger opening for discharging water at the flood control level. An orifice opening should typically be no smaller than 2-5 inches (unless a special non-clogging design is provided). A water-tight seal (rubber boot or equivalent) must be provided between all riser and pipe joint connections to minimize leakage. Seepage control or anti-seep collars should be provided for all outlet pipes. Additional outlets must be submitted to the San Antonio River Authority for approval.

An orifice is a circular or rectangular opening of a prescribed shape and size. The flow rate depends on the height of the water above the opening and the size and edge treatment of the orifice. For a single orifice, as illustrated in Figure B-52(a), the orifice discharge can be determined using the standard orifice equation below.

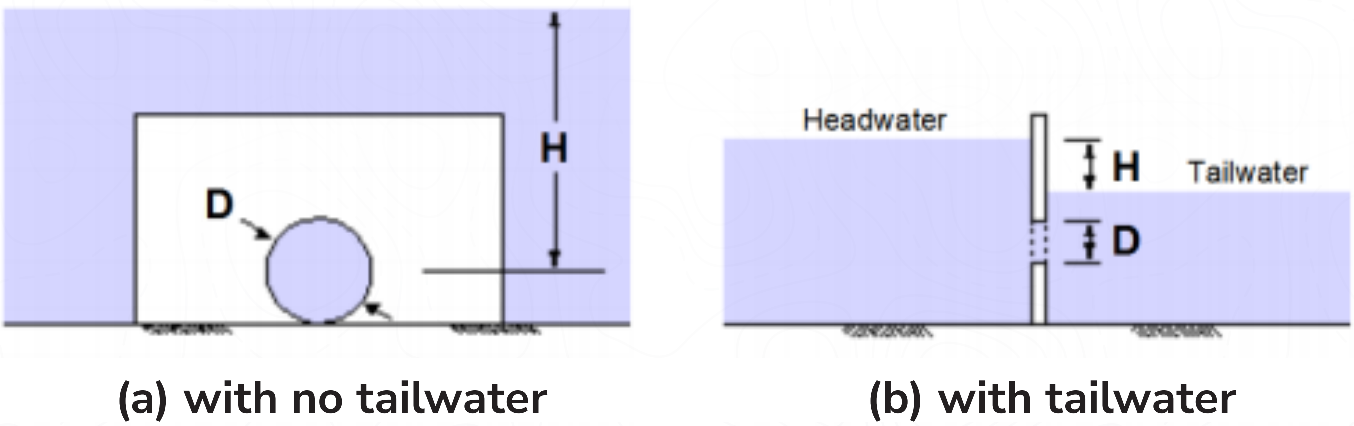

Figure B-52. *Orifice definitions (a, left; b, right)*

[Equation B-8-1]

$$Q=CA\sqrt{2gH}$$

where:
- **Q** = discharge (cfs)
- **C** = orifice coefficient
- **A** = cross sectional area of orifice (ft^2), computed using diameter "D"
- **H** = effective head on the orifice, from the center of orifice to the water surface (ft)
- **D** = orifice diameter (ft)
- **g** = gravitational acceleration (32.2 ft/s^2)

The effective head at the orifice (H) varies depending on the tailwater condition. If the orifice discharges as a free outfall (See Figure B-52(a).), then H is measured from the center of the orifice to the upstream (headwater) surface elevation. If the orifice discharge is submerged (See Figure B-52(b).), then H is the difference in elevation of the headwater and tailwater surfaces as shown in Figure B-52(b).

TABLE B-14. ORIFICE COEFFICIENTS FOR DIFFERENT CONFIGURATIONS

	Sharp Edged	Round Edged
Material Thickness < Orifice Diameter	0.6	0-92
Material Thickness > Orifice Diameter	0.8	

For square-edged entrance conditions the generic orifice equation can be simplified:

[Equation B-8-2]

$$Q=0.6A\sqrt{2gH}=3.78D^2\sqrt{H}$$

where:
D = orifice diameter (ft)
H = Effective head at the orifice (ft) - See paragraph above.

Trash Racks
As shown in Figures B-54 and B-55, trash rack designs vary from case to case and many are designed specifically for the outlet structure they are placed around. In selecting a trash rack design, it is important to take maintenance access and safety into account. Ideally, outlet structures should be placed in or close to the basin embankment. Such a placement would allow the trash rack on the overflow opening to be sloped into the embankment. This enables access for maintenance and provides assistance for a child or small animal to climb up and away from the overflow outlet structure.

Trash racks must be large enough so that partial clogging will not adversely restrict flows reaching the control outlet. The surface area of all trash racks should be maximized and the trash rack should be located a suitable distance from the protected outlet to avoid interference with the hydraulic capacity of the outlet. The trash rack can be sized using the orifice and/or outlet size according to the relationship shown in Figure B-53. The location and size of the trash rack depend on a number of factors, including head losses through the rack, structural convenience, safety, and size of outlet.

The spacing of trash rack bars must be proportioned to the size of the smallest outlet protected. In many cases this means that the orifice plate will need a separate trash rack grate or screen to prevent the orifices from clogging, while a larger trash rack can be placed over the overflow opening or around the entire outlet structure. Spacing of the rack bars should be wide enough to avoid interference, but close enough to provide the level of clogging and safety protection required. In designing both the orifice screen and the overflow trash racks, it is important to consider how the racks will attach to the outlet structure.

Racks should be secured to the outlet in a manner that will allow maintenance personnel to remove any accumulated sediment or materials from behind the rack while at the same time avoiding gaps between the rack and the outlet structure. The trash rack should be located at a suitable distance away from the outlet, such that the hydraulic capacity of the outlet is not reduced due to interference.

The trash racks must have a combined total open area such that partial plugging will not adversely restrict flows through the outlet structure. A common rule-of-thumb is to provide a trash rack open area at least 10 times larger than the control outlet orifice (ASCE, 1992).

EXAMPLE STEP 7: SIZE THE TRASH RACK

Per City of San Antonio regulations, the hydrographs from the pond must match the pre-development hydrographs for the 5, 25, and 100 year events. Through an iterative process using a pond design software, a 24 inch pipe size has been selected in order to maintain the pre-development hydrographs while maintaining the minimum required 1 foot freeboard.

Given a 24-inch diameter outlet pipe, calculate the total grate open area on the overflow outlet trash rack.

