2I-5 Bioswales (Numbering pending)

Source: Steve Anderson, 2011

BENEFITS Low = <30% Medium = 30-65% High = 65-100%			
	Low	Med	High
Suspended Solids			■
Nitrogen	■	■	
Phosphorous	■		
Metals			■
Bacteriological	*	*	*
Hydrocarbons	*	*	*

* Insufficient Data

Description: Bioswales are essentially bioretention cells designed with positive grade, passing flows from small frequent storms at slow velocities to promote filtration through vegetation and infiltration into constructed soil media layers. A bioswale system consists of an open conveyance channel with a filter bed of permeable soil overlaying a perforated pipe underdrain system. Flow passes into and is detained in the main portion of the channel, where it is filtered through the soil bed. Infiltrated runoff is collected and conveyed by a perforated pipe and gravel underdrain system to the outlet. Bioswales can also be designed to effectively convey flow from larger storms at moderate velocities (to prevent erosion and re-suspension of captured pollutants).

Runoff from the WQv event is temporarily retained in a pool or series of pools created by permanent check dams. The holding time provides an opportunity for sedimentation of particulates and facilitates infiltration of runoff.

Typical uses:
- Manage runoff from residential sites, parking areas, and along perimeter of paved roadways.
- Located in a drainage easement at the rear or side of residential parcels.
- Road shoulder rights-of-way; used adjacent to paved roadways in place of curb and gutter, or used as a conveyance channel on the back side of curb-cut openings.

Advantages/benefits:
- Cost saving alternative to underground pipe systems.
- Mitigate runoff from impervious surfaces.
- Remove sediment and pollutants to improve water quality.
- Reduce runoff rate and volume in highly impervious areas; reduce runoff velocity.
- Provide for groundwater recharge if design and site soils provide sufficient infiltration.
- Good option for small area retrofits – replacing existing drainage ditches.
- Good retrofit opportunities for residential or institutional areas of low to moderate density.
- Linear configuration works well with highway or residential street applications.

Disadvantages/limitations:
- Sediment/pollutant removal sensitive to proper design of slope and vegetation density.
- Caution should be used when a high water table is present; an evaluation should be made for potential groundwater contamination.
- Difficult for use on steep slopes (>6%).
- Higher surface maintenance than curb and gutter systems.

Maintenance requirements:
- Need routine landscape maintenance; maintain grass height of approximately 4 to 6 inches.
- Inspect annually for erosion problems; remove accumulated trash and debris.
- Remove sediment from forebay and channel (if necessary).

A. Overview

1. **Description.** Bioswales are conveyance channels engineered to capture and treat the water quality volume for a drainage area. They differ from a normal drainage channel or swale through the incorporation of specific features that enhance stormwater pollutant removal effectiveness. They are designed with limited longitudinal slopes to force the flow to be slow and shallow, thus allowing for particulates to settle and limiting the effects of erosion. Berms and/or check dams installed perpendicular to the flow path promote settling and infiltration. The following description provided by Claytor and Schuler (1996):

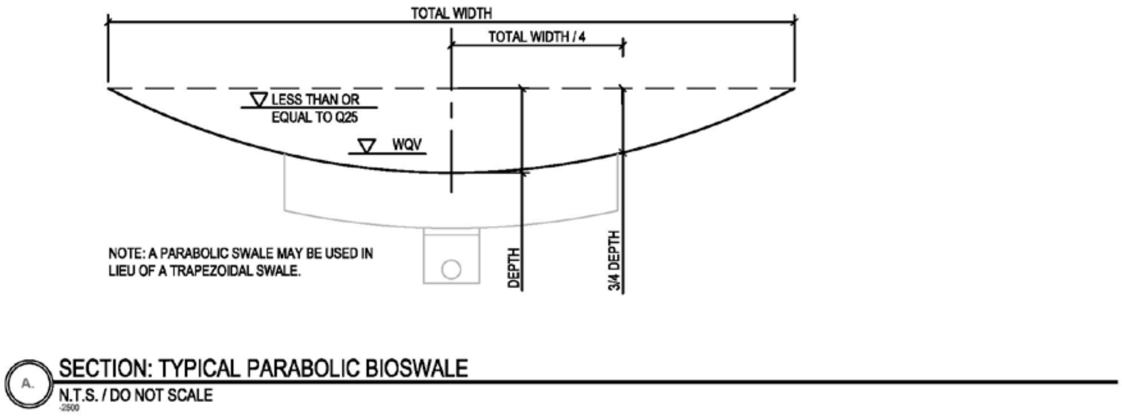
   > "A bioswale consists of an open channel capable of temporarily storing the water quality treatment volume, and a filtering medium consisting of a soil bed with an underdrain system." "It is designed to drain down between storm events within approximately one day. The water quality treatment mechanisms are similar to bioretention practices, except that the pollutant uptake may be more limited if only a grass cover crop is available for nutrient uptake. Bioswales are sized to allow the entire WQv to be filtered or infiltrated through the bottom of the swale. Because they are dry most of the time, they are often the preferred option in residential settings."

Figure 1: Bioswale

Source: Amy Johannsen, 2011

Bioswales are not to be confused with a filter strip or grass swale, which are limited-application structural controls, used mostly for pretreatment and are not considered acceptable for meeting the water quality volume goals. Ordinary grass swales are not engineered to provide the same treatment capability as a well-designed bioswale with filter media and subdrainage systems. Filter strips and grass swales may be used for pre-treatment or included in a treatment train approach where redundant treatment is provided. Figures 2 through 5 provide several views and configurations of bioswales with rock and earthen check dams.

Figure 2: Configuration and design components of a bioswale.
A is a cross-section of a parabolic-shaped bioswale and B a trapezoidal bioswale.



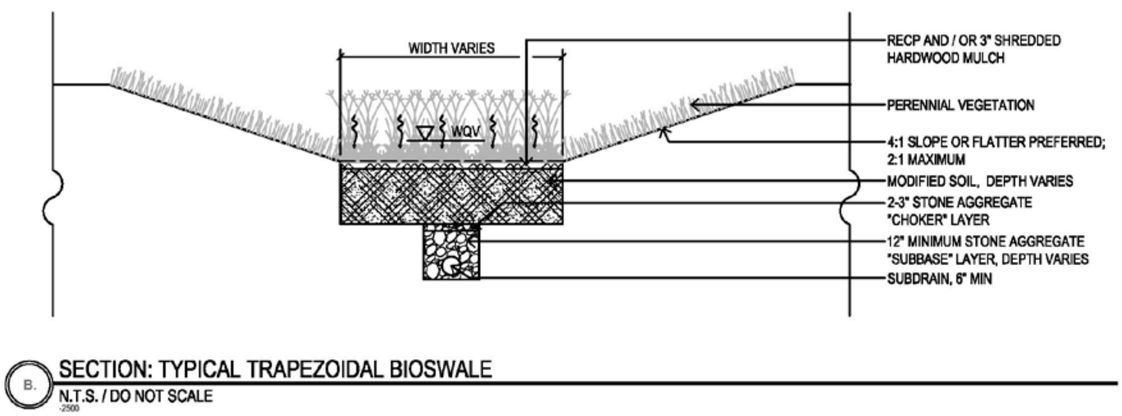


Figure 3: Configuration of a bioswale with a rock check dam showing a top view(A), side view (B), and cross-section (C).

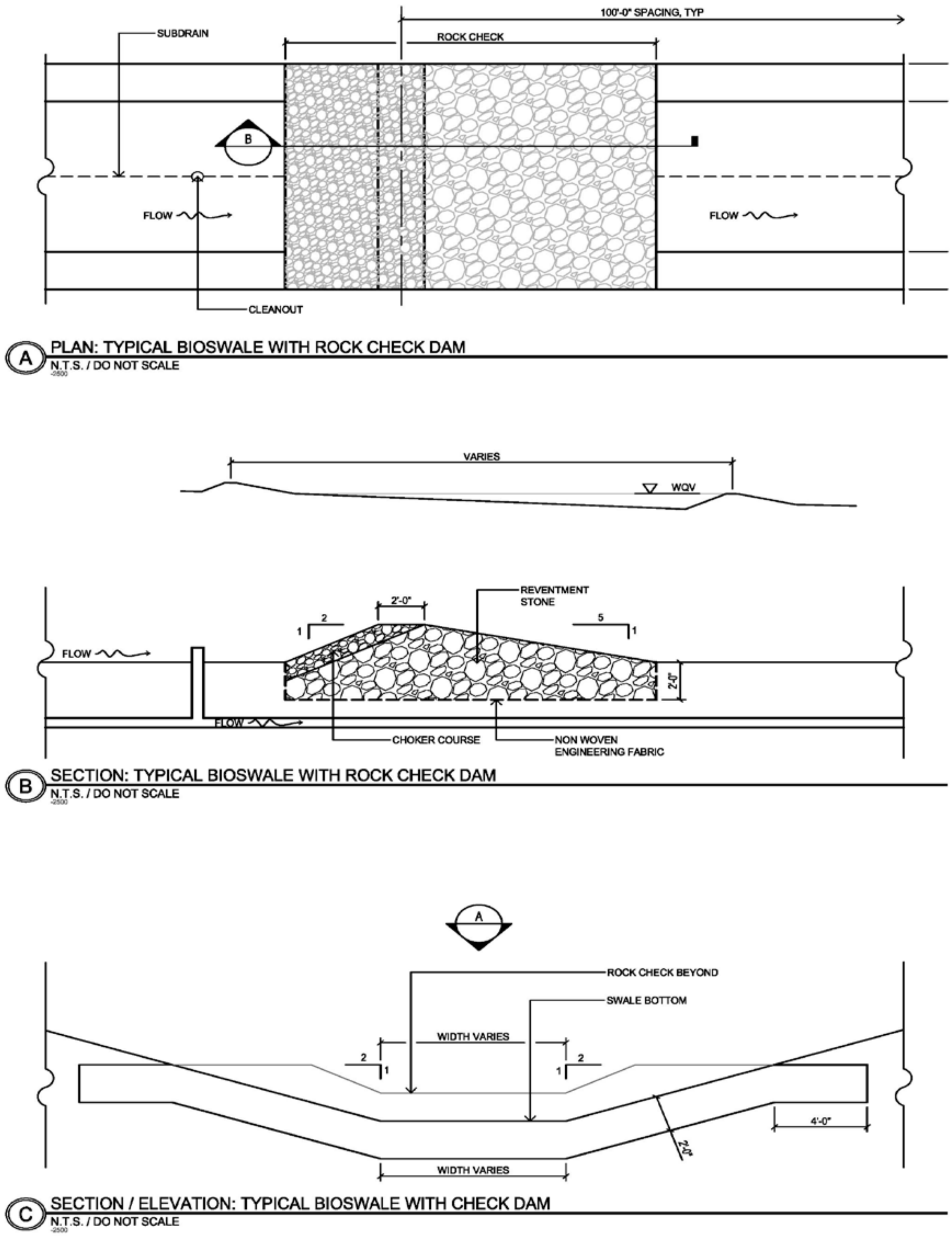


Figure 4: Configuration of a bioswale with an earthen check dam showing a top view(A), side view (B), and cross-section (C).

