Iowa Storm Water Management Manual

Design Standards Chapter 9- Vegetated Swale Systems

A. Introduction

Vegetated swale (open channel) systems are practices explicitly designed to capture and treat the full water quality volume (WQv) within dry or wet cells formed by check dams or other means and include systems designed to convey and treat either shallow flow (swales) or sheet flow (filter strips) runoff. These BMPs are commonly referred to as biofilters, since the grasses and vegetation filter the stormwater runoff as it flows through and/or over the vegetated surface (US EPA, 1999b). A degree of treatment, storage, and infiltration can be provided by conveying stormwater runoff in vegetated systems and can help to reduce the overall volume of stormwater runoff generated from a project drainage area. Open channel vegetated systems can be an effective alternative to traditional curb-and- gutter and storm sewer conveyance systems.

Vegetated swales (also known as bio-swales, enhanced swales, or water quality swales) are vegetated open channels that receive directed flow and convey stormwater. Swales intercept both sheet and concentrated flows and convey these flows in a concentrated, vegetation lined channel.

Filter strips (also known as bio-filtration strips or vegetated filter strips (VFS) are vegetated sections of land over which stormwater flows as overland sheet flow. Grass filter strips intercept sheet runoff from the impervious network of streets, parking lots, and rooftops, and divert stormwater to a uniformly-graded meadow, buffer zone, or another downstream structural BMP.

Vegetated swales and vegetated filter strips can function as pre-treatment systems for water entering bioretention system or other BMPs. If these systems are to succeed in filtering pollutants from the water column, the planting design must consider the hydrology, soils, and maintenance requirements of the site. Removal of pollutants is accomplished through the process of filtration by vegetation, sedimentation, adsorption onto soil particles, infiltration into the soil surface, and deeper percolation into the soil strata. Vegetated strips and swales are primarily effective at removing debris and suspended solids. Dissolved materials are primarily removed by adsorption within the soil profile.

B. Types of vegetated practices

1. **Vegetated swales.** A vegetated swale is an infiltration and filtration method that is typically used to provide pre-treatment before stormwater runoff is discharged to treatment systems. There are three types of vegetated swales:

   a. **Grass swales.** A grass swale, frequently referred to as a grassed waterway, is a broad and shallow earthen channel vegetated with erosion-resistant and flood-tolerant grasses. Grass swales have traditionally been used as a low-cost stormwater conveyance practice in low-to- medium density residential developments (e.g., ¼ to ½-acre lots) to safely move concentrated flow and as a pre-treatment practice upstream of other BMPs. A number of Iowa jurisdictions have typical rural road section standards that allow the use of grass swales within the public right-of-way. Figure C9-S1- 1 provides a representative typical section, including both a cross section and plan view of a grass swale.

   b. **Dry swale with filter media.** The dry swale (or bio-swale) consists of an open channel that has been modified to enhance its water quality treatment capability by adding a filtering medium consisting of a soil bed with an underdrain system (CRC, 1996). The dry swale system is sized to accept the entire WQv and allow it to be filtered through the treatment medium and/or infiltrate through the bottom of the swale. The dry swale system is designed to drain down between storm events within about one day. Since this system is dry most of the time, it is the preferred system for residential applications. The water quality treatment mechanisms are similar to bioretention practices, except that the pollutant uptake is likely to be more limited since only a grass cover crop is available for nutrient uptake. Figure C9-S1- 2 illustrates the design components of the dry swale with filter media (MDE, 2000).

   c. **Wet swales.** The wet swale (or wetland channel) also consists of a broad open channel capable of temporarily storing the WQv, but does not have an underlying filtering bed (CRC, 1996). The wet swale is constructed directly within existing soils and may or may not intercept the water table. Like the dry swale, the WQv within the wet swale should be stored for approximately 24 hours. The wet swale has water quality treatment mechanisms similar to stormwater wetlands, which rely primarily on settling of suspended solids, adsorption, and uptake of pollutants by vegetative root systems. These systems are often called wetland channel systems since they are basically a linear shallow wetland system. Figure C9-S1- 3 illustrates the design components of the wet swale (MDE, 2000).