1. Using Figure B-53, the ratio of grate open area: total outlet area at a diameter of 24 inches is 4:1

2. The total outlet area for a 24-inch pipe is:

$$A = \pi * \frac{2^2}{4} = 3.14 \text{ square feet (SF)}$$

3. Using the ratio **4:1, 4 *** A_{out} = A_t so **4 * 3.14 = 12-56 SF**

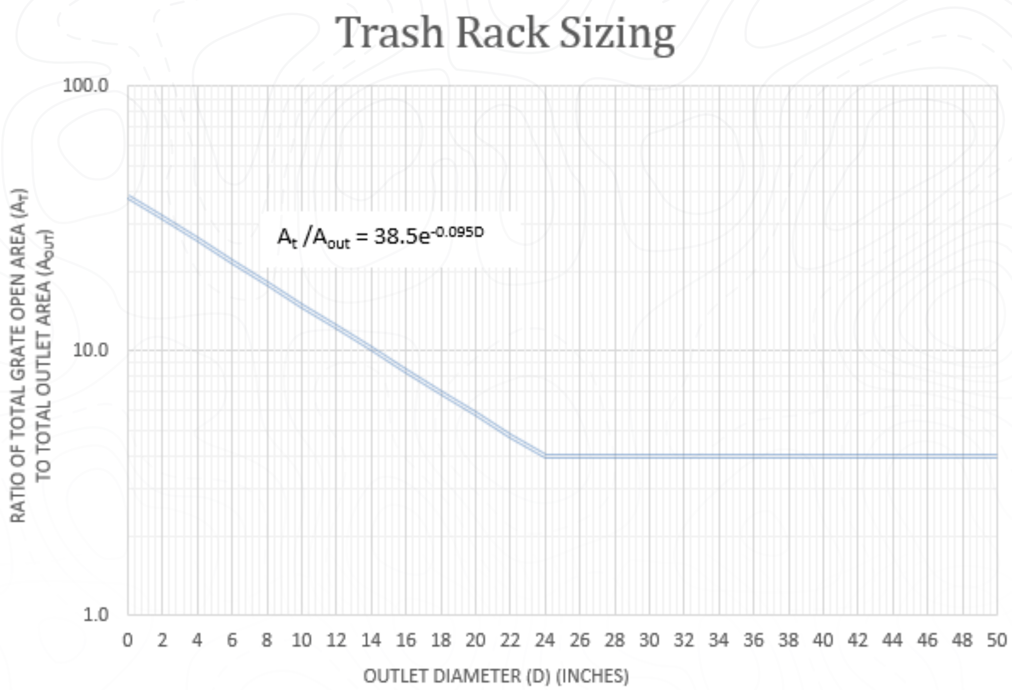

Figure B-53. *Trash rack sizing*

Figure B-54. *(left) Sloped trash rack with parallel wing walls. Source: Urban Drainage Flood Control District*
Figure B-55. *(right) Vertical trash rack with flared wing walls. Source: Urban Drainage Flood Control District*

Outfall and Emergency Spillway
Flared pipe end sections that discharge at or near the stream invert, as shown in Figures B-56 and B-57, are recommended. The channel below the pond outfall should be modified to conform to natural dimensions, and lined with large stone riprap placed over filter cloth. A silting basin may be required to reduce flow velocities from the primary spillway to non-erosive velocities (Barrett, 2005).

Figure B-56 (left). *Emergency spillway. Source: Randy Rath, the Huletts Current*
Figure B-57 (right). *Outfall. Source: Cranberry Township, Pennsylvania*

Minimum freeboard should be provided based on the appropriate local requirements, measured from the top of the water surface elevation for the extreme flood, to the lowest point of the dam embankment not counting the emergency spillway.

An emergency spillway must be included in the extended dry detention basin design to safely pass the design flood control event flood flow using the appropriate local standards. The spillway prevents pond water levels from overtopping the embankment and causing structural damage. The emergency spillway must be located so that downstream structures will not be impacted by spillway discharges.

The goal of designing the emergency spillway is to design a spillway which will not erode with the peak flow rate. The design process is iterative and is often conducted with software that simulates the flow rates of the design storm (most often a 100 year storm). The depth of a broad crested weir is adjusted until the design event is completely held by the weir structure and the velocity of the outflow does not erode the selected rip rap size. Once the depth of the weir is identified, account for the required freeboard. For more information consult local stormwater guidance. Stabilize the emergency spillway with non-erodible materials and provide energy dissipation as necessary.

The outfall should be adequately stabilized with energy-dissipation devices to prevent scour of downstream sediment (for energy dissipation configurations, see Appendix A, Common Design Elements). A pipe collar or other engineering solution should also be installed to prevent seepage of water through the soil along the edge of the pipe (Barrett, 2005). Riprap, plunge pools or pads, or other energy dissipators should be included at the downstream end of the outlet to prevent scouring and erosion. If the basin discharges to a channel with dry weather flow, care should be taken to minimize tree clearing along the downstream channel, and to reestablish a forested riparian zone in the shortest possible distance.

Step 8. Select Vegetation

Vegetation must be established before the basin is considered complete. Established vegetation will provide erosion control and sediment entrapment. Wet tolerant species of diverse native and meadow grasses should be planted within the basin bottom, berms, and side slopes. Avoid planting turf grasses that require routine mowing on the side slopes and basin bottom unless safe access has been designed. Using a mix of native grasses and forbs can eliminate for routine mowing. Vegetation requiring irrigation is not recommended. See Plant List (Appendix E). Fertilizers and pesticides are not recommended. Compaction of soils at the bottom of the basin should be prevented to allow for healthy plant growth and infiltration.

A minimum 25-foot vegetative buffer area should extend away from the top of the pond slope in all directions. Although trees and shrubs can provide habitat, shade, and aesthetic benefits, take care to immediately remove woody vegetation from embankments and buffer to prevent geotechnical failures. Woody vegetation should not be planted or allowed to grow within 15 feet of the toe of the embankment as well.

APPENDIX B: **Additional Extended Detention Basin Design Guidance**

EXTENDED DETENTION BASIN REFERENCES

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Notes:

APPENDIX B: **Additional Cistern Design Guidance**

Cisterns

Figure B-58. *Cisterns at Cliff Morton Development & Business Services Center, San Antonio, TX. Source: Bender Wells Clark Design*

CISTERN DESIGN

The design of a cistern or rain barrel can be broken down into an eight-step process, as listed in Table B-15. Additional resources are provided in TCEQ (2011), Texas Water Development Board (2005), and Texas A&M AgriLife Extension Services (2013).

TABLE B-15. CISTERN ITERATIVE DESIGN STEP PROCESS

Design step		Design component/ consideration	General specification
1	**Determine BMP Size (pg 236)**	Use Appendix J	
2	**Determine BMP Configuration (pg 236)**	Based on volume and desired alternative uses, incorporate next to buildings or underground. A foundation of gravel should be provided if the weight of the cistern at capacity is less than 2,000 pounds, otherwise a concrete foundation should be provided.	
3	**Select and Size Inlet Configuration (pg 237)**	Conveyance type	Runoff should be conveyed to the cistern such that no backwater onto roofs occurs during the 100-year event. Two types of inlet configurations are available: • Dry conveyance: conduit freely drains to cistern with no water storage in pipe • Wet conveyance: a bend in the conduit retains water between rainfall events.

TABLE B-15. CISTERN ITERATIVE DESIGN STEP PROCESS (CONT.)