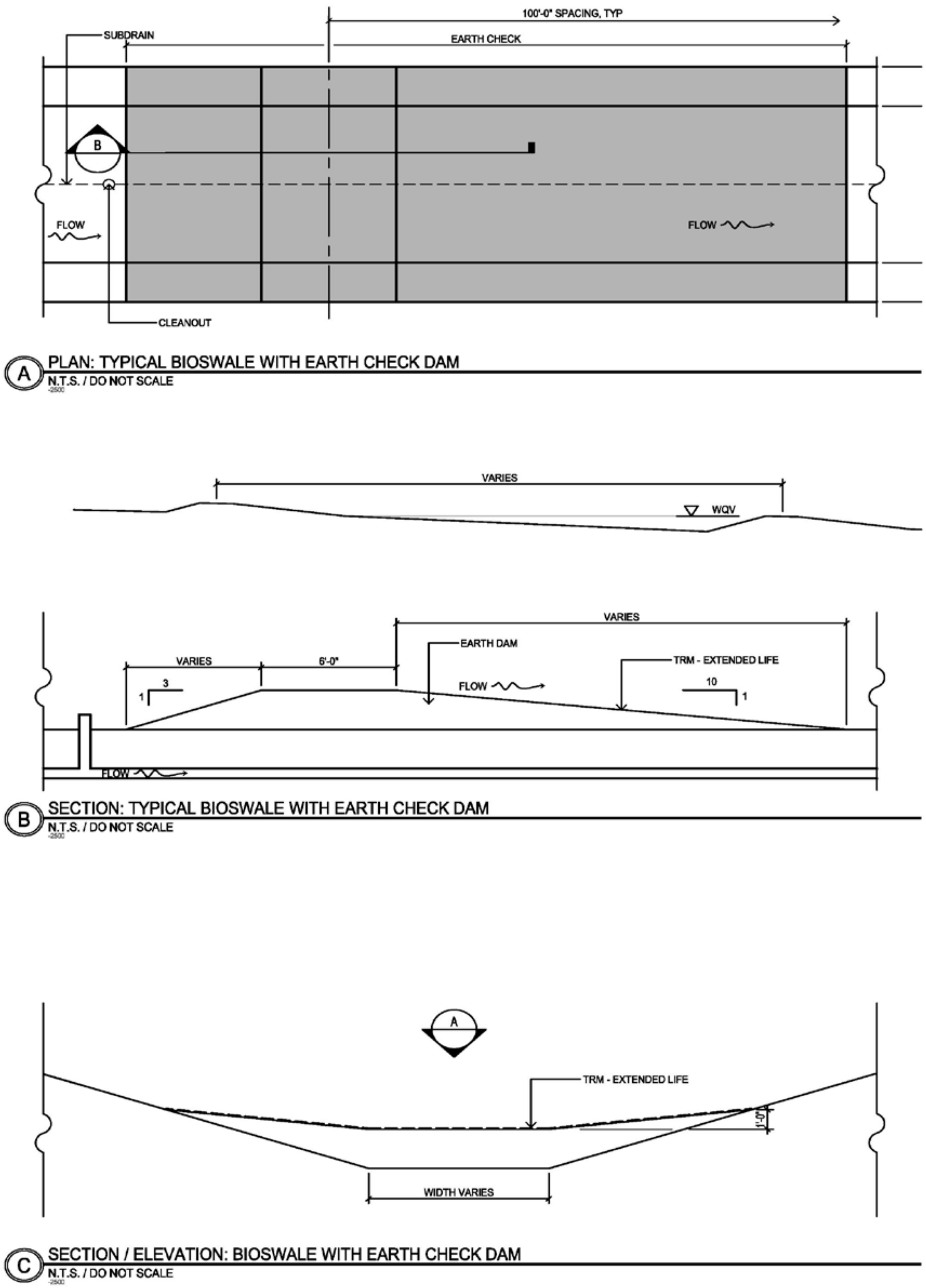
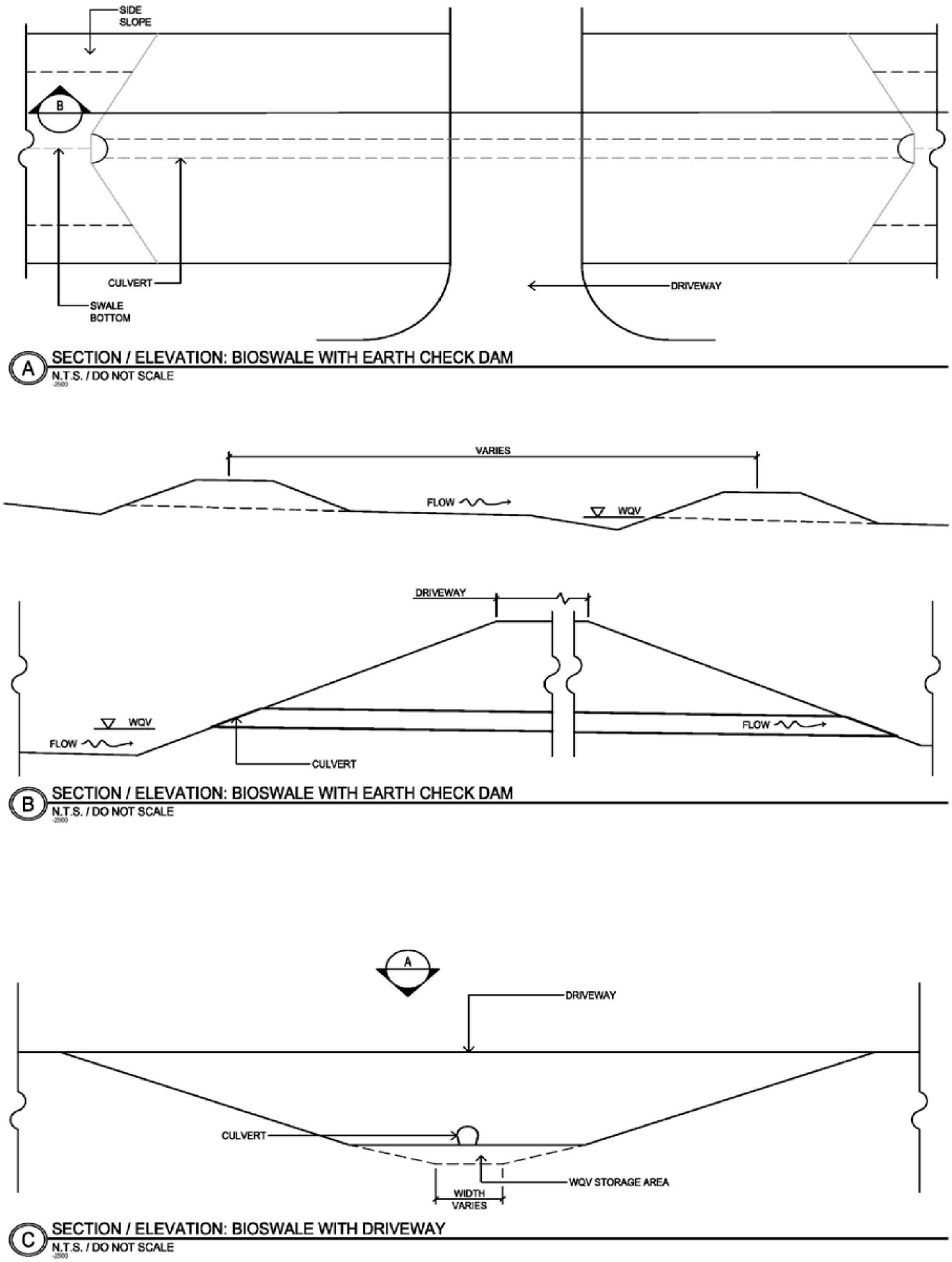


Figure 5: Configuration of a bioswale with an earthen check dam in a driveway application showing a top view(A), side view (B), and cross-section (C).



1. **Applications for stormwater management (Uniform Sizing Criteria).** Bioswales are designed primarily to address water quality management for small storms. While they are able to convey flow from larger storm events from one point to another in a non-erosive manner, they usually only have a limited amount of storage that would be required to meet management needs for these less frequently occurring events. Refer to Section 2B-1 for more discussion on Unified Sizing Criteria.

   a. **Water quality (WQv).** Bioswales rely primarily on filtration through an engineered media to provide removal of stormwater contaminants. Properly designed bioswales are capable of addressing the WQv requirements for a given site. Section 2I-1 provides expected pollutant removal efficiencies that can be used for planning and design purposes.

   b. **Channel protection (Cpv).** Generally only the runoff from the WQv event is considered to be treated by a bioswale. Bioswales are usually used in series with another structural control to provide extended detention of the Channel Protection Volume. However, for some smaller sites, a swale may be designed with a perforated riser or other slow release structure to capture and slowly release runoff from this type of event, which is stored above the surface of the bioswale.

   c. **Overbank and Extreme Flood Protection (Qf and Qp).** Bioswales should be able to convey runoff from larger flood events up to the 25 year storm (Qp25) without eroding channel linings or large scale re-suspension of sediments. High water elevations for the 100 year storm event (Qp100) should be checked to determine any flood impact to adjacent structures or properties.

2. **Pollutant removal capabilities.** Bioswales are designed to manage the water quality volume when sized, designed, constructed, and maintained according to the recommended guideline. For information and data on pollutant removal capabilities for bioswales, see the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the International Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

3. **Application and feasibility.** Bioswales can often be used in place of traditional storm sewer systems. Residential rear yard areas, road and highway corridors, and parking buffers and islands are just some examples where bioswales can be applied.

   They can be installed in watersheds of varying size and land uses. The limiting factor of their use is the ability to have a cross-section that can convey the runoff from the WQv event at slow enough velocities, for a long enough period of time to allow for filtration through vegetation and ultimately infiltration into the soil media. The upper threshold of their use is when runoff rates become too high to allow flow requirements for treatment to be met, or the area required to meet them becomes too large.