Vegetated grass swales have a number of desirable attributes with respect to total stormwater management (MDE, 2000, ASCE, 1998, CRC, 1996 and Yu, 1993). These attributes include:

- Slower flow velocities than pipe systems, resulting in longer times of concentration and corresponding reduction of peak discharges.
- Ability to disconnect directly connected impervious surfaces such as driveways and roadways, thus reducing the computed runoff curve number (CN) and peak discharge (see Chapter 3).
- Filtering of pollutants by grass media and infiltration of runoff into the soil profile, thus reducing peak discharges and providing additional pollutant removal.
- Uptake of pollutants by plant roots.

2. **Vegetative filter strips and buffers.** Vegetated filter strips (VFS) are areas of land with vegetative cover that are designed to accept runoff as overland sheet flow from upstream development. They can be constructed or existing vegetated buffer areas can be used. Dense vegetative cover facilitates sediment attenuation and pollutant removal. Unlike grass swales, VFS are effective only for overland sheet flow and provide little treatment for concentrated flows. Grading and level spreaders can be used to create a uniformly-sloping area that distributes the runoff evenly across the filter strip (Haan et al., 1984, Hayes et al., 1984, Barfield and Hayes, 1988 and Dillaha et al., 1989). Filter strips have been used to treat runoff from roads and highways, roof downspouts, very small parking lots, and pervious surfaces. Filter strips are often used as pre-treatment for other structural practices, such as infiltration basins and infiltration trenches. Figure C9-S1- 4 illustrates the primary design components of the filter strip (CRC, 1996).

3. **Bioretention.** The bioretention concept was originally developed as an alternative to traditional BMP structures (Clar et al., 1993 and 1994). Bioretention is a practice that manages and treats stormwater runoff using a conditioned planting soil bed and planting materials to filter runoff stored within a shallow depression. The method combines physical filtering (filtration process), adsorption with biological processes; and, on sites with moderately permeable soils, some portion of the WQv in these systems can be infiltrated into the soil profile. The system consists of a flow regulation structure, pre-treatment filter strip or grass swale, sand bed, pea gravel overflow curtain drain, shallow ponding area, surface organic layer of mulch, planting soil bed, plant material, gravel underdrain system, and an overflow system. A more detailed description and design procedure for bioretention systems is included in Chapter 5. The primary design components of the bioretention system are included in Figure C9-S1- 5 for comparison to the other vegetated systems (MDE, 2000).