4	**Design Inlet Pretreatment Configuration (pg 239)**	Inlet filter	A self-cleaning inlet filter should be provided to strain out large debris such as leaves. Some systems incorporate built-in bypass mechanisms to divert high flows.
		First flush diverter	A passive first flush diverter should be incorporated in areas with high pollutant loads to capture the first washoff of sediment, debris, and pollen during a rainfall event. First flush diverters are typically manually dewatered between events.
5	**Select and Design the Outlet and Over-flow/Bypass Method (pg 241)**	Low-flow outlet	An outlet should be designed to dewater the water quality storage volume to a vegetated area in 2 days minimum. The elevation of the outlet depends on the volume of water stored for alternative purposes.
		Overflow or bypass	Emergency overflow (set slightly below the inlet elevation) or bypass must be provided to route water safely out of the cistern when it reaches full capacity.
6	**Specify Cautionary Signage, Pipe Color, and Locking Features (pg 242)**	Signage	Signage indicating: “Caution: Reclaimed Water, Do Not Drink” (preferably in English and Spanish) must be provided anywhere cistern water is piped or outlets.
		Pipe color and locking features	All pipes conveying harvested rainwater should be Pantone color #512 and be labeled as reclaimed water. All valves should feature locking features.
7	**Design for Multi-Use Benefits (Appendix C)**	Harvested rainwater should be used to offset potable water uses, such as irrigation, toilet flushing, car washing, etc. Additionally, educational signage and aesthetically pleasing facades should be specified.	
8	**Additional Design Specifications (Appendix D)**	Vector control	All inlets and outlets to the cistern must be covered with a 1-mm or smaller mesh to prevent mosquito entry/egress
		Routing water for use	Regardless of gravity or pumped flow, adequate measures must be taken to prevent contamination of drinking water supplies
		Cistern material	Tanks should typically be opaque to prevent algal growth.

Step 1. Determine BMP Size

The volume of water to be treated will help managers determine the appropriate cistern size and configuration. Methods for calculating the volume required for treatment are outlined in Appendix J. The treatment volume must be treated on-site and can be treated by multiple BMPs. Cisterns will typically be part of a treatment system that would include cisterns and other BMPs including bioretention or pervious pavement. The cistern could be included to reduce the size of another BMP. Peak runoff flow rates should also be calculated using the methods in Appendix J, San Antonio Unified Development Code, or San Antonio River Basin Regional Modeling Standards for Hydrology and Hydraulic Modeling such that pipes can be sized accordingly to allow overflow or bypass of the 100-year peak discharge.

Step 2. Determine BMP Configuration

Cisterns are available commercially in numerous sizes, shapes, and materials. Many are made to custom fit the available space and can be short and wide, tall and narrow, round, rectangular, and almost any size imaginable. They can be made from multiple materials but are primarily constructed of plastic or metal. Plastic cisterns can be covered with wood facades to provide a more finished appearance or can be painted with any image desired.

Cisterns are usually intended to capture runoff from elevated surfaces, such as rooftops, and, therefore, must be next to structures where runoff can be collected. Cisterns are typically designed to capture runoff from concentrated sources or collection systems such as a downspout. Multiple cisterns placed around a structure can be hydraulically connected to take advantage of maximum storage capacity.

APPENDIX B: **Additional Cistern Design Guidance**

The conditions or layout of the site could determine if the foundation can be excavated and what materials will be used to support the cistern. Cisterns, especially large systems, must have a proper foundation to support the weight when they are at capacity. Two options exist for foundations (Jones and Hunt 2008):

- Cisterns exerting less than 2,000 pounds per square foot: The foundation of the cistern should be cleared and leveled. The foundation should be at least 6 inches of No. 57 gravel or concrete, depending on the stability of the underlying soils.
- Cisterns exerting greater than 2,000 pounds per square foot: The area beneath the cistern should be cleared and leveled. Concrete should be poured such that gravity flow can be maintained and the cistern can be drained to the level of the outlet valve.

The threshold where a concrete pad is required will vary depending on the soil type. If the structural capacity of the site to support a full cistern is in doubt, a geotechnical evaluation should be performed to determine the structural capacity of the soils. Figure B-59 to Figure B-61 shows the foundation options.

Figure B-59 (left). *Cistern less than 2,000 psi on a gravel foundation, New Bern, North Carolina. Source: North Carolina State University Department of Biological and Agricultural Engineering*
Figure B-60 (center). *Cistern greater than 2,000 psi on a concrete foundation, Phil Hardberger Park, San Antonio, Texas.*
Figure B-61 (right). *Construction of a concrete foundation for cistern at San Antonio River Authority Main Office, San Antonio, Texas.*

Step 3. Select and Size Inlet Configuration

Inlet connections can feature either dry conveyance or wet conveyance. The following subsections describe each configuration.

Dry Conveyance
When downspouts freely drain to the cistern without any trapped water, the system uses dry conveyance. Connections can be made through the top of the cistern as shown in Figure B-62 and Figure B-63 or through the sides of the vertical portion formed for the opening of the cistern, often referred to as the manway, as shown in Figure B-64. Inlet connections made through the top of the cisterns can also include a basket filter as an inlet filter option. Inlet connections through the sides with the proper gaskets are recommended for ease of maintenance and access to the cistern.

When designing dry conveyance, downspout pipes should be sized to convey the 100-year discharge without causing any backwater on the roof.

APPENDIX B: **Additional Cistern Design Guidance**

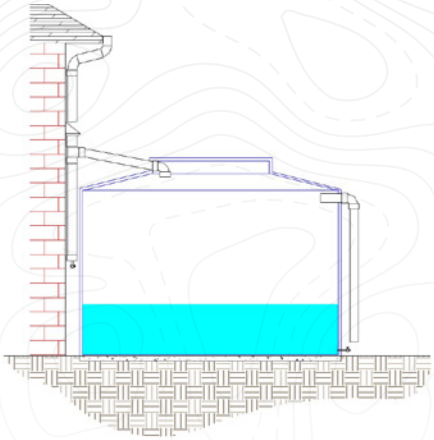


Figure B-62 (left). *Dry conveyance inlet configuration*
Figure B-63 (center). *Inlet in the top of the cistern at Texas A&M University at San Antonio, San Antonio, Texas.*
Figure B-64 (right). *Inlet in the sides of the man way, Greensboro, North Carolina. Source: Tetra Tech*

Wet Conveyance
When the downspout features a bend, causing water to be trapped between runoff events, this system is known as wet conveyance (Figure B-65 and Figure B-66.). Wet conveyance systems with buried downspouts can allow for cisterns to be placed further from buildings and might be preferable for aesthetic or overhead clearance purposes. When designing wet conveyance systems, the 100-year discharge from the catchment must be conveyed without any backwater onto the rooftop (considering all head losses through the pipe). Because water will permanently be stored in the downspout, watertight connections must be used to prevent leakage. A drain at the lowest elevation of the downspout can be installed, if desired, for dewatering and emergency maintenance.

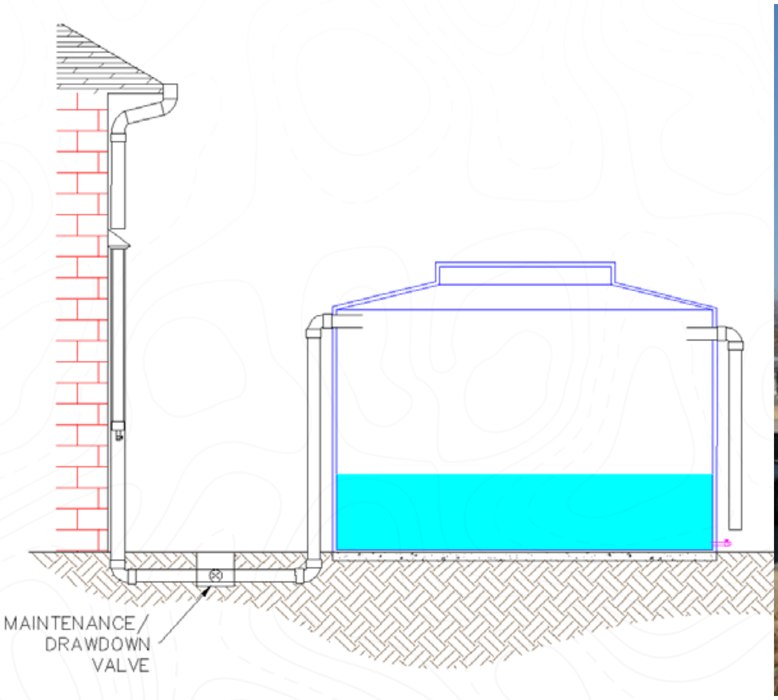


Figure B-65 (left). *Cistern with wet conveyance featuring a drawdown valve for maintenance*
Figure B-66 (right). *Cistern with a wet conveyance inlet configuration, Dallas, Texas. Source: North Carolina State University Department of Biological and Agricultural Engineering*