   The topography and soils of a site will determine the applicability of the use of swale designs. Overall, the topography should allow for the design of a swale with sufficient slope and cross-sectional area to maintain required treatment velocities. The following criteria should be evaluated to ensure the suitability of a bioswale for meeting stormwater management objectives on a site or development. Table 1 provides a list of considerations when planning for a bioswale.

   a. **General feasibility.**
      - Suitable for residential subdivision usage – yes
      - Suitable for high-density/ultra-urban areas – may be limited by required size
      - Regional stormwater control (for overbank or flood protection) – no

b. **Physical feasibility – physical constraints at project site.**
- **Space required.** The required length of swale is defined by the design treatment flow velocity and average residence time (time flow takes to travel through the bioswale) will define the length required (i.e. flow at 1.0 fps for 10 minutes = 600 feet required).
- **Site slope.** Typically no more than 4% slope (may need to design check dams or drops to provide flatter slope as needed to maintain maximum treatment velocity).
- **Recommended minimum head.** Elevation difference needed at a site from the inflow to the outflow is 3 to 5 feet.
- **Aquifer protection.** Highly contaminated runoff should not be allowed to contaminate aquifers. If runoff from high pollutant loads or high spill potential (hot spots) areas is expected then consider modifications to prevent groundwater contamination.
- **Minimum depth to water table.** 2 feet is recommended between the bottom of a bioswale and the elevation of the seasonally high water table.
- **Depth to bedrock.** The bottom of the aggregate layer should have 2 feet of vertical separation from bedrock.
- **Soils.** Modified soil sand and compost mixture.

Table 1: Planning criteria for bioswales

Distributed Placement and Location	It is preferred to consider stormwater management during initial site design. These practices can treat more manageable amounts of runoff closer to its source. They should be integrated into the site planning process, and aesthetic considerations should be taken into account in their siting and design. Elevations must be carefully worked out to ensure the desired runoff flow enters the facility with no more than the maximum design depth.
Site Integration	Can be placed close to the source of runoff generation. Stormwater management site integration is a preferred alternative to end-of-pipe BMP design, where feasible.
Drainage Area	Bioswales are best used in conjunction with upland practices through a “treatment train” approach. They are suited to both small and large drainage areas.
On-line or Off-line	They are almost always designed as an on-line system. A bioswale is considered on-line if all runoff from the upstream area enters the practice. Off-line systems employ some type of diversion structure, which typically diverts the first flush of flow to the treatment practice, but allows flows from larger events to bypass the practice. This can prevent erosion within the practice and re-suspension of captured sediments.
Flow Diversion for Off-line Swales	A diversion weir, flow splitter, or other practice needs to be designed to route flows from the WQ event to the bioswale, while allowing most of the flows from larger events to bypass the system (via parallel storm sewer system or other conveyance). Refer to Section 2F-1, F for additional design information.
Intermittent Flow	They are designed for intermittent flow and must be allowed to drain and re-aerate between rainfall events.
Storm Events	Typically, bioswales are used to manage the WQv but convey larger storms. Refer to Section 2C-6 for additional information about small storms. Bioswales may offer the possibility to attenuate or detain flows from larger storm events. Design to prevent: • Erosive flow velocities • Deep ponding that could compact soil layers Extended drawdown periods that could affect desired plants

B. Design Methods

1. **Initial Design Consideration and Preliminary Investigation.** For new development sites, it is urged that consideration is given to how post-construction water quality will be addressed early in the design process. Best management practices are most effective when they are located in well distributed locations to be used for stormwater treatment as close as possible to the source of runoff. Distributed practices allow for the creation of a chain of smaller treatment practices, reducing the impact on downstream areas if a single practice should fail. Sites with fewer, larger practices are generally less effective at achieving pollutant and runoff reductions, as each practice has a larger amount of runoff to treat; and should practices fail, a greater proportion of runoff would be mismanaged. Redevelopment sites may have less flexibility, but smaller distributed practices are still preferable to a single, larger practice.

   Before choosing to employ a bioswale, review the feasibility information included earlier in this section. If feasible, proceed with designing this practice, starting with a review of the initial design considerations listed in Table 2, as well as the preliminary investigation information in Table 3.

Table 2: Initial design considerations

Limiting Conditions	Determine the depth to bedrock or typical groundwater elevation. Verify with geotechnical explorations or other methods.
Local Requirements	Determine if there are any local restrictions and/or surface water, groundwater, watershed, or water quality requirements that may apply.
Separation Distances	Ensure that room is available for installation, including any other local setback and/or separation requirements. Recommended setbacks are 25 feet from the foundation of a building; 5 feet from a property line; 50 feet from a private well; 20 feet from a geothermal well field; 100 feet from a municipal well.
Intermittent Flow	Design for intermittent flow and must be allowed to drain and re-aerate between rainfall events.
Character of Runoff	Determine the land use and the percent imperviousness that will generate runoff directed to the bioswale. Identify the pollutants of concern
Treatment Train	Consider multiple practices as part of a treatment train system for pollutant removal and maintenance considerations.
Quality Control	Design for water quality volume and conveyance of larger storm events.
Quantity Control	Not intended for stormwater detention but should be designed to safely convey larger storm events.
Aesthetics and Site Plans	Bioswale locations should be integrated into the site planning process, and aesthetic considerations should be taken into account in their siting and design.
Plant Materials	Native species are recommended. Consider plant establishment, flow conditions, and salt tolerance.
Maintenance	Develop a maintenance plan that includes access paths for equipment required for maintenance. See Table 7.

The following table includes information required to complete the design procedure for bioswales within this section. Determine the values for each variable as accurately as possible. Assumed values may need to be used in preliminary design, and then revised later as site design proceeds and more accurate values can be determined.

Table 3: Preliminary investigations	
Properties of the Drainage Area to a Bioswale	Determine the expected drainage area to be routed to the bioswale and the projected amount of impervious surfaces. Bioswales should be sited such that the topography allows for the design of a channel with sufficiently mild slope (unless small drop structures are used) and cross-sectional area to maintain required treatment velocities. Surface properties required to determine time of concentration will be needed for final design (refer to Section 2A-4). The area upstream of the bioswale should be stabilized prior to construction or adequate controls put in place to prevent high sediment loads being delivered to the bioswale.
Space Required	The relationship between percent of impervious area and required size of a bioswale is not easily defined. The target velocity will set the length required to achieve the desired residence time, then a cross-section developed that can convey the flow for the Water Quality event (WQv) below the maximum allowable velocity for treatment.
Slope	Channel slopes of less than 2% are recommended unless topography necessitates a steeper slope, in which case control structures can be placed to limit the slope to within the desired range. Flow control devices may include check dams and/or drop structures. Energy dissipation may be required below the control structures.
Minimum Head	Make sure there is sufficient elevation difference to pond water as needed and drain the soil and aggregate layers through a subdrain and/or outlet works to a finished surface, swale, or storm sewer system.
Existing Site Soils	Evaluate the existing site soils to determine existing percolation rates.

2. **Typical Components of Bioswales.** Before proceeding with final design, it is important to understand the function and purpose of the elements that make up this type of practice. Table 4 provides a summary of bioswale components and their function.

Table 4: Bioswale design components	
Inlet Structures	Stormwater may be routed to bioswales in many ways, such as sheet flow off hard surfaces, or as concentrated flow from curb openings, downspouts, and pipe outlets. Inlet structures may also include features that divert only a portion of stormwater runoff to the bioswale (known as an off-line configuration). Level spreaders can be used to disperse concentrated flows to sheet flows reducing flow depths and velocities, enhancing pretreatment possibilities.
Pretreatment Area	Use pretreatment areas when sediment and debris are anticipated. These areas reduce the potential for clogging and future maintenance.
Energy Dissipation	Energy dissipation controls may be needed at inlets and outlets.
Ponding Area	Bioswales are sized to store and infiltrate the entire WQv. Water is intended to be temporarily ponded below the crest elevation of each check dam or drop structure, then allowed to infiltrate into the modified soil layer.
Channel	The channel shape should be parabolic or trapezoidal.
Check Dams	Toe of upstream check should be at same elevation as crest of next check dam downstream. They can be constructed of rock or earth materials.
Modified Soil Layer	The modified soil layer filters stormwater. Typically this layer is 6 to 12 inches deep and consists of a uniform mixture of 75 to 90% washed concrete sand, 0 to 10% approved organic compost, 0 to 25% soil with a soil texture that includes A-horizon characteristics, and meets specifications.
Choker Aggregate Layer	The choker layer separates the modified soil layer and aggregate subbase and prevents the modified soil from entering into the aggregate subbase. The 2 to 3 inch layer consists of clean, durable 3/8 inch diameter chip.
Stone Aggregate Subbase Layer	The aggregate layer at the bottom of the structure provides additional temporary storage capacity for the captured runoff after filtration. The layer consists of an open-graded, clean, durable aggregate generally of 1 to 2 inches diameter with a porosity of 35 to 40%.
Subdrain	A minimum 6 inch diameter perforated pipe is required, but generally an 8 inch diameter pipe is recommended to accommodate cleaning and maintenance. They provide the outlet for filtered water in areas with soils with poor percolation rates and act as a secondary outlet where soil percolation rates are better.
Outlet Structures	To avoid excessive ponding depths and drawdown times, outlet controls are needed to manage runoff from larger storm events. An overflow spillway set above the ponding depth can release flows in a non-erosive manner (velocities less than 5 feet per second below the outlet). For on-line configurations, riser pipes, intakes, or weirs may be used to release runoff from larger storms more rapidly than it could infiltrate through the soil layers. The underdrain system should discharge to the storm drainage infrastructure or a stable outfall. The riser can be used as an observation port.
Hydrologic Design	The primary goal of the practice is to treat runoff from the WQv event while having the ability to convey the larger storms. The bioswale should be sized to convey runoff from the largest design storm with a minimum of 6 inches of freeboard and without damage to adjacent property.