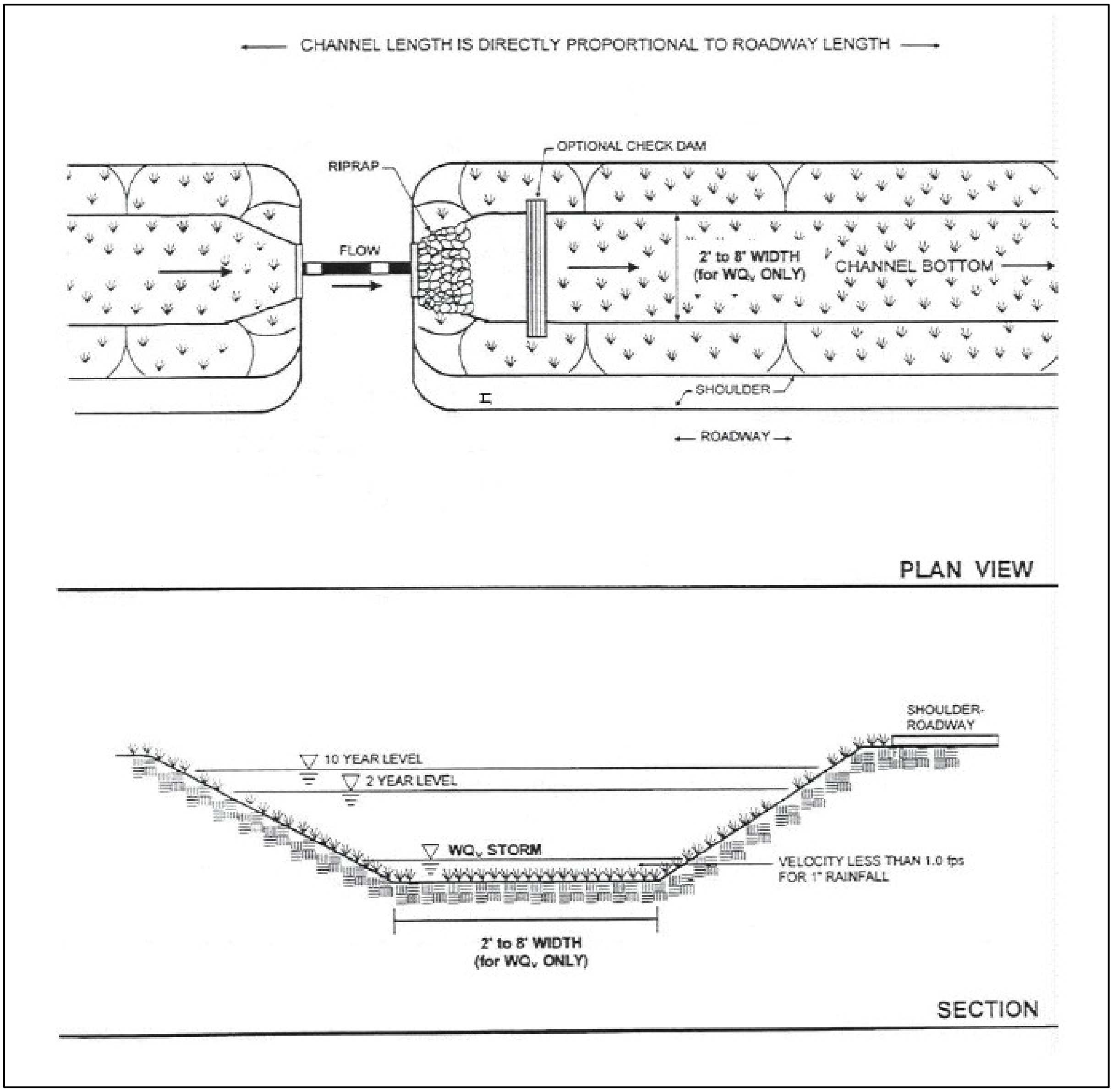


Figure C9-S1- 1: Grass swale
Source: MDE, 2000

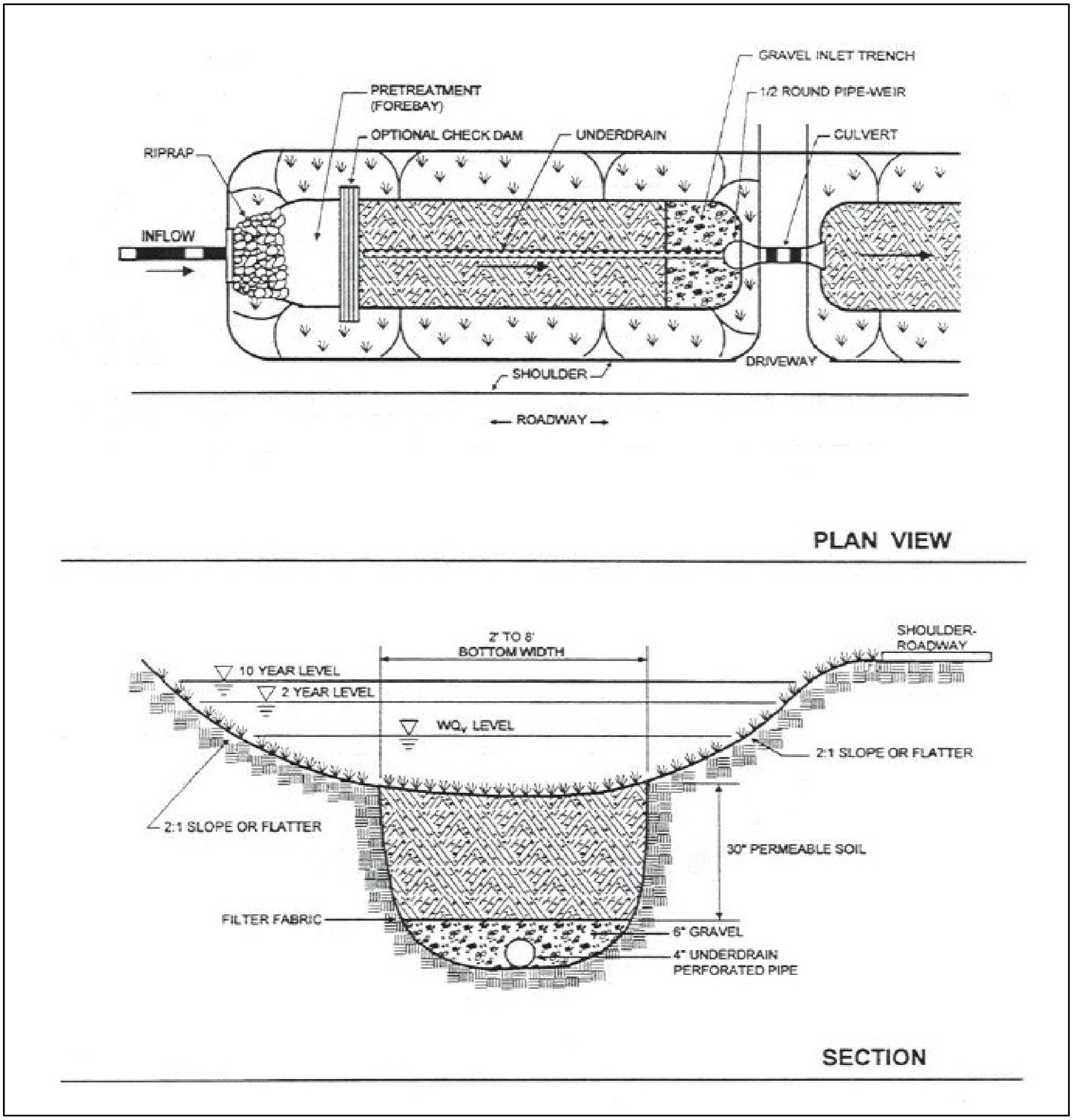


Figure C9-S1- 2: Dry swale with filter media