Step 4. Design Inlet Pretreatment Configuration

Stormwater runoff must be filtered before it enters the cistern to remove debris and particles that could clog the outlet. Two types of systems can be used: inlet filters and first-flush diverters. The following subsections discuss each pretreatment configuration in greater detail.

Inlet Filters

Inlet filters are designed to remove particles as runoff passes through the filters before entering the cistern; many filter options are available. The size and type of filter used will depend on the size of the area draining to the downspout. The filters can be installed at the gutter as shown in Figure B-67 or at the end of the downspout as shown in Figure B-68 depending on the configuration of the downspouts. Flow through filters that force all the runoff through the filter can be used for smaller drainage areas (less than 1,500 square feet). Filters capable of bypassing larger event flow could be required for larger drainage areas (1,500 to 3,000 square feet). A self-cleaning screen used for inlet filters should provide a minimum angle of declination of at least 45 degrees from horizontal, but angles of more than 45 degrees tend to enhance self-cleaning and prevent clogging (Nel 1996). Examples of two types of filters are shown in Figure B-69 and Figure B-70.

Figure B-67 (left). *Inlet filter at the gutter at San Antonio River Authority Main Office, San Antonio, Texas. Figure B-9-12. Flow through inlet filter. Source: Tetra Tech*

Figure B-68 (right). *Inlet configuration at the downspout at San Antonio River Authority Main Office, San Antonio, Texas.*

Figure B-69 (left). *Flow-through inlet filter. Source: Tetra Tech*
Figure B-70 (right). *Self-flushing filter with a bypass. Source: Tetra Tech*

First Flush Diverter

First-flush diverters can be installed after the inlet filter and are designed to divert an initial volume of water away from the cistern to prevent small particles—initially washed off of the roof—from clogging the outlet. First-flush diverters are typically attached to the inlet or, in some cases, the inlet filter with a 4- to 6-inch diameter pipe with a small relief valve from which water can be diverted. The volume of water diverted away from the cistern depends on the length of the pipe. Once the diverter is full, a valve closes and water flows into the cistern. A first-flush diverter is not always required and inclusion is up to the designer depending on site conditions. A first-flush diverter is recommended for sites where pollen or other fine particles might not be removed by an inlet filter. Diverters must be routinely drained to provide capacity for the next runoff event.

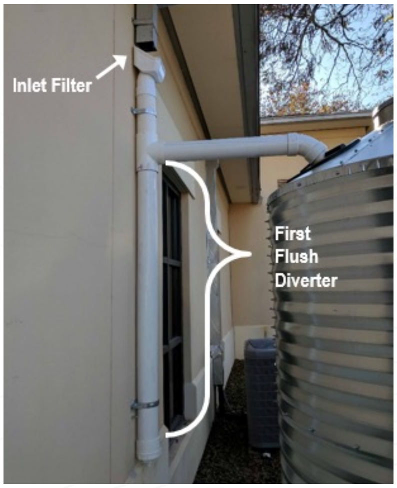


Figure B-71 (left). *Valve for a first-flush diverter. Source: North Carolina State University Department of Biological and Agricultural Engineering*
Figure B-72 (right). *First-flush diverter configuration at the downspout at San Antonio River Authority Main Office, San Antonio, Texas.*

Step 5. Select and Design the Outlet and Overflow/Bypass Method

Low Flow Outlet

The outlet of the cistern should be designed to release the volume of captured runoff at a rate below the design storm rate at its maximum capacity. The outlet of the cistern should be directed to a bioretention area or other pervious surface with enhanced infiltration capacity as demonstrated in Figure B-73. Irrigation area requirements for the Edwards Aquifer Recharge, Contributing, and Transition Zones are presented in Table B-16; these requirements are applicable to all areas.

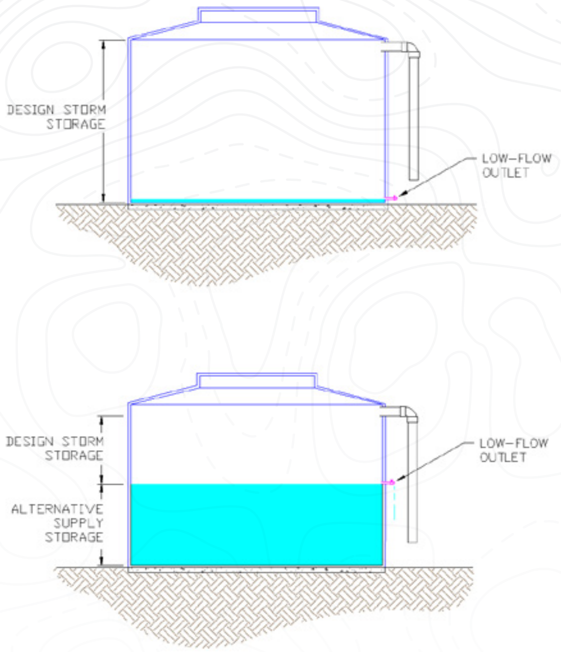
Figure B-73. *Cistern outlet into a planter box in San Diego, California. Source: Tetra Tech*

TABLE B-16. IRRIGATION AREA REQUIREMENTS FOR CISTERNS IN THE EDWARDS AQUIFER RECHARGE, CONTRIBUTING, AND TRANSITION ZONES (APPLICABLE TO ALL AREAS)

Irrigation/infiltration area requirements
• 12 inches of soil cover, according to geotechnical investigation • 100 feet from wells, septic systems, natural wetlands, and streams • No sensitive or geologic features that could allow water to directly enter the aquifer • Coarse soil material (diameter greater than 0.5 inch) does not make up more than 30% of the soil volume • Slopes less than 10% • Soil permeability and surface area sufficient to produce no runoff
Source: Barrett 2005

The elevation of the low-flow outlet depends on the demand for alternative water use. When water demand and use is high (such as when the cistern is being used for toilet flushing, car washing, or consistent irrigation), the low-flow outlet can be placed such that half of the tank remains full for use. If stormwater management is the sole purpose of the cistern, the low-flow outlet should be placed at the bottom so that the tank can dewater and provide maximum capacity for storage of subsequent rain events. Figure B-74 illustrates example low flow outlet placement. Regardless of where the outlet is placed, temporary storage must be provided above the outlet elevation to capture the design storm volume. Models, such as the Rainwater Harvester Design Model (North Carolina State University 2008), can be used to optimize orifice placement.