Table 5: Summary design criteria for bioswales	
Parameter	**Bioswale Design Criteria**
Pre-treatment Volume	Target of 0.10 inches of runoff volume per impervious acre at inflow point(s).
Bottom Width	For trapezoidal shapes, 4 feet minimum, 8 feet maximum. For widths wider than 8 feet, use berms to create parallel trapezoidal channels or use a parabolic shape. [Trapezoidal channels with a bottom wider than 8 feet may allow a meandering flow path to be created, reducing the effectiveness of the channel.]
Side Slopes	2:1 maximum; 4:1 or flatter is preferred
Longitudinal Slope	Less than or equal to 2% typically. With included subdrains, flatter slopes can be used without concerns for long-term ponding. On greater slopes, frequent check dams or drop structures may be necessary to maintain required treatment velocities.
Sizing Criteria	Length, width, depth, and slope needed to maintain maximum treatment velocities for the WQv event. Outlet structures sized to infiltrate or slowly drain surface ponding areas over a 12 hour period.
Hydraulic Residence Time	Minimum: 5 minutes (partial credit); Optimum: 10 minutes (full credit)
Average Flow Velocity	Optimum: 1.0 fps or less (full credit); Maximum: 1.5 fps (partial credit)
Length	Minimum: 100 feet; Optimum: As needed to achieve full WQv treatment
Check Dams	Check for erosive velocities for overflow conditions
Depth and Capacity	Surface storage with a maximum depth of 18 inches upstream of each check dam for water quality treatment (maximum 12 inch average depth); safely convey 25 year storm peak discharge with non-erosive velocity (maximum 5 fps); adequate conveyance capacity for the 100 year storm peak discharge with 6 inches of freeboard to top of bank (or overbank path provided clear of flood prone structures).

One variation of a bioswale is constructing stepped bioretention cells (each with a level bottom – no longitudinal slope) in series with check dams or drop structures in between. This allows each cell to "step" downgrade, dealing with site slopes in that manner.

Without positive grade, Manning's formula or other methods to calculate velocity in open channels will not apply. For such an application, we would recommend referring to the design section for bioretention cells (Section 2E-4). Collectively, these bioretention cells would need to provide the required surface area for ponding to allow for proper infiltration of the WQv volume to be addressed by that series of "bioretention cells."

Refer to Section 2E-4 for the design procedure for bioretention cells, making sure to check for velocities and erosion potential when larger storms are expected to overtop each check dam or drop structure (Step 12 of that design procedure).

3. **Bioswale Sizing and Design Calculations.** The following design procedure assumes that the designer has completed preliminary investigations, and understands the design components of a bioswale, as outlined in Tables 4 and 5. It is recommended that these calculations be completed as early as possible in the design process so that adequate room is reserved for stormwater management as site design development continues. Calculations can be adjusted as final site design is completed.

The bioswale is sized using the velocity check method to slowly convey the peak runoff rate of flow from the WQv event through the bioswale at very slow velocities. To be considered as treating 100% of WQv for a given site, flow should travel through the swale at no greater than 1.0 fps, for an average length of time of no less than 10 minutes (hydraulic residence time). This assumes that water quality is enhanced by slow filtration of runoff through desired vegetation, then infiltrated into the layers of modified soils. For recommendations for considering partial credit for treatment related to bioswales, read the notes following this design procedure.

Step 1: Compute the WQv peak runoff rate.

To calculate peak rates of flow (in cubic feet per second) for the WQv event, use NRCS TR-20 or TR-55 calculation methods, using adjusted curve numbers (CNs) for this small event (1.25" in 24-hours). Refer to Section 2C-6 for additional information.

Step 2: Compute the peak runoff rates for other key rainfall events.

Refer to: Section 2C-1 - General Information for Stormwater Hydrology
Section 2C-2 - Rainfall and Runoff Analysis
Section 2C-3 - Time of Concentration
Section 2C-5 - NRCS TR-55 Methodology
Section 2C-7 - Runoff Hydrograph Determination

The peak rates of flow and volumes of runoff will need to be determined for the following events:

Use the method outlined in Section 2C-6, C, to compute the peak rate of flow (in cubic feet per second) and volume of stormwater runoff (in cubic feet) for the Channel Protection Volume (CPv).

Use methods such as the NRCS TR-20, TR-55 (Section 2C-7) or other acceptable methods to generate hydrographs to determine peak rates of flow (in cubic feet per second) and runoff volumes (in cubic feet) for the following events:

Overbank Flood Protection Volume Requirements (Qp); Section 2B-1, F
2 year (50% annual recurrence or AR), 5 year (20% AR)
10 year (10% AR) - only if applicable to local storm sewer design

Extreme Flood Volume Requirements (Qf); Section 2B-1, F
10 year (10% AR) - if not applicable to local storm sewer design
25 year (4% AR), 50 year (2% AR), 100 year (1% AR)

Note: The annual recurrence (AR) is the likelihood of a certain rainfall event of a given depth and duration occurring once during any given calendar year.

Step 3: Identify if the bioswale system is intended to be an on-line or off-line system.

If planning for an on-line system, there is no need to design a flow diversion structure; proceed to *Step 4*.

If planning for an off-line system, a diversion weir, flow splitter, or other practice needs to be designed to route flows from the WQ event to the bioswale, while allowing most of the flows from larger events to bypass the system (via parallel storm sewer system or other conveyance). Refer to Section 2F-1, F for additional design information. Include calculation details for the diversion structure with this design procedure.

Step 4: Select, locate, and size pretreatment practice(s).

Forebays, grass filter strips, grass swales, and mechanical separators are some of the options that can be used as pretreatment. Bioswales can fail if too much debris or sediment is allowed to enter the swale, reducing the ability of the modified soil layer to infiltrate stormwater. Pretreatment is needed to filter or capture larger sediment particles, trash, and debris before it can enter the portion of the bioswale area where check dams and outlet restrictions are planned to be used to force temporary ponding to occur. Collected materials will need to be removed over time, so consider how the facility is expected to be maintained when evaluating methods of pretreatment.

For grass swales, refer to Section 2I-2, E for general sizing requirements. The target flow velocity for bioswale treatment is 1.0 fps during the WQv event. Section 2I-2 includes methods on how to modify the value of "n" for Manning's equation to evaluate shallow flow in grass swales.

For filter strips, refer to Section 2I-4, C for sizing requirements.

Forebays should have a storage volume of 0.1 inches of runoff volume per impervious acre drained (Section 2C-11). Sediment will need to be mechanically removed from the forebay over time, so a depth marker and durable, solid materials are recommended for the bottom (to be certain when excavation is complete). The volume of WQv to be used to size the ponding areas of the bioswale can be reduced by the amount addressed in the pretreatment area(s) (typically no more than 10% of WQv).

Step 5: Review entrance designs.

To reduce the potential for surface erosion or displacement of planting materials, it is recommended that flow velocities entering the bioswale should not exceed 5 feet per second (for all storm events reviewed). For on-line systems, the peak velocity of flow entering the swale during the largest Qf event (1% AR) should be checked. Redesign the cross-section of the entrance as needed. Provide stabilization at pipe outlets and areas of rapid expansion as necessary (USDOT FHA HEC-14 is a recommended resource for energy dissipater design).

Step 6: Design geometric elements.

Choose the bottom width, depth, length, and slope necessary to convey the WQv event through the swale with a velocity to promote filtration, settlement, and infiltration. This may need to be an iterative procedure, repeating *Steps* 6 *and* 7 until a solution is reached.

Slope along path of flow: It is recommended that slope along the path of flow range between 0.5 to 2%. Flow path slopes flatter than 0.5% may be used for bioswales, as amended soils and subdrains are recommended for use. Flow path slopes exceeding 2% may make it difficult to create adequate storage between check dams and to keep flow velocities at non-erosive levels.

Bottom width and shape: It is recommended that bottom widths range between 4 and 8 feet. If the bottom is too narrow, it will be difficult to construct. Wider sections promote slower velocities, but if the section is too wide, braiding or meandering may occur in the bottom of the channel. Where feasible, it is encouraged to shape the bottom of the channel with a slight parabolic curve, with the center of the swale being the lowest point (strongly recommended for bottom widths greater than 8 feet). Otherwise, a trapezoidal cross-section for the bioswale is acceptable.

Side slopes: It is recommended that side slopes not exceed 2:1, and slopes of 4:1 or flatter are encouraged for ease of maintenance, additional stability and to provide a larger cross-section to further slow flow velocities.

Check dams: Check dams should not exceed 18 inches in height and be spaced so that the toe elevation of the upstream dam is the same as the crest elevation of the next dam downstream. Dams may be constructed out of either rock or earth materials.

Rock Check Dams

Stone materials should be "clean" or free of fine sediments, rock chips, and other small materials, which could clog the dam prematurely and not allow free percolation through the check dam. It is recommended to use erosion stone (smaller watersheds), Class E revetment stone (larger watersheds), or another similarly sized "clean" stone as the base material and a 12 inch layer choker course of 1 inch clean stone material on the upstream face. Refer to the Section 1I-3 for more detailed descriptions of these stone materials.

The overflow crest of the rock check dam should be constructed level for at least 2 feet long (measured along length of channel) and be the same width as the bottom of the channel for trapezoidal sections or be no less than 8 feet wide for parabolic sections. The slope of the upstream face of the check should not exceed 2(H):1(V), and the downstream face should not exceed 5(H):1(V). On each side of the overflow crest, the top of the dam should be elevated, so that flow is focused away from the sides of the channel (dam should be at least 12 inches higher at the edges of the channel than the crest elevation). Revetment materials should be trenched in 2 feet below the flowline of the channel and keyed in at least 4 feet into the side slopes of the channel. Engineering fabric may be specified under the rock check dam at the discretion of the designer.

Over time sediments, litter, clippings from landscaping, etc. may partially or completely clog the filtering capability of the rock check dam. Most often this material will be trapped by the choker layer, so required maintenance would be to remove that 12 inch thick layer of smaller rock, and replace it with new, clean material.

Earth Check Dams

Earth check dams should be shaped with a level crest of at least 6 feet long (measured along the length of the channel) and be the same width as the bottom of the channel for trapezoidal sections or be no less than 8 feet wide for parabolic sections. They should have maximum 3(H):1(V) slopes on the upstream face and 10(H):1(V) slopes on the downstream face of the dam. On each side of the overflow crest, the top of the dam should be elevated so that flow is focused away from the sides of the channel (dam should be 12 inches higher at the edges of the channel than the crest elevation). Use of a turf reinforcement mat (TRM) over the dam and the surrounding sloped areas is recommended to prevent erosion of the dam or channel during overtopping.

For earth check dams, a secondary overflow (perforated riser, small diameter intake, etc.) should be used to limit ponding depths to 9 inches (similar to maximum ponding depth for biocells) to avoid extended periods of standing water, since water will not be able to percolate through the check dam.

Sub-surface elements: The modified soil layer should be 6 to 12 inches deep and consist of a uniform mixture of 75 to 90% washed concrete sand, 0 to 10% approved organic compost, 0 to 25% soil that includes A-horizon characteristics, and meets the specifications.