Source: MDE, 2000

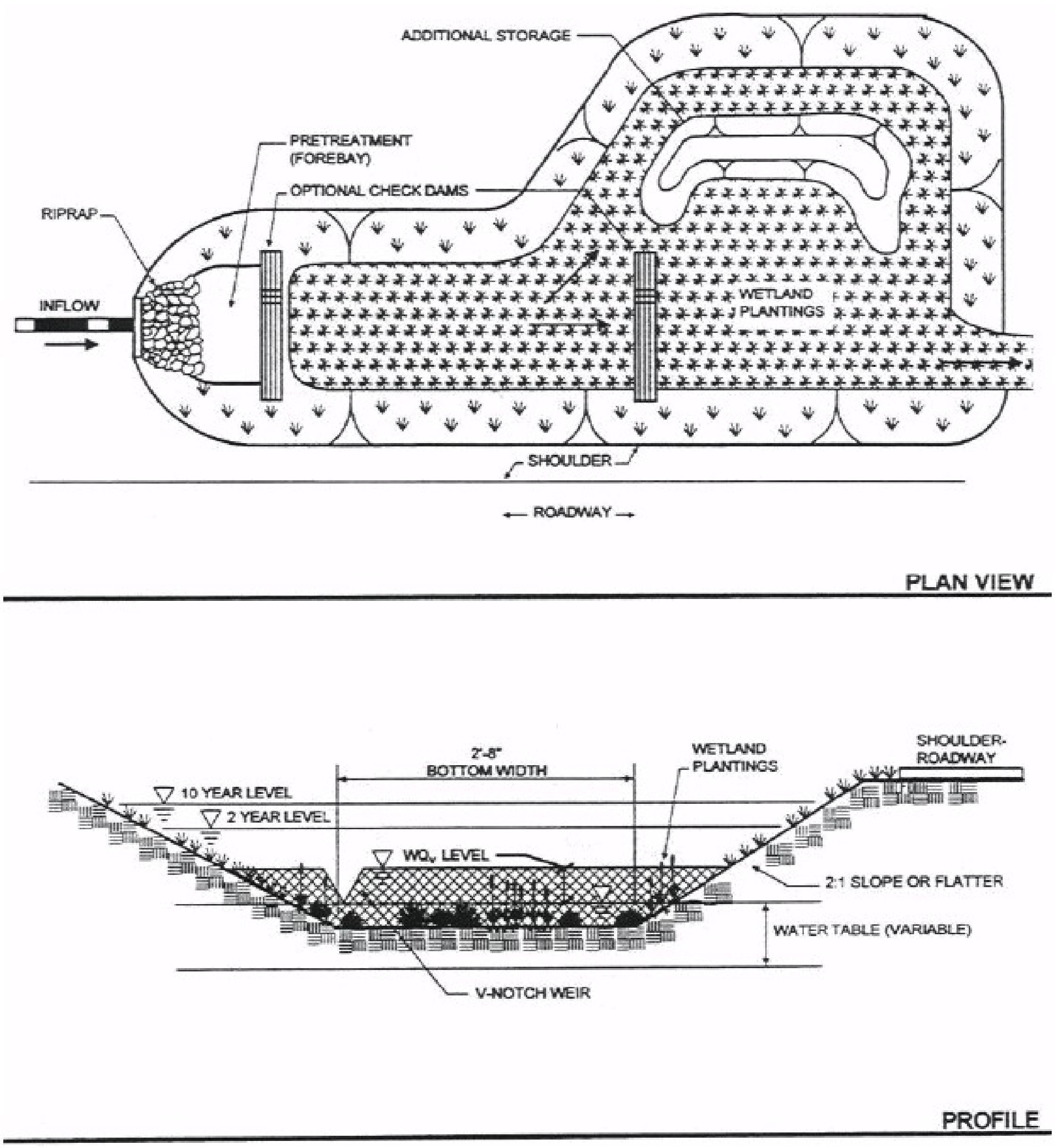


Figure C9-S1- 3: Wet swale
Source: MDE, 2000

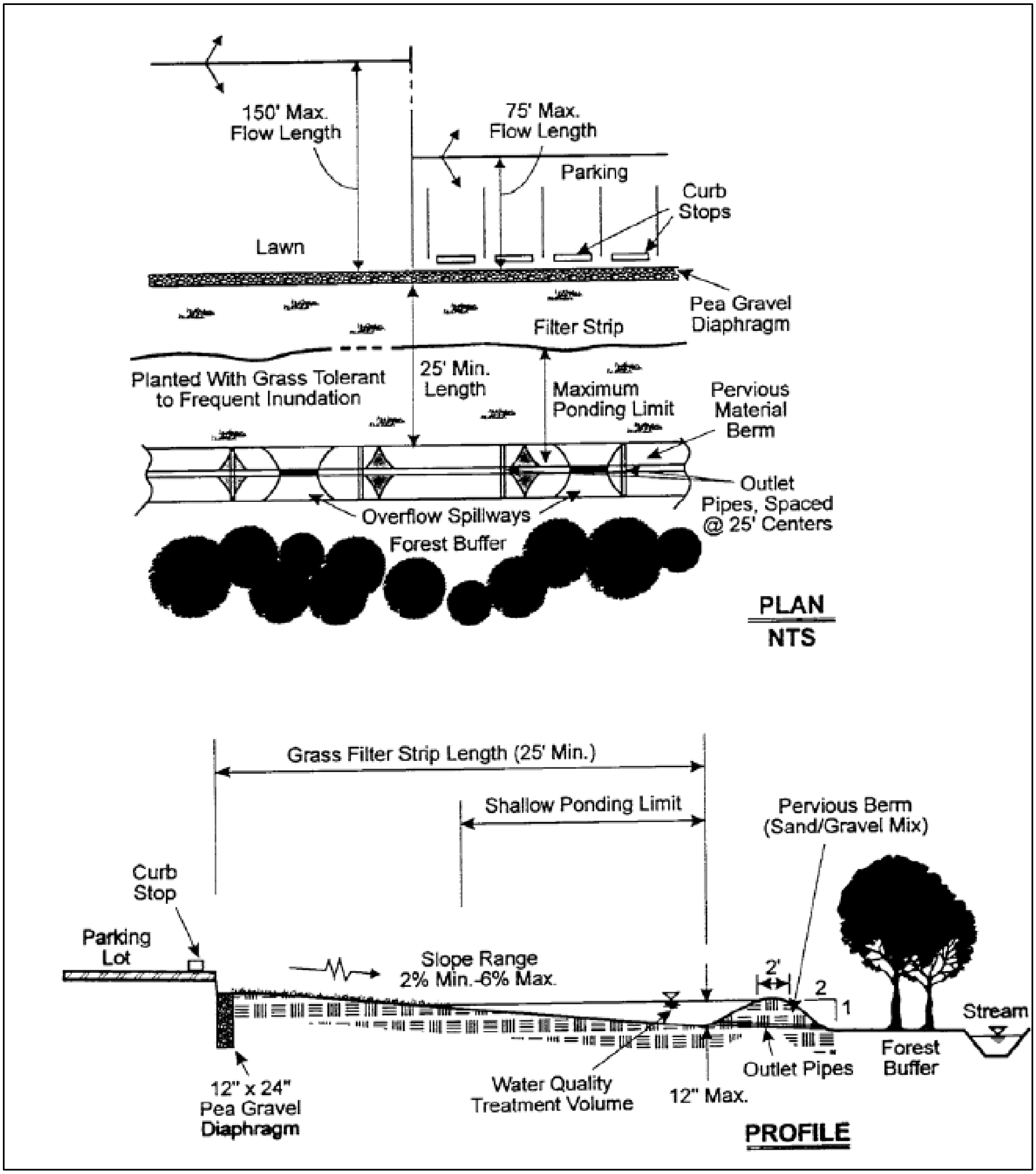


Figure C9-S1- 4: Filter strip with berm