APPENDIX B: **Additional Cistern Design Guidance**



Overflow or Bypass

All cisterns should have an overflow for runoff volumes that exceed the capacity of the cistern. The overflow should be set slightly below the inlet. Overflow connections should be connected to the tank using appropriate watertight gaskets. An additional bypass can be incorporated using an appropriate inlet filter. Examples of an overflow discharging to vegetated areas are provided in Figure B-75.

Figure B-74 (left) . *Top: a flow outlet is placed to provide equal parts detention storage and storage for alternative use. Bottom: placing the low flow outlet at the bottom of the cistern ensures maximum design storm storage*

Figure B-75 (right). *A concrete channel (left) directs overflow away from the building at the Shavano Park Fire Station, Shavano Park, Texas; and a cistern overflows to an adjacent bioretention area lined with cobble (right) at Mission Library, San Antonio, Texas. Source: Bender Wells Clark Design*

All overflow and outlet volumes should be directed safely away from all structural foundations and any areas where infiltration could have an adverse effect. Overflow and bypass mechanisms should be sized to safely convey the 100-yr discharge without any backwater onto the adjacent roof. Calculation of 100-yr conveyance should account for head losses through all pipe sections, elbows, entrances, and exits.

Step 6. Specify Cautionary Signage, Pipe Color, and Locking Features

Refer to the International Plumbing Code and local amendments on the following:

- Clear and obvious signage wherever harvested rainwater is used, including entrances to rooms and where harvested water is piped or used, irrigation and automobile washing hoses, low-flow outlet orifices, toilet tanks that use harvested water for flushing, and any spigots, drawdown pipes, or access hatches, and cistern manways restricting access.
- Pipe color indicates that the water is not safe to drink.
- Locking features for valves.

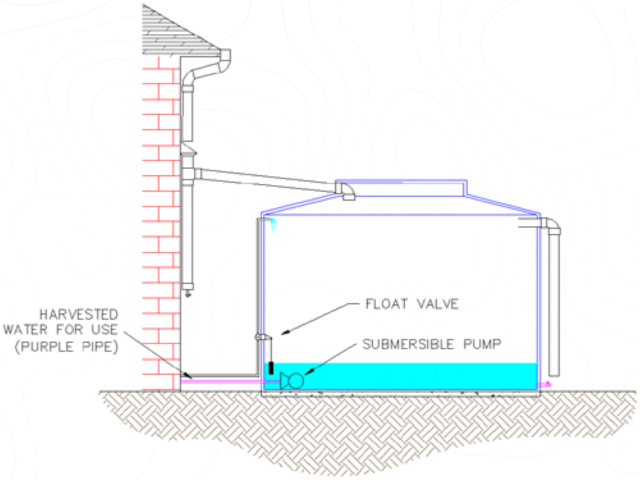
Step 7. Additional General Design Guidance

The following considerations relevant to safety and water reuse should be included in design plan notes and specifications.

Vector Control
The inlets and outlets of cisterns and rain barrels should be covered with a simple piece of filter material, such as a screen or wire mesh, to prevent mosquito breeding. A 1 mm or smaller mesh is recommended. Screens at the inlet should be placed downstream of debris filters to prevent clogging by leaves. Overflow/bypass openings should be covered with a non-clogging configuration, such as a screen mesh flap that hangs across the pipe opening—the bottom of the flap should be weighted or attached with small magnets such that it remains closed when no flow is present, but can easily open to allow overflow when the tank is full.

APPENDIX B: **Additional Cistern Design Guidance**

Figure B-76. Two cisterns with purple pipe connection at Phil Hardberger Park, San Antonio, Texas.



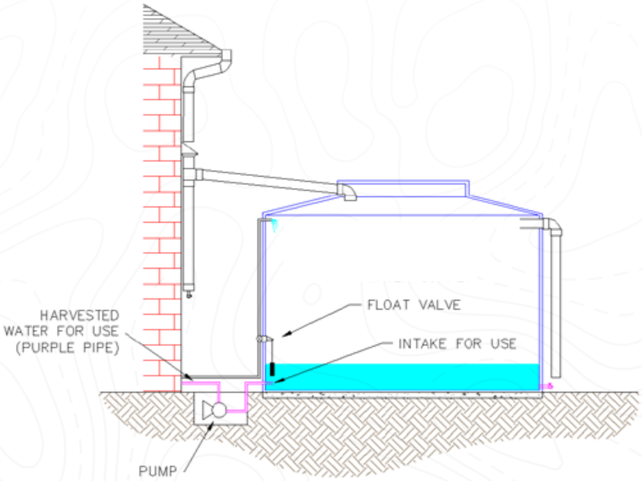


Figure B-77: Top: Conceptual schematic of cistern with submersible pump. Bottom: conceptual schematic of cistern with external pump.

Routing Water for Use

The method of routing water depends on the intended use. For basic irrigation, gravity can often be used to route harvested rainwater to nearby vegetation beds or infiltrating stormwater practices. To route water for use inside nonresidential structures or for greater distances from the cistern, a pump might be required. Submersible water pumps are commonly used, but pumps can also be installed in utility boxes next to the cistern. Pipes conveying harvested water may not be placed in the same trench as potable water pipes, a 2-foot horizontal separation must be maintained between harvested and potable water at all times. Buried potable water pipes that cross harvested water pipes must be at least 12 inches above the harvested water and must have a PVC sleeve that extends horizontally 2 feet to either side of the crossing. Harvested water should also be protected from contamination by sewer pipes in the same manner as potable water pipes. Refer to the International Plumbing Code and local amendments.

Cistern Material Specification

Rainwater harvesting tanks are typically constructed of plastic, metal, or concrete. The specified material will affect the quality of captured runoff, aesthetics, configuration, installation, and cost. Plastic tanks can experience algal growth if not completely opaque. In general, cisterns are expected to last 20 to 50 years (Kowalsky and Thomason 2011). A detailed description of cistern materials is provided in Texas Water Development Board (2005).

APPENDIX B: **Additional Cistern Design Guidance**

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APPENDIX B: Additional Vegetated Swale Design Guidance

Vegetated Swales

VEGETATED SWALE DESIGN

Vegetated swales can be used as pretreatment for other stormwater BMPs or in a treatment train, but they should not typically be installed as standalone practices for water quality improvement. For water quality swale design, see Bioswales. The design of vegetated swales can be broken down to an eight-step process (summarized in Table B-17).