The choker layer is a 2 to 3 inch layer that separates the modified soil layer and aggregate subbase and prevents the modified soil from entering into the aggregate subbase. It consists of clean, durable 3/8 inch diameter chip.

The stone aggregate subbase layer is recommended to be at least 12 inches deep. Material should be 1 to 2 inch clean aggregate. The aggregate subbase layer should have a porosity of 35 to 40%. The aggregate subbase layer will contain a subdrain (see *Step 11*).

The depth of the aggregate subbase layer can be increased to provide for additional storage or to enhance infiltration to subsoil layers. However, it is desired that the aggregate subbase layer should drain out within 48 hours after a storm event. Percolation rates of undisturbed, intact subsoils or the capacity of the subdrain system may limit the depth of storage that can be provided below a subdrain outlet. For example, subsoils with percolation rates of 0.50 inch/hour may be able to drain down 24 inches of water stored in the aggregate subbase layer below the subdrain over the 48 hour drawdown period.

Step 7: Calculate the peak flow velocity for the WQv event.

The peak rate of flow for the WQv event should have been determined in *Step 1* of this procedure. Use Manning's equation with the appropriate values for the geometry of the swale selected in *Step* 6 to determine the expected channel velocity for this storm event. To provide adequate water quality treatment, a peak flow velocity of 1.0 feet per second or less through the bioswale is recommended.

Reminder that Section 2I-2 includes methods on how to modify the value of “n” for Manning’s equation to evaluate shallow flow in grass swales.

Step 8: Determine the length of flow required treating runoff.

To provide for adequate treatment via filtration, it is recommended to assume a minimum average residence time of 10 minutes. Residence time is the length of time it takes for runoff to flow from one end of the bioswale to the other. We can compute the length required from the following equation:

$$L_{bioswale} = T_{res} \text{ x } V \text{ x } (60 \text{ sec/min})$$

<u>Variables:</u>

($L_{bioswale}$)	=	required length of bioswale, feet
(Tres)	=	average hydraulic residence time, minutes
(V)	=	peak velocity within the bioswale, during the WQv event, fps

This equation assumes that all flow enters the bioswale at the upstream end, and flows through the entire length before leaving the practice. If this would not be the case then the designer should weight the answer above to account for that (see design example for more details).

Step 9: Check larger storm events for erosion potential due to flow velocity.

The peak rates of flow for larger events should have been determined in *Step 2*. Use Manning’s equation, with the appropriate values for the geometry of the swale selected in *Step 6* to determine the expected channel velocity for these storm events. It is recommended to revise channel geometry if these calculations indicate that channel velocities exceed 5 fps for events equal to or smaller than a 24 hour storm having a 25 year occurrence interval (4% AR).

Reminder that Section 2I-2 includes methods on how to modify the value of “n” for Manning’s equation to evaluate shallow flow in grass swales.

Step 10: Check the drawdown time for the ponding areas.

The swale system should be designed to drain the temporary stored water from the surface within 12 hours through one or more of the following methods:

Soil infiltration: The rate of infiltration will equal the expected infiltration rate of the soil times the surface area where infiltration is expected. This should only be considered for where amended soils are planned to be used with an aggregate subsoil layer including a subdrain. Calculate the infiltration rate from the ponded area as follows:

$$Q_{inf} = k \text{ x } A \text{ x } (1 \text{ ft} / 12 \text{ in}) \text{ x } (1 \text{ hr} / 3600 \text{ s})$$

Solve for (Af) = Required ponding area to treat WQv, in square feet

Variables:
(k) = coefficient of permeability, inches/hour
(A) = footprint area of the bottom of the swale, square feet

Percolation through a check dam: The rate of percolation through a rock check dam can be calculated similar to a method developed by the NAHB/NRC Designated Housing Research Center at Penn State University. This method assumes that the check dam is constructed out of clean stone (see Figure 3).

$$Q_{perc} = (h^{3/2} \text{ x } W) / [(L/D + 2.5 + L^2)^{1/2}]$$

Solve for (Q_{perc}) = Percolation rate through a check dam

Variables:
(h) = depth of ponding from the water elevation to the surface at the dam, feet
(W) = average width of the dam measured across the swale, feet
(L) = horizontal flow path length through the check dam along the flow direction, feet
(D) = average rock size diameter, feet

Note that where the 1 inch washed stone choker course is used, flow through that part of the dam may be slower than what the larger stone on the downstream will allow. In that case, L = 1 foot (for a 1 foot thick choker course) and D = 0.083 feet (for 1 inch diameter stone). The choker course may need to maintained or removed and replaced should it become clogged by sediment, clippings, or other debris.

Water quality inlets, notch weirs, orifices, etc.: Inlet structures, riser pipes, weirs, or orifice restrictions are options for features that can be used to control the release rate for the WQv from the ponding areas. Refer to Section 2C-12 on how to correctly size the selected type of control structure.

To achieve proper treatment, flow through surface outlets such as these should be limited so that if soil infiltration is ignored, surface water would be drained from the ponding areas in approximately 12 hours (too quick of a drawdown time through surface outlets will allow water to bypass filtration through the soil media).

By reviewing the options for drawdown, a stage-storage relationship can be developed for outflow from the system. Perform calculations to demonstrate that the portion of the WQv to be captured by the bioswale can be infiltrated into subsoil layers or discharged through other controls within 12 hours after such an event.

Step 11: Subdrain system design.

For a bioswale system, a subdrain is needed to drain the aggregate layer over a 24 hour period. The design flow rate can be taken from the soil infiltration equation in *Step 10*.

After solving for Q, use typical engineering methods to size pipe diameter.

Subdrain materials should comply with requirements for Type 1 subdrains in SUDAS Specifications Section 4040. A minimize size of 6 inches is required. However, a minimum size of 8 inches is recommended for cleaning and inspection.

The length of pipe should extend along the entire length of the bioswale, and the aggregate layer should surround the subdrain as required in SUDAS Specifications Figure 4040.231. Note that the portion of the aggregate layer below the invert of the subdrain can only be drained through infiltration into the native soils below; refer to notes within *Step 6*.

Step 12: Additional provisions for large storm flows.

Bioswale systems are typically designed as on-line systems. They will receive flows from larger storms, which need to be safely conveyed through the swale system and possible into outlet structures (i.e. culverts, storm sewers, etc.) at the downstream end of the project. Flow across check dams in larger storms should be checked using weir equations (refer to Section 2C-12) to make sure that larger flows can be passed over the dams and remain within the cross-section of the swale and be kept at non-erosive velocities.

Inlet structures, riser pipes, weirs, or stabilized spillways are options for features that can be used as a second stage for controlled release of larger storms into the downstream system (typically at a culvert or other inlet at the downstream end of the bioswale). Small storms need to be captured and slowly released from the ponding areas, while large storms need to be able to pass downstream without causing flooding.

Step 13: System outlet and overland spillway design considerations.

Check peak flow velocities near pipe outlets and spillways expected to be overtopped during large storms. For all storm events reviewed, velocities at any pipe outlets should be less than 5 feet per second and stabilization provided (refer to HEC-14). Overflow spillways should be designed with sufficient width to keep velocities less than 5 feet per second, and be properly stabilized or reinforced to withstand such velocities. Refer to Section 2C-12, H for additional information.

There may be situations where there is insufficient space to design a swale that can convey the expected runoff from the WQv event through a bioswale, while maintaining a maximum flow velocity for 1.0 fps for residence times of 10 minutes or longer. In such cases, partial credit may be considered for the treatment provided by the bioswale. To meet full treatment requirements at a given site, the remainder of the WQv treatment for a given site will need to be achieved by other practices installed in series, often referred to as a "treatment train" (i.e. multiple practices such as biocells, filter strips, etc. that collectively meet site requirements).

It is recommended that partial credit for WQv treatment by a bioswale be applied in the following manner:

1) *For expected WQv event flow velocities between 1.0 and 1.5 fps (VELOCITY FACTOR):*
Reduce credit given linearly from 100% at 1.0 fps to 50% at 1.5 fps. No credit should be considered for WQv flow velocities that exceed 1.5 fps.

Maximum calculated WQv Velocity (fps)	Velocity Credit Factor (%)
Less than or equal to 1.0	100%
1.1	90%
1.2	80%
1.3	70%
1.4	60%
1.5	50%
Greater than 1.5	0%

2) *For residence times between 5 and 10 minutes (RESIDENCE TIME FACTOR):*
Partial credit will begin at 50% for a residence time of 5 minutes and increase linearly to full credit at 10 minutes. No credit should be considered for residence times of less than 5 minutes.

Calculated Average Residence Time (min)	Residence Time Credit Factor (%)
20 or more	200%
15	150%
10	100%
5	50%
Less than 5	0%

3) *Combined residence time and velocity adjustments:*
Portion of required WQv treated, % = (Velocity factor, %) x (Residence time factor, %)

The credit factors for VELOCITY and RESIDENCE TIME above are multiplied to determine the overall partial credit factor. For example, a bioswale designed with a WQv flow velocity of 1.5 fps and a residence time of 5 minutes would receive 50% credit under each criteria. Multiplying these together yields = 0.50 x 0.50 = 0.25.

So in that case, the bioswale would be considered to treat 25% of the given site's required treatment volume. Other practices would need to be designed in a series to manage the remainder of the runoff from the WQv event.

Credit can also be considered for residence times longer than 10 minutes, to offset expected treatment reductions due to faster flow velocities. For example, a swale with a residence time of 20 minutes could be considered for a 200% credit. Such a swale designed with a WQv velocity of 1.5 fps the partial credit formula would be applied as follows = 2.00 x 0.50 = 1.00. So, under these circumstances, the swale would be considered to be fully treating the WQv at the given site.

Note: If practices at a given site exceed the WQv treatment requirements, the excess treatment should not be used to offset runoff from other sites left untreated. The benefits of providing excess treatment of runoff at one site will not effectively offset the amount of pollution and runoff that is released from an untreated site. In this way, managing small storms is different management of larger storms where in some circumstances excess detention can be provided on one site to offset additional release rates from another.