Source: Claytor and Schueler, CRC, 1996

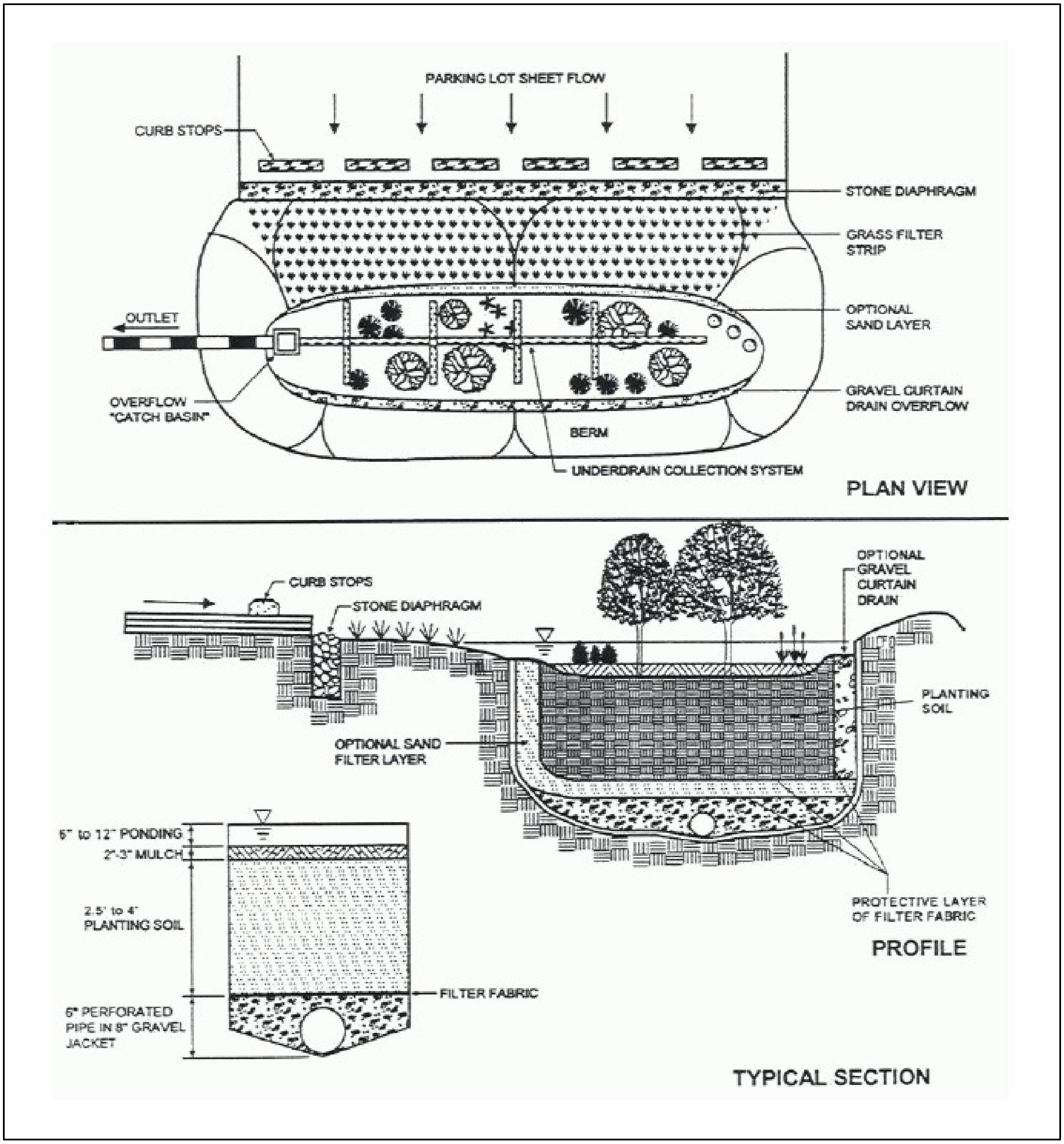


Figure C9-S1- 5: Bioretention system
Source: MDE, 2000

C. Comparative pollutant removal capability

The pollutant removal capabilities for biofilter-type BMPs are summarized in Table C9-S1- 1. Biofilters are similar with respect to performance. For example, all typically report relatively high removal rates of suspended sediment, ranging from 68% for the grass swale to 90% or more for the dry swale and the bioretention cell.