TABLE B-17. VEGETATED SWALE ITERATIVE DESIGN STEP PROCESS

Design step		Design component/ consideration	General specification
1	**Determine Design Flows (pg 245)**	Use Appendix J and local guidelines	
2	**Adjust Preliminary Swale Layout to Fit Site (pg 245)**	Swale dimensions	Determine allowable swale dimensions per site constraints
3	**Calculate Swale Cross Sectional Dimensions (pg 245)**	Bottom width, side slopes, and longitudinal slope	Design flow depth should not exceed two-thirds the height of vegetation for optimum pretreatment
4	**Determine Water Quality Design Flow Velocity (pg 246)**	Design velocity	Velocity should be less than 1 ft/s to reduce risk of erosion
5	**Calculate Swale Length (pg 247)**	Residence time	If designed for water quality improvement, the hydraulic residence time should be at least 10 minutes to promote sedimentation
6	**Provide Conveyance Capacity for Flows Higher than the Design Storm (pg 247)**	25-year, 24-hour storm	The 25-year, 24-hour storm should be conveyed at less than 3 ft/s to prevent erosion
7	**Determine if Soils Need to be Amended (pg 247)**	If additional water quality improvement and infiltration are desired, amend the soil with minimum 2 inches of soil media (for media standards, see bioretention)	
8	**Select Vegetation (pg 248)**	Native, noninvasive turf grasses (not bunch grasses) should be planted and maintained at a minimum height of 4 inches (see Appendix E)	

Step 1. Determine Design Flows

Swales are conveyance, flow-based BMPs, so treatment is based on a water quality design flow. The flow associated with the water quality design storm should be calculated based on information provided in Appendix J, San Antonio Unified Development Code, or San Antonio River Basin Regional Modeling Standards for Hydrology and Hydraulic Modeling. In addition to the water quality design flow, the vegetated swale should be designed to safely convey the 25-year storm event unless a diversion structure is installed to allow only the water quality flow into the swale (for details on diversion structures, see Appendix A, Common Design Elements).

Step 2. Adjust Preliminary Swale Layout to Fit Site

Vegetated swales can be in many different areas, particularly road sides, around parking lots, medians, and open areas. The linear structure of swales favors their use in the treatment of runoff from highways, residential roadways and common areas in residential subdivisions, along property boundaries and in and around parking lots. If permitted, vegetated swales are an excellent alternative to curbs and gutters, providing water quality and quantity benefits and adding an aesthetic appeal. Generally, a vegetated filter strip or buffer should be placed between the roadway and the vegetated swale to limit the amount of sediment entering the swale.

Step 3. Calculate Swale Cross Sectional Dimensions

The flow capacity of a vegetated swale is a function of the longitudinal slope (parallel to flow), the resistance to flow (e.g., Manning's roughness), and the cross-sectional area. The cross section is normally approximately trapezoidal, and the area is a function of the bottom width and side slopes. The flow capacity of vegetated swales should be such that the design water quality flow rate will not exceed a flow depth of two-thirds the height of the vegetation in the swale or 4 inches at the peak of the water quality design storm intensity.

The design procedure detailed below uses an iterative method for solving Manning's equation for a trapezoidal, open channel when the longitudinal channel slope, Manning's roughness, and design flow rate are known. The general Manning's equation is as follows, assuming the design flow rate is $\mathbf{Q_{wq}}$:

[Equation B-10-1]

$$Q_{wq}=\left(\frac{1.49}{n}\right)AR^{\frac{2}{3}}s^{\frac{1}{2}}$$

where:

Q_{wq} = design storm flow rate (cfs)
n = Manning's roughness coefficient (no units)
A = cross-sectional area of flow (ft^2)
R = hydraulic radius (ft) = area (A) divided by wetted perimeter (P)
P = wetted perimeter, the perimeter that is in contact with the swale during the design flow
s = longitudinal channel slope (along direction of flow) (ft/ft)

For the purposes of the trial and error process presented below, Manning's equation can be rearranged as follows (Barrett 2005):

[Equation B-10-2]

$$b=\left(\frac{0.134Q}{y^{1.67}s^{0-5}}\right)-zy$$

where:

b = swale bottom width (ft)
y = depth of flow (ft)
z = side slope of swale in the form **z:1** (should not exceed 3)

An iterative process is best used to determine the depth of flow, y, bottom width, b, and side slope, z. Trial values of bottom width, flow depth, and side slope should be used to determine A, P, and R for the swale's cross section until the equations are equal and the flow depth, bottom width, and channel side slope are within the guidelines established in the previous sections. The equations for A and R for a trapezoidal channel are provided below:

[Equation B-10-3]

$$R=\frac{A}{P}$$

[Equation B-10-4]

$$A=(b=zd)d$$

[Equation B-10-5]

$$P=b+2d(1+z^2)^{0-5}$$

Although slope is often determine by site conditions, the slope should not exceed 2% for optimum water quality performance. Check dams can be used to reduce the effective slope of a swale (see Bioswales and Barrett 2005 for check dam design information). While not required, spreadsheet or computer-based models with "goal seek" functions can assist with this analysis.

Step 4. Determine Water Quality Design Flow Velocity

The flow continuity equation should be used to calculate the design flow velocity through the swale:

[Equation B-10-6]

$$V_{wq} = \frac{Q_{wq}}{A_{wq}}$$

where:

Q_{wq} = design flow (ft3/sec)

V_{wq} = design flow velocity (ft/sec)

A = (b + zd)d = cross-sectional area (ft^2) of flow at the design depth,

where:

z = side slope length per unit height with a maximum slope of 3:1.

The swale should convey the design storm without the threat of erosion. If the design flow velocity exceeds 1 ft/sec, one or more of the design parameters (longitudinal slope, bottom width, or flow depth) must be altered to reduce the design flow velocity to 1 ft/sec or less. It is desirable to have the design velocity as low as possible, both to improve treatment effectiveness and to reduce swale length requirements.

Step 5. Calculate Swale Length

The residence time in a swale should be at least 10 minutes to optimize pretreatment and sediment removal, although this is not always feasible given certain site constraints. Use the following equation to determine the necessary swale length to achieve a hydraulic residence time of at least 10 minutes (600 seconds):

[Equation B-10-7]

$$L = 600v_{wq}$$

where:

L = swale length (ft)

V_{wq} = design flow velocity (ft/sec)

If the swale is too long to fit in the site, the design parameters can be adjusted to provide the flow velocity required to meet the recommended residence time. Additionally, a sinuous pattern can be used to increase total swale length (and decrease bed slope) over a distance.

Step 6. Provide Conveyance Capacity for Flows Higher than the Design Storm

Vegetated swales are often designed as online systems that convey flows higher than the design storm flow but can be designed as offline systems incorporating a high-flow bypass or diversion structure upstream of the swale inlet. A high-flow bypass usually results in a smaller swale size. If a high-flow bypass is required, see details on designing diversion structures in Appendix A, Common Design Elements.

If the swale will be designed as an online system, confirm that the swale can convey the post-development peak stormwater discharge rate for the 25-year, 24-hour storm event (or local surrogate). The post-development peak stormwater runoff velocity for the 25-year, 24-hour storm should be less than 3.0 ft/sec. If the 25-year, 24-hour peak flow velocity exceeds 3.0 ft/sec, increase the bottom width or reduce the longitudinal slope as necessary to reduce the peak flow velocity to 3.0 ft/sec or less. If the longitudinal slope is reduced, the swale bottom width must be recalculated and must meet all guidelines established in the previous section.

APPENDIX B: **Additional Vegetated Swale Design Guidance**

Step 7. Determine if Soils Need to be Amended

If enhanced infiltration and water holding capacity is desired, vegetated swale soils may be amended with 2 inches of soil media (for soil media specifications, see bioretention) unless the organic content is already greater than 5 percent. The soil media should be mixed into the native soils to a depth of 6 inches to prevent soil layering.