4. **Design Example.**

Figure 6: Recreation center, Central Iowa

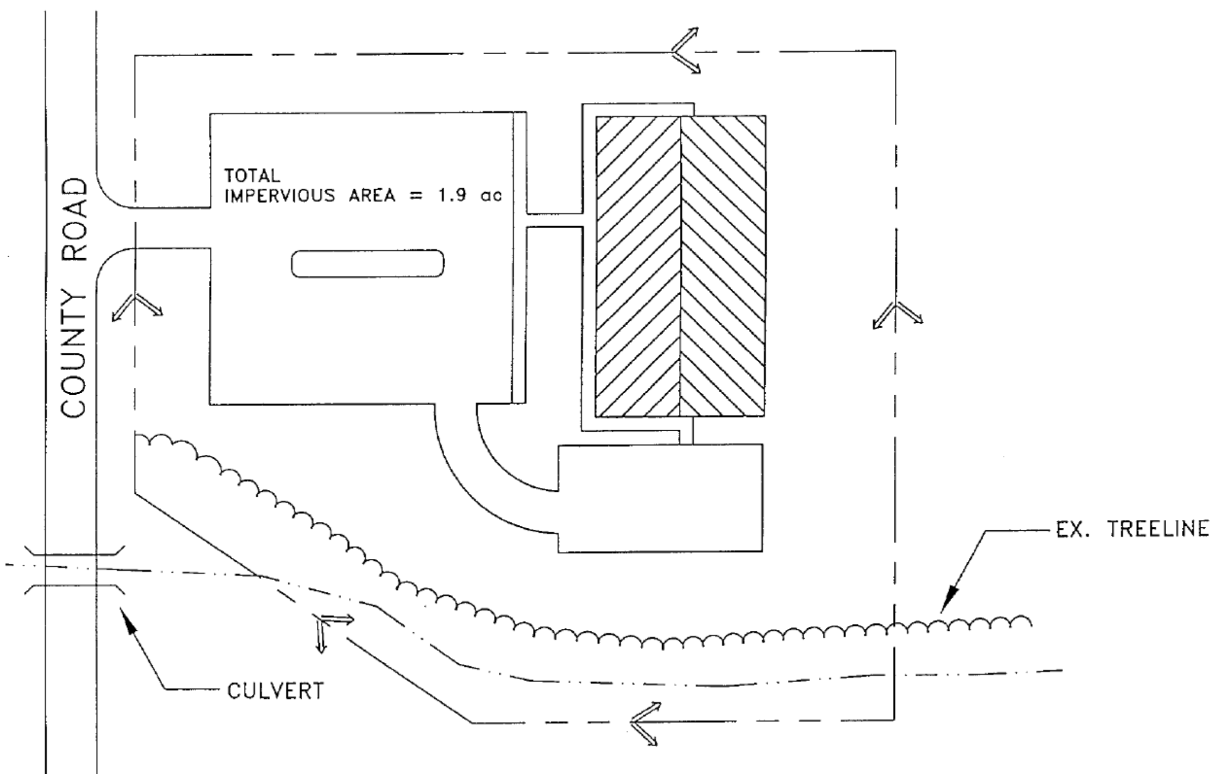


Table 6: Site data			
Base Site Data	**Hydrologic Data**		
Total site drainage area (A) = 3 ac		Pre-	Post-
Impervious area = 1.80 ac; I = 1.80 / 3.0 = 60%	CN	58	88
Soils: pre-developed HSG B (loam) developed use HSG C for compaction	t_c	25 min 0.42 hr	10 min 0.17 hr

Using the velocity check method, the bioswale is sized to slowly convey the peak runoff rate of flow from the WQv event through the bioswale at a velocity of less than 1.0 fps, for an average length of time of no less than 10 minutes (hydraulic residence time) to be considered as providing full-treatment of the WQv as a stand alone practice.

Step 1: Compute the WQv peak runoff rate.

(DA) = 3 acres
(I) = 60 %
(P) = 1.25 inches

To calculate peak rates of flow (in cubic feet per second) for the WQv event, use NRCS TR-20 or TR-55 calculation methods, using adjusted curve numbers (CNs) for this small event (1.25 inches in 24hours). Refer to Section 2C-6 for additional information.

Storm Event	Curve Number NRCS Adjusted	Rainfall Depth (inches)	Post-developed	
			Peak Rate (cfs)	Volume (cubic feet)
WQv	94	1.25	3.1	8,035

Using an adjusted CN value, the volume of runoff from this calculation should be close to the value of WQv volume that can be calculated using methods from Section 2C-6. (8,031 CF ≈ 8,035 CF)

The peak runoff rate being routed through the bioswale may be reduced by employing other practices upstream for pre-treatment or as other best management practices placed in a series or "treatment train" (i.e. bioretention, filter strips, etc.).

Step 2: Compute the peak runoff rates for other key rainfall events.

Use method outlined in Section 2C-6, C, to compute the peak rate of flow (in cubic feet per second) and volume of stormwater runoff (in cubic feet) for the Channel Protection Volume (CPv).

For this example, TR-55 software was used, with results as follows:

1 year, 24 hour storm; for Central Iowa = 2.91 inches rainfall depth
Type II rainfall distribution, shape factor 484 (default values)

Condition	CN	Tc (minutes)	Peak Rate (cfs)	Volume Watershed (inches)	Volume (cubic feet)
Pre-developed	58	25	0.06	0.10	1,100
Post-developed	88	10	5.5	1.3	14,400

Use methods such as the NRCS TR-20, TR-55 (Section 2C-7), or other acceptable methods to generate hydrographs to determine peak rates of flow (in cubic feet per second) and runoff volumes (in cubic feet) for the following events:

	Storm Event	Rainfall Depth (inches)	Pre-developed		Post-developed	
			Peak Rate (cfs)	Volume (cubic feet)	Peak Rate (cfs)	Volume (cubic feet)
Qp	2 year	2.91	0.29	2,600	7.5	20,000
	5 year	3.64	0.96	5,500	10.2	27,000
	10 year	4.27	1.8	8,500	13	33,000
Qf	25 year	5.15	3.1	13,000	16	43,000
	50 year	5.87	4.5	18,000	19	51,000
	100 year	6.61	5.9	23,000	21	59,000

Step 3: Identify if the bioswale system is intended to be an on-line or off-line system.

Most of the time, a bioswale will be designed as an on-line system. For this exercise, assume that is the case here. Runoff from all storm events will pass through this bioswale from this site. There is no need to size a diversion structure in this case.

Step 4: Select, locate, and size pretreatment practice(s).

Alternatives to evaluate for pretreatment are:

Grass swale (without modified soil layers): For grass swales, refer to Section 21-2E for sizing requirements. Using a site imperviousness of 60%, and a slope of less than 2%; a 45 foot long, 2 foot (minimum) wide swale is needed to meet pretreatment requirements.

Filter strip: If the 10 inch discharge pipe is connected to a level spreader to convert concentrated flow to sheet flow, a filter strip could be used. For filter strips, refer to Section 2I-4.C.4 for sizing requirements. The chart uses a maximum inflow approach length for impervious areas of 75 feet. To have an equivalent impervious approach length maximum of 75 feet, the 1.8 acres (78,408 square feet) of impervious surfaces in this example needs to be spread over a width of 1,045 feet (= 78,408 SF / 75 feet). Providing this length does not seem feasible. A filter strip might be a better option with a level spreader in a smaller watershed area, or as an on-line system receiving sheet flow runoff from paved areas that are less than 75 feet in length.

Forebay: A forebay with a storage volume of 0.1 inches of runoff volume per impervious acre drained is an option.

Storage required = (DA) x (I/100) x (0.1 inch) x (1 foot / 12 inches) x (43,560 SF / acre)
= (3 acres) x (60/100) x (0.1 inch) x (1 foot / 12 inches) x (43,560 SF / acre)
= 653 CF (or 8% of WQv)

A 15 foot wide by 15 foot long by 3 foot deep wet forebay would meet this requirement (675 CF).

Combination of Practices: A combination of practices could also be considered to meet pretreatment requirements, with each practice meeting a certain portion of the requirement. For example, runoff from the parking bays could be directed via sheet flow across a buffer strip to the bioswale, while runoff from downspouts and other hard surface areas is directed via storm sewer to a forebay at the upstream end of the bioswale.

Evaluate the Options and Choose Control(s): For this example, it is assumed that only the forebay option (at the upstream end of the bioswale) will be chosen.

Step 5: Review entrance designs.

To reduce the potential for surface erosion or displacement of planting materials, it is recommended that flow velocities entering the bioswale should not exceed 5 feet per second (for all storm events reviewed). For on-line systems, the peak velocity of flow entering the swale during the largest Qf event (1% AR) should be checked. Redesign the cross-section of the entrance as needed. Provide stabilization at pipe outlets and areas of rapid expansion as necessary (USDOT FHA HEC-14 is a recommended resource for energy dissipater design).

We can estimate the velocity of flow leaving the level edge of the forebay at the beginning of the bioswale in this example by using the weir equation. Flow over a rectangular weir is calculated as follows:

$$Q = C \times L \times H^{1.5}$$

Where Q = flow (in cubic feet per second, cfs)
L = length of the crest of the weir (feet, measured perpendicular to flow)
H = head, or depth of flow just upstream of the weir (feet)
C = a standard coefficient (approximately 3.3 for a rectangular weir)

Rearranging to solve for H yields => $H = [Q / (C \times L)]^{2/3}$

For this example, $H = [\,21 \text{ cfs} / (3.3 \times 15 \text{ feet})\,]^{2/3}$

So the area of flow over the edge of the forebay would be $H \times L = 0.56 \times 15 \text{ feet} = 8.4 \text{ ft}^2$

From the continuity equation => $V = Q/A = 21 \text{ cfs} / (8.4 \text{ ft}^2) = 2.5 \text{ fps} < 5 \text{ fps}$ (OK)

Step 6: Design geometric elements.

Choose the bottom width, depth, length, and slope necessary to convey the WQv event through the swale with a peak velocity of less than 1.0 feet per second under free flow conditions. This may need to be an iterative procedure, repeating *Steps 6 and 7* until a solution is reached.

Initially for this example, the following parameters have been chosen:

Slope along path of flow: 1% along the length of the bioswale (based on site topography).

Bottom width and shape: A trapezoidal section with a 6 feet wide bottom.

Side slopes: For ease of maintenance, additional stability and to provide a larger cross-section to further slow flow velocities, side slopes of 4: 1 (H:V) are to be used at this site.

Subsurface elements: The subsurface cross-section will be similar to that of a bioretention cell. The modified soil layer is chosen to be 6 inches deep and consist of a uniform mixture of 80% concrete sand and 20% approved organic compost material that meets specifications.

The choker layer is to be a 3 inch layer that separates the modified soil layer and aggregate subbase and prevents the modified soil from entering into the aggregate subbase. It consists of clean, durable 3/8 inch diameter chip. The stone aggregate subbase layer is recommended to be 12 inches deep. Material should be 1 to 2 inch clean aggregate. The aggregate subbase layer will contain a subdrain, with the aggregate material extending at least 1 foot on either side of the subdrain (for wide parabolic cross-sections, the aggregate subbase material should extend across at least 10% of the area that would be covered by water during the WQv event).