Table C9-S1- 1: Estimated pollutant removal capability of biofilters (%)

Biofilter	Data	TSS	TN	TP	Bacteria	Metals	Hydrocarbons
Grass Swale	1	68	NA	29	Negative	Cu-42%; Zn -45%	*
Dry Swale	1	93	92	83	*	Cu-70%; Zn -86%	*
Wet Swale	1	74	40	28	*	Cu-11%; Zn -33%	*
Filter Strip	2	70	30	10	*	40-50%	*
Bioretention	3,4	86	43	71-90	*	Cu-93%; Zn-99%; Pb-99%	COD–97% Oil & Grease -67%

NA – Not applicable
* - no data available

Sources: (1) Winer, 2000 (2) CRC, 1996 (3) Yu, et al., 1999 and (4) Davis et al., 1998

D. Suitability and selection considerations

Three factors are considered in selecting the appropriate vegetated system for a particular development site:

- Compatibility of the vegetated system with the land use type
- Compatibility of the system with site conditions such as space consumption, available head, cost, or maintenance consideration
- Effectiveness of the system design in removing the key pollutants of concern

Usually by the time all three factors are considered, the options are narrowed down to one or two design options. The designer can then compare the criteria for the remaining options and select one based on cost and effectiveness.

E. Land use factors

As a group, vegetative biofilter systems can be applied to a diverse range of development conditions. However, individual designs are limited to a much narrower range. These common development situations include urban retrofit sites, parking lots, roads and streets, small residential subdivisions, and backyard/rooftop drainage. Table C9-S1- 2 provides a matrix illustrating the most economical and feasible biofilter designs for each of these five broad categories of development, as well as those that are not applicable. In urban retrofit settings where space is at a premium, the bioretention cell has proven to be one of the most versatile. In these types of urban retrofit cases, the space requirements of grass swales, swales and filter strips are so great that they are often eliminated from consideration.

Table C9-S1- 2: Land use and vegetated systems suitability

Land use	Suitability of vegetated system
Urban retrofit	Bioretention cell has proven very versatile for use in retrofit conditions. Swales are usually not well-suited
Parking lots	Bioretention cell is well-suited for use in parking lots. Swales may be suitable under certain conditions (space, soils, water table). Filter strips can be effective.
Roads	City streets in high-density urban areas generally do not provide enough space for most vegetated biofilter systems. Suburban areas, in particular large- to medium-lot size subdivisions, can accommodate all of the vegetated biofilter systems.
Highway	Highways may accommodate if sufficient space is available in median or side slopes. Grass swales and filter strips can most often be integrated into the roadway configuration and provide a logical treatment option.
Residential	Low-density residential provides opportunities for all vegetated biofilter types. High-density residential may provide limited opportunities based on space limitations.
Rooftops	Roof drain disconnections to filter strips or bioretention areas are recommended where feasible.

F. Site conditions

Table C9-S1- 3 compares how each vegetated biofilter design rates with respect to a number of site conditions, including media, water table, drainage area, slope, head, and required area.

Table C9-S1- 3: Physical site conditions and biofilter suitability

System type	Media	Water table Depth (ft)	Maximum drainage area (acres)	Maximum slope (%)	Head (ft)	Ratio size to drainage area (%)
Grass swale	Soil	2	5	6	2	6.5
Dry swale	Engineered soil mix	2	5	6	3-6	10-20
Wet swale	Soil	Below water table	5	6	1	10-20
Filter trip	Soil	2	N/A	6	N/A	10-20
Bioretention	Engineered soil mix	2	2	15	5	5-10

Notes:
N/A – not applicable
Media – key evaluation factors are based on initial determination the NRCS HSG at the site. A more detailed geotechnical investigation is usually required for infiltration feasibility and during design to confirm permeability and other soil factors.
Water table depth – the minimum depth to the seasonally high water table from the bottom or invert of the BMP. *Maximum drainage area* – the recommended maximum allowable drainage area for a practice. When the drainage area exceeds the recommended maximum, some design judgment may be applied or multiple structures can be installed.
Head – an estimate of the elevation difference required at the site form the inlet to the outlet to allow for gravity flow.
Ratio size to drainage area – indicates percentage of the total drainage area required for the BMP.
Source: Adapted from MDE, 2000

G. Flow regulation

The four design variations presented in this section are primarily online stormwater treatment practices. The inherent nature of the practices and their applications for use do not fit with many offline applications. Clearly, it is best to divert the