Step 8. Select Vegetation

Swales must be vegetated to provide adequate treatment of runoff via filtration. Vegetation, when chosen and maintained appropriately, also improves the aesthetics of a site. It is important to maximize water contact with vegetation and the soil surface. The following criteria should be used for selecting appropriate vegetation:

1 The swale area must be appropriately vegetated with a mix of erosion-resistant plant species that effectively bind the soil. A diverse selection of low-growing plants that thrive under the specific site, climatic, and watering conditions should be specified. A mixture of dry-area and wet-area grass species that can continue to grow through silt deposits is most effective. Native or adapted grasses are preferred because they generally require less fertilizer, limited maintenance, and are more drought resistant than exotic plants. When appropriate, swales that are integrated in a project can use turf or other more intensive landscaping, while swales that are on the project perimeter, in a park, or close to an open space area should be planted with a more naturalistic plant palette. Vegetation in the swale must be rooted before the wet season. If vegetation cannot be rooted in time, turf should be installed and properly stabilized.

2 Trees or shrubs can be used along the banks as long as they do not over-shade the turf—woody vegetation should generally be avoided in the bottom of the swale to prevent increased velocities as water flows around the trunks.

3 Above the design treatment elevation, a typical lawn mix or landscape plants can be used, provided they do not shade the swale vegetation.

4 Temporary irrigation is required if the seed is planted in spring or summer. Seed should be properly stabilized with straw or equivalent mulch. Drought-tolerant grasses should be specified to minimize irrigation requirements.

5 Sod is the most effective and efficient way to vegetate swales; ensure that sod remains adequately irrigated during establishment. Sod should be laid perpendicular to flow and staggered such that no preferential flow paths are created by the seams between sod rolls. To maximize incidental infiltration, sod should be sourced from facilities that do not grow sod in clay soils. Washed sod can be also be furnished if desired.

6 Vegetative cover should be planted and maintained at a minimum height of 4 inches. Swale water depth will ideally be 2 inches below the height of the shortest plant species. See Plant List (Appendix E).

APPENDIX B: Additional Vegetated Filter Strip Design Guidance

Vegetated Filter Strips

VEGETATED FILTER STRIP DESIGN

The primary function of vegetated filter strips is to maintain sheet flow of runoff for pretreatment and energy dissipation. The steps for designing vegetated filter strips are provided in Table B-18 below.

TABLE B-18. VEGETATED FILTER STRIP ITERATIVE DESIGN STEP PROCESS

Design step		Design component/ consideration	General specification
1	**Determine the Design Flow Rate (pg 249)**	Use Appendix J and local guidelines	
2	**Determine Available Filter Strip Width and Slope (pg 249)**	Based on existing site conditions	
3	**Determine Vegetative Cover (pg 249)**	Vegetation	Native, drought-tolerant turf grasses (not bunch grasses) should be maintained at a height of no less than 4 inches. See Appendix E.
4	**Calculate the Design Flow Depth (pg 250)**	Design flow depth	Flow depth should be less than 1 inch to achieve effective water quality improvement
5	**Calculate the Design Velocity (pg 250)**	Design velocity	Velocity should be less than 1 ft/s for the water quality event and less than 3 ft/s for the 25-year, 24-hour event
6	**Calculate the Length of the Filter Strip (pg 250)**	Length and residence time	Filter strip length should provide for a 10-minute hydraulic residence time if substantial water quality improvement is desired.
7	**Design the Level Spreader/ Energy Dissipater if Needed (pg 250)**	Level spreader	A level spreader and energy dissipater must be designed if concentrated flows are present.
8	**Determine if Soils Need Amending (pg 251)**	If additional water quality improvement and infiltration are desired, amend the soil with 2 inches of media (for media standards, see bioretention)	
9	**Specify Signage (pg 251)**	Signage should identify filter strip as stormwater treatment practice and prohibit foot traffic and other activities that could compact or rut filter strip soils.	

Step 1. Determine the Design Flow Rate

Vegetated filter strips are conveyance, flow-based BMPs, so treatment is based on a water quality design flow. The flow associated with the water quality design storm should be calculated according to information provided in Appendix J, San Antonio Unified Development Code, or San Antonio River Basin Regional Modeling Standards for Hydrology and Hydraulic Modeling. In addition to the water quality design flow, the filter strip should be designed to safely convey the 25-year storm event.

Step 2. Determine Available Filter Strip Width and Slope

In some design cases, the filter strip width and slope are predetermined on the basis of existing conditions. However, in many cases, determining the final width and slope are part of the design process.

Step 3. Determine Vegetative Cover

Select vegetative cover for the filter strip that is appropriate for local soil and climate conditions. Considerations should include requirements for maintenance, irrigation, and fertilization. See vegetated swale vegetation specifications.

Step 4. Calculate the Design Flow Depth

Hydraulically, filter strips should be designed according to two primary criteria: maximum depth of flow and maximum flow velocity.

Depth of runoff flow generated by the design storm in the filter strip should be limited to less than or equal to 1 inch. The design configuration having the greatest effect on those design standards are the contributing watershed area, longitudinal slope (along the direction of flow), the resistance to flow (Manning's n), and the width and slope of the filter strip. The design flow depth (d) is calculated on the basis of the width and the slope (parallel to the flow path) using a modified Manning's equation as follows:

[Equation B-11-1]

$$d=\left(\frac{Q_{wq}\times n_{wq}}{1.49ws^{0-5}}\right)^{0.6}$$

where:

d	= design flow depth (ft)
Q_{wq}	= water quality design flow rate (cfs)
w	= width of strip perpendicular to flow that equals the width of impervious surface contributing to the filter strip (ft)
s	= slope (ft/ft) of strip parallel to flow, average over the whole width
n_{wq}	= Manning's roughness coefficient (0.025–0.03)

If d is greater than 1 inch, a smaller slope is required, or the filter strip may not provide substantial water quality improvement.

Step 5. Calculate the Design Velocity

Maximum design storm flow velocity should be limited to 1 ft/sec. The design flow velocity is based on the design flow, design flow depth, and width of the strip as follows:

[Equation B-11-2]

$$v_{wq}=\frac{Q_{wq}}{dw}$$

where:

v_{wq}	= water quality design flow velocity (ft/sec)
Q_{wq}	= water quality design flow rate (cfs)
d	= design flow depth (ft)
w	= width of strip perpendicular to flow that equals the width of impervious surface contributing to the filter strip (ft)

Step 6. Calculate the Length of the Filter Strip

Determine the required length (L) to achieve a desired residence time of 10 minutes using this equation:

[Equation B-11-3]

$$L = 600v_{wq}$$

where:

L	= swale length (ft)
v_{wq}	= design water quality flow velocity (ft/sec)

If the design parameters as computed in steps 1 through 6 above are not within the recommended standards, an alternative BMP, such as a grassed swale should be considered to treat stormwater runoff.

APPENDIX B: **Additional Vegetated Filter Strip Design Guidance**

Step 7. Design the Level Spreader/Energy Dissipater if Needed

The transition of stormwater runoff from upslope, impervious areas to the gently sloping, vegetated surface of a filter strip is critical to the proper function of the BMP. Flow should not be concentrated and should not transition to flow over the filter strip such that it causes concentration or erosive flows. Where flow originates on roadways and parking lots, the designer can elect to incorporate an energy-dissipation device at the interface between the hardened pavement surface and the filter strip. Energy dissipaters typically take the form of a gravel flow spreader consisting of a gravel filled trench that is perpendicular to the direction of flow.

The gravel flow spreader should have the following characteristics:

- The gravel flow spreader must be a minimum of 6 inches deep and 12 inches wide.
- The gravel surface should be a minimum of 1 inch below the surface of the adjacent pavement.