Step 7: Calculate the peak flow velocity for the WQv event.

The peak rates of flow for the WQv event were determined in *Step 1.* Use Manning's equation with the appropriate values for the geometry of the swale selected in *Step* 6 to determine the expected channel velocity for this storm event.

Depth (feet)	Adjusted Manning's "n"	Channel Capacity	
		Peak Rate (cfs)	Velocity (fps)
0.53	0.115	3.1	0.72

Reminder that Section 2I-2 includes methods on how to modify the value of "n" for Manning's equation to evaluate shallow flow in grass swales.

Since the channel velocity is less than 1.0 fps, the channel geometry is OK, and we can proceed to *Step 8*.

Step 8: Determine the length of flow required for treating runoff.

To provide for adequate treatment, it is recommended to assume an average residence time of 10 minutes. Residence time is the length of time it takes for runoff to flow from one end of the bioswale to the other. We can compute the length required from the following equation:

$$L_{bioswale} = T_{res} \text{ x V x (60 sec/min)}$$

<u>Variables:</u>
(Lbioswalc) = required length of bioswale, feet
(Tres) = average hydraulic residence time, minutes
(V) = peak velocity within the bioswale, during the WQv event, fps

$$L_{bioswale} = \text{(10 minutes) x (0.72 fps) x (60 sec/min)} = \underline{\text{432 feet}}$$

For this example, we are assuming that all flow enters the bioswale at the upstream end and flows through the entire length before leaving the practice. If this would not be the case, the designer should account for that. If all flows enters uniformly along the sides, consider a stepped bioretention cell.

Step 9: Check larger storm events for erosion potential due to flow velocity.

The peak rates of flow for larger events should have been determined in *Step 2.* Use Manning's equation, with the appropriate values for the geometry of the swale selected in *Step 6* to determine the expected channel velocity for these storm events. It is recommended to revise channel geometry if these calculations indicate that channel velocities exceed 5 fps for events equal to or smaller than a 24 hour storm having a 25 year occurrence interval (4% AR).

Reminder that Section 2I-2 includes methods on how to modify the value of "n" for Manning's equation to evaluate shallow flow in grass swales.

Assuming free-flow in the channel and solving Manning's equation for various depths (with the adjusted coefficients listed) yields the following results. The flow rate through the bioswale for the

25 year event is expected to be 16 cfs (from *Step 2*), so a peak flow velocity of around 2 fps would be expected for this event. We can also see from the results below that the channel has the capacity to convey even flows larger than the 100 year flow (21 cfs, from *Step 2*) with a velocity of less than 5 feet per second.

Depth (feet)	Adjusted Manning's "n"	Channel Capacity	
		Peak Rate (cfs)	Velocity (fps)
0.20	0.150	0.52	0.38
0.40	0.138	1.9	0.63
0.60	0.102	5.4	1.1
0.80	0.066	14	1.9
1.00	0.030	48	4.8

To prevent erosion, it is also wise to check the velocity of flow as water crests over each check dam during the 25 year storm event. One approach would be to treat the crest as a rectangular weir having a width equal to the width of the channel. The recommended shape for a check dam creates a "notch" in the middle of the dam (equal to the width of the channel bottom), with the dam being higher where it meets the side slopes of the channel. This focuses the flows into the middle of the channel and helps reduce the potential for erosion along the side slopes of the channel.

Flow over a rectangular weir is calculated from the following equation: $Q = C \times L \times H^{1.5}$

Where Q = flow (cubic feet per second, cfs)
L = length of the crest of the weir (feet, measured perpendicular to flow)
H = head, or depth of flow just upstream of the weir (feet)
C = a standard coefficient (approximately 3.3 for a rectangular weir)

Rearranging to solve for H yields => $H = [Q / (C \times L)]^{2/3}$

For this example, $H = [\,16 \text{ cfs} / (3.3 \times 6 \text{ feet})\,]^{2/3} = 0.87$ feet

So the area of flow over the dam would be $H \times L = 0.87 \text{ feet} \times 6 \text{ feet} = 5.2 \text{ ft}^2$

From the continuity equation => $V = Q / A = 16 \text{ cfs} / (5.2 \text{ ft}^2) = 3.1 \text{ fps} < 5 \text{ fps}$ (OK)

So under both free flow and weir conditions, the flow in the bioswale remains below 5 fps for events up through the 25 year, 24 hour duration storm.

<u>Note:</u> Another check dam design option would be an earthen berm (which won't allow flow to percolate through it). This would require a method of draining the ponded water above the check dam into the subdrain system. Options would be a French drain (aggregate layers extended to surface) or overflow riser connected to the subdrain. Inlets should be designed to reduce the potential for sediment or other debris from entering the subdrain system. Caution should also be taken to not drain ponded water too quickly through surface inlets, allowing runoff to bypass filtration through the modified soil layers.

Step 10: Calculate the drawdown time for the ponding areas.

To treat the desired portion of the WQv, water should drain out of each temporary ponding area in one of the following ways: infiltration through the modified soil to the choker aggregate layer and into the aggregate subbase layer and into the subdrain system, percolation through a check dam, or slow release through a water quality inlet (small notch weir, perforated riser, or orifice). The swale system should be designed to drain the temporary stored water from the surface within 12 hours.

Soil infiltration: Calculate the infiltration rate from the ponded area as follows:

$$Q_{inf} = k \text{ x } A \text{ x } (1 \text{ ft} / 12 \text{ in}) \text{ x } (1 \text{ hr} / 3600 \text{ s})$$

(k) = 1 inch/hour (for the modified soil mix)
(A) = length x width of bottom of bioswale = 432 feet x 6 feet = 2,592 square feet

$$Q_{inf} = 1 \text{ in/hr x } 2{,}592 \text{ SF x } (1 \text{ ft} / 12 \text{ in}) \text{ x } (1 \text{ hr} / 3600 \text{ s}) = 0.06 \text{ cfs}$$

Percolation through a check dam: The rate of percolation through a check dam can be calculated similar to a method developed by the NAHB/NRC Designated Housing Research Center at Penn State University. This method assumes that the check dam is constructed out of clean stone.

$$Q_{perc} = (h^{3/2} \text{ x } W) / [(L/D + 2.5 + L^2)^{1/2}]$$

(h) = depth of ponding from the water elevation to the surface at the dam, feet (varies)
(W) = average width of the dam measured across the swale, feet (varies with "h")
(L) = horizontal flow path length through the check dam along the flow direction = 1 foot
(D) = average rock size diameter = 0.083 feet

Note that where the 1 inch washed stone choker course is used, flow through that part of the dam may be slower than what the larger stone on the downstream will allow. In that case, L = 1 foot (for a 1 foot thick choker course) and D = 0.083 feet (for 1" diameter stone).

The results of this equation are going to vary with depth.

Depth (h) (feet)	Avg. Width (W) (feet)	Percolation Rate (Qperc) (cfs)	w/ Clogging Factor x 20%
0.3	7.2	0.30	0.06
0.6	8.4	0.99	0.20
0.9	9.6	2.1	0.42
1.2	10.8	3.6	0.72
1.5	12.0	5.6	1.12

Water quality inlets, notch weirs, orifices, etc.: None proposed in this example.

By adding the outflows due to infiltration, percolation, and other outlet types, a stage-storage relationship can be developed for outflow from the system. It is recommended to use computer software packages to perform routing calculations to demonstrate that the portion of the WQv to be captured by the bioswale can be infiltrated into subsoil layers or discharged through other controls within 12 hours after such an event.

Step 11: Subdrain system design.

For a bioswale system, a subdrain is needed to drain the aggregate layer over a 24 hour period. The infiltration rate was calculated in *Step 9* as 0.06 cfs. The minimum recommended diameter of 6 inches will have sufficient capacity. The aggregate layer should surround the subdrain as required in SUDAS Specifications Figure 4040.231.

Step 12: Additional provisions for large storm flows.

Bioswale systems are typically designed as on-line systems they will receive flows from larger storms, which need to be safely conveyed through the swale system and possible into outlet structures (i.e. culverts, storm sewers, etc.) at the downstream end of the project. Flow across check dams in larger storms should be checked using weir equations (refer to Section 2C-12) to make sure that larger flows can be passed over the dams and remain within the cross-section of the swale.

We already checked the capacity of the channel to convey the 100 year storm event in *Step 9*, and it was okay for free-flow conditions. Looking at the check dams, we have the following:

Flow over a rectangular weir is calculated from the following equation: $Q = C \times L \times H^{1.5}$

Where Q = flow (cubic feet per second, cfs)
L = length of the crest of the weir (feet, measured perpendicular to flow)
H = head, or depth of flow just upstream of the weir (feet)
C = a standard coefficient (approximately 3.3 for a rectangular weir)

Rearranging to solve for H yields => $H = [Q / (C \times L)]^{2/3}$

For this example, $H = [\,21 \text{ cfs} / (3.3 \times 6 \text{ feet})\,]^{2/3} = 1.04$ feet

So the area of flow over the dam would be H x L = 1.04 feet x 6 feet = 6.2 ft^2

From the continuity equation => $V = Q / A = 21 \text{ cfs} / (6.2 \text{ ft}^2) = 3.4$ fps

It is recommended that the channel has 0.5 foot of freeboard above the 100 year flow elevation. The channel would need to be slightly more than 3.0 feet deep to meet this standard. (Total bioswale depth = 1.5 feet to crest + 1.04 feet of head + 0.5 feet of freeboard = 3.04 feet)

Step 13: System outlet and overland spillway design considerations.

Assuming the flow is controlled by check dams only in this example, there is nothing more to check in this step. If the bioswale drained into an intake, culvert, or other storm sewer system, the designer should check peak flow velocities near pipe outlets and spillways expected to be overtopped during large storms. For all storm events reviewed, velocities at any pipe outlets should be less than 5 feet per second and stabilization provided (refer to HEC-14). Overflow spillways should be designed with sufficient width to keep velocities less than 5 feet per second and be properly stabilized or reinforced to withstand such velocities. Refer to Section 2C-12, H for additional information.

Figure 7: Site plan for project example

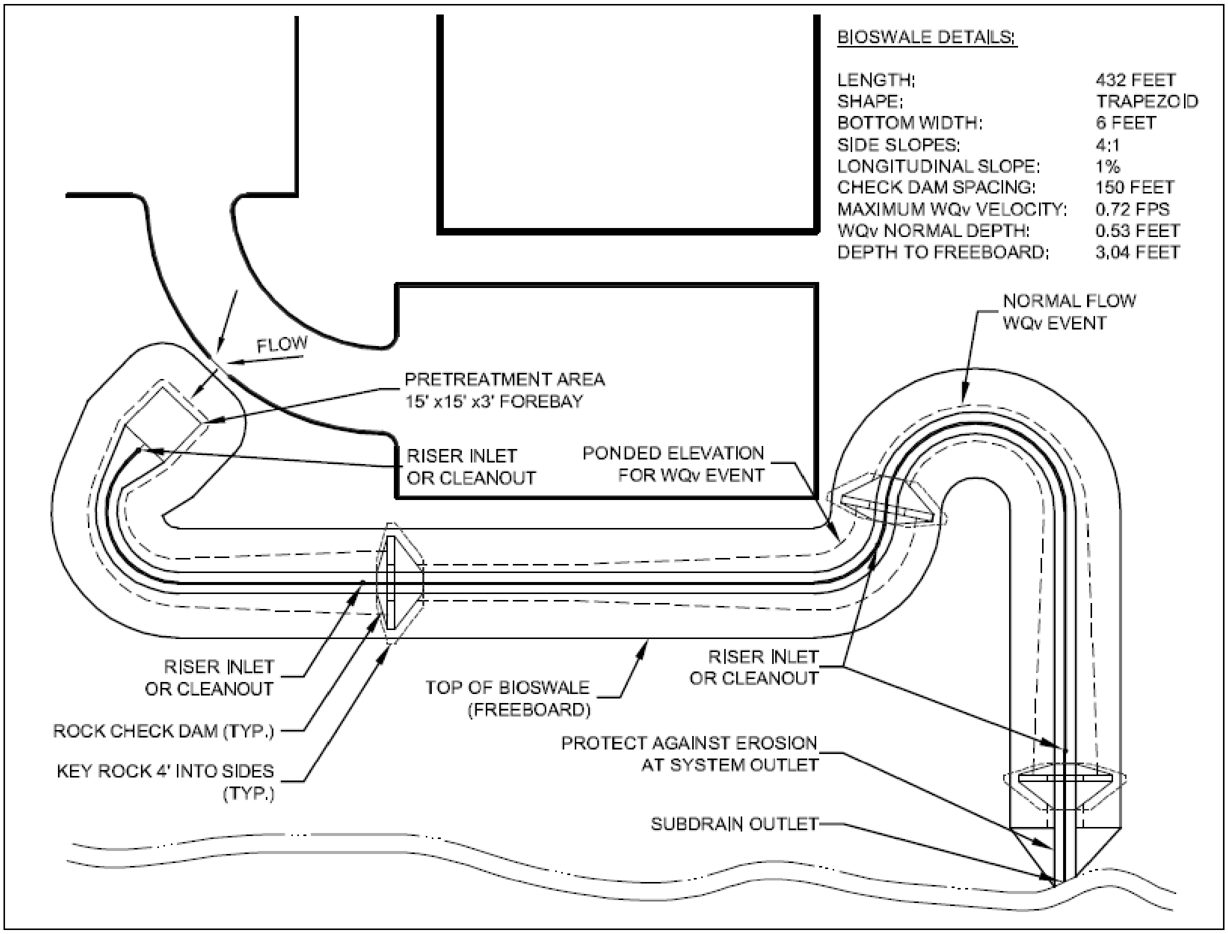
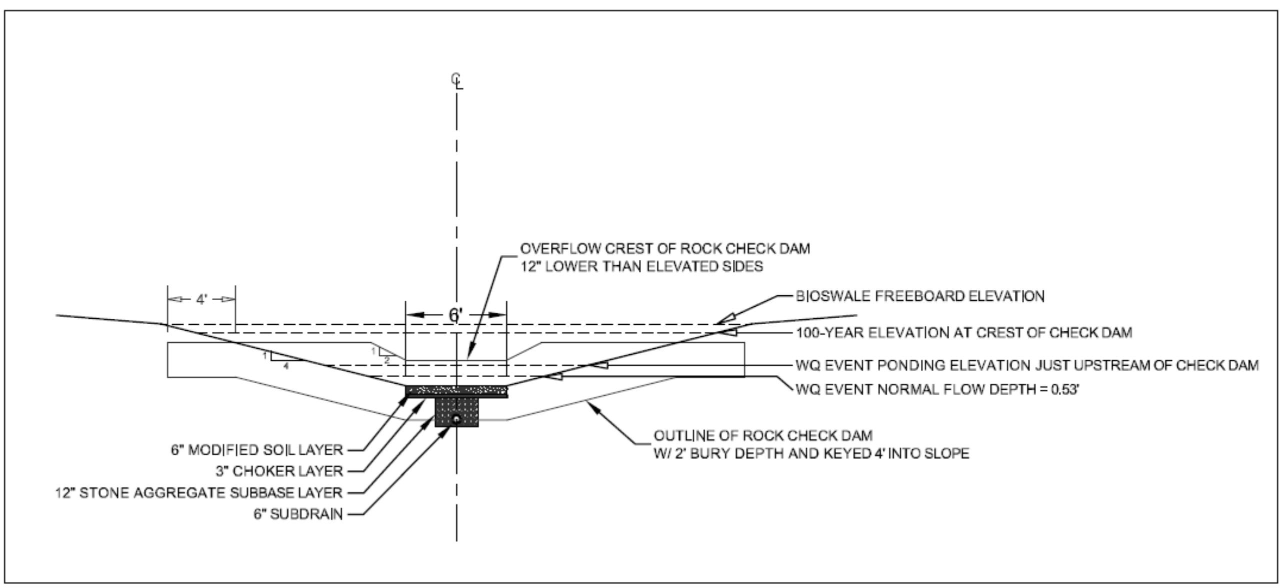


Figure 8: Cross-section for project example



C. Construction

1. **Preconstruction meeting.** Design and installation staff should meet prior to any on-site construction to discuss the placement of all permanent stormwater management practices. This discussion should focus on minimizing soil compaction, identifying areas where infiltration practices will be placed, staging of construction to ensure site stabilization prior to the installation of bioswales, and a discussion of the design details associated with the installation of the swale systems.

2. **Staging.** The construction project should be staged so the bioswale is installed during the final construction stage. Prior to swale installation, all soils within the area that will drain to the swale must be stabilized with permanent vegetation and/or other erosion, sediment, and velocity controls. If the swale is to be used as a sediment basin prior to use as a swale, it should be excavated to the dimensions, side slopes, and 1 foot above the bottom of the modified soil layer elevations shown on the drawings.

3. **Construction considerations:**

   a. **Staking.** The bioswale area should be staked prior to any site construction to minimize traffic and compaction.

   b. **Construction site stabilization.** Contributing drainage areas should be permanently stabilized against erosion and sedimentation prior to construction of a bioswale.

   c. **Weather.** Construction of the bioswale should not begin or be conducted during rainy weather resulting in saturated soil conditions.

   d. **Excavation.** After all vegetation is established within the drainage area of the bioswale, all sediment in the swale should be completely removed.

      Excavators and backhoes, operating on the ground adjacent to the bioswale, should be used to excavate the cell area to the greatest extent possible. Otherwise, excavation should be performed using low ground-contact pressure equipment.

      Any discharge of sediment that affects the performance of the bioswale will require reconstruction of the swale as originally specified to restore its defined performance.

   e. **Compaction avoidance and remediation.** Heavy equipment should not be used within the perimeter of the water quality swale before, during, or after placement of the modified soil layer in bioswales. After placement of the under drain system and before the modified soil layer is placed, the bottom of the excavation should be roto-tilled to a minimum depth of 6 inches to alleviate compaction. Should the soils be severely compacted, ripping or deep tillage equipment may be needed to break up the compacted layers prior to roto-tilling.

   f. **Placement of modified soil layer.** Any ponded water should be removed from the bottom of the excavation and discharged to a vegetated area but not discharged directly to a storm sewer.

      The modified soil layer should be placed and graded using low ground-contact pressure equipment, or by excavators and/or backhoes operating on the ground adjacent to the bioswale. Heavy equipment should not be used within the perimeter of the bioswale before, during, or after placement of this layer.

The modified soil layer should be placed in horizontal layers not to exceed 12 inches for the entire area of the bioswale. It should be saturated over the entire area of the cell after each lift of the modified soil layer is placed, until water flows from the underdrain, to lightly consolidate the mixture. Water for saturation should be applied by spraying or sprinkling in a manner to avoid separation of the BSM components. An appropriate sediment control device should be used to treat any sediment-laden water discharged from the underdrain during this process.

If the modified soil layer becomes contaminated with sediment or other deleterious material during, or after, construction of the cell, the contaminated material should be removed and replaced with uncontaminated material.

Final grading of the modified soil layer shall be performed after a 24 hour settlement period. Upon completion of final grading, the surface of this layer should be roto-tilled to a depth of 6 inches.

g. **Planting, mulch, netting.** Mulch should first be spread in bioswales prior to planting. When using wood mulch, select fibrous, hardwood mulch. Netting may be needed on top of the surface of the mulch to minimize floating of the mulch.

Information on appropriate turf grass species for Iowa can be found in the SUDAS Specifications Section 9010. *S*wale plants may require watering over several months to aid establishment, especially during drought periods.

Pesticides, herbicides, or fertilizer should not be used during landscape construction, plant establishment, or maintenance.

When small plants are used, consider delaying curb cuts or placing diversions in front of the cuts until plants are established.

1) **Plant selection and arrangement.**
Source: Rainscaping Iowa
http://www.rainscapingiowa.org/index.php/practiceslink/biocells

D. Maintenance

Bioswales require seasonal maintenance. It is imperative that they be maintained to function properly and provide continuous visual aesthetics.

Table 7: Bioswale maintenance requirements

Activity	Schedule
• Prune and thin out plants when needed. Remove weeds throughout the growing season, preferably by pulling or trimming. Replace plants when needed. • Remove trash and debris from pretreatment area and bioswale.	Fall, spring, as needed
• Inspect inflow points for clogging (off-line systems). Remove any sediment. • Inspect filter strip/grass channel for erosion or gullying. Re-seed or sod as necessary. • Plants should be inspected to evaluate their health and remove any dead or severely diseased vegetation.	Semi-annually
• Look for evidence of standing water in the riser pipe. This may be a sign of hydraulic failure.	Annually
• Replace choker layer materials on rock checks when clogged. • When ponding routinely exceeds the design drainage time then investigate to determine the cause and take corrective measures.	As necessary

Acknowledgements:

Approved by the Iowa Stormwater Partnership Technical Committee 11/7/13

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