Vegetated filter strips are often used in combination with concrete level spreaders to provide energy dissipation.

Step 8. Determine if Soils Need Amending

If enhanced infiltration is desired, vegetated filter strips can be amended with 2 inches of soil media (for soil media specifications, see bioretention design chapter) or plant-derived compost unless the organic content is already greater than 5 percent. The amendment should be mixed into the native soils to a depth of 6 inches to prevent soil layering.

Step 9. Specify Signage

It is important to specify installation of signage so that the vegetated filter strip is properly maintained. Signage should label the practice as a stormwater BMP, prohibit foot traffic, and instruct maintenance crews to maintain vegetation at a height of approximately 4 inches—this will ensure maximum treatment and soil stabilization.

VEGETATED FILTER STRIP REFERENCES

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Notes:

APPENDIX B: **Additional Treatment Train Design Guidance**

Optimal Treatment Train Approach

There are four progressive levels of treatment in the optimal treatment train. Each of the four levels of treatment progressively enhances the treatment of stormwater. The first level (level 1) is to maximize runoff capture. The second level (level 2) is pre-treatment and conveyance with the option of including green roof or rainwater harvesting. The third level (level 3) is lot level BMPs that are intended to capture smaller drainage areas closer to the source of runoff (impervious cover) and are integrated into the development. The development level BMPs (level 4) are intended to treat larger drainage areas and are often larger BMPs that can be designed to also address water quantity. Additional information about each of the levels is provided below.

In addition, there is a preferred order for types of major functions. The first is settling and sedimentation to remove large sediment and debris. Next is filtration and infiltration to remove smaller suspended materials. The final type of function is biological and chemical treatment to treat dissolved pollutants. It's important to note that maximum treatment levels may not be achievable in all site conditions. If a treatment level is skipped, it may still be considered an acceptable treatment train; however, all levels of treatment should be considered prior to eliminating levels of treatment.

Level 1: Maximize Capture of Runoff

Minimize total and effective impervious area, maximize capture of stormwater runoff, and maximize treatment of pollutant load in specific lot level and development level BMPs.

This level of treatment should consider conserving native areas to allow for stormwater runoff to infiltrate into vegetated areas prior to the treatment train system. All remaining runoff will be addressed by the treatment train.

Level 2: Pre-Treatment & Conveyance (or Rainwater Harvesting)

The next step for treatment can be a green roof, a rainwater harvesting BMP, or a pre-treatment BMP. Green roofs also have the ability to be connected with a rainwater harvesting BMP, including cisterns and rain barrels. During planning, when considering rainwater harvesting systems, consider the potential volume limitations and the impact on the treatment train system.

Pre-treatment and conveyance are primarily used as an initial step to remove debris and sediment and enhance the performance of downstream BMPs. Pre-treatment BMPs include vegetated filter strips and vegetated swales. These are not standalone BMPs and should be combined with a bioswale, lot level, or development level BMP. The BMPs that should be used for conveyance of stormwater include vegetated swales or bioswales. An additional pre-treatment option would be to include a sediment forebay in combination with a lot level or development level BMP to help remove debris and sediments.

Level 3: Lot Level BMP

Lot level BMPs typically serve smaller drainage areas and receive runoff from nearby impervious areas (e.g. parking lots, roadways, sidewalks). Infiltration and filtration BMPs are often used as lot level BMPs to remove suspended solids and sediment associated pollutants and reduce runoff volumes and peak flows near the source. These BMPs may also allow for biological and chemical treatment to remove dissolved or suspended pollutants. They are designed to collect and treat flows from the water quality event or frequent storm events. A lot level BMP typically provides flexibility for the site layout to fit into areas that may have limited space available. Multiple lot level BMPs may be necessary to collect the desired stormwater runoff and may be placed in series or parallel depending on stormwater treatment requirements or other site conditions. If this level is skipped because the lot level BMP cannot be placed within the development, then pre-treatment and conveyance (level 2) may be discharged directly into a development level BMP (level 4).

Level 4: Development Level BMP (or Rainwater Harvesting)

Development level BMPs typically have larger tributary drainage areas and can also be used to mitigate water quantity. A development level BMP may collect stormwater from the source or be routed from lot level BMPs. Stormwater is typically detained in a development level BMP facility where it may also undergo biological and chemical treatment prior to discharging into a receiving water body. Additionally, if a development level BMP is determined to not be necessary due to sufficient pollutant removal from lot level BMPs or is not feasible for a site, due to size requirements or other site conditions, the lot level BMPs (level 3) may be discharged directly into a receiving water body with appropriate discharge mitigation. Runoff from lot level BMPs may also be collected and stored for rainwater harvesting then reused.

Figure B-78 includes an example rendering for a treatment train system in a commercial setting. There are two types of BMPs that function as the initial BMPs in the treatment train system, the green roofs and permeable pavement. Each building in the example has a green roof (level 2). The parking lot has areas with permeable pavement (level 3) that does not receive flow from the green roof. The green roof and permeable pavement function as a first step for treatment and should be sized based on the drainage area or considering limitations of the available footprint. For several of the buildings, the discharge from the green roof mixes with stormwater from other surfaces and is conveyed into the vegetated swale (vegetated conveyance) and ultimately a sand filter (level 4). For another building, discharge from the green roof mixes with flow from other areas and flows directly to the sand filter. Again, this shows some of the flexibility of the treatment train. The permeable pavers also discharge to the vegetated swale and then the sand filter. As in the first rendering, the treatment train system provides a lot of flexibility for how the BMPs are sized and located. Although each BMP may not treat the entire water quality volume of its drainage area, subsequent BMPs in the series can treat additional runoff to help meet the water quality runoff for the site overall.

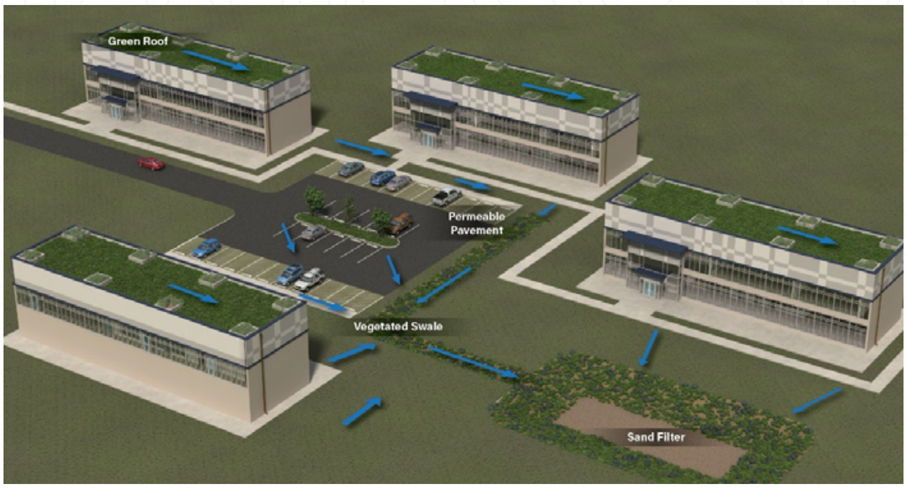

Figure B-78. *Example of commercial treatment train*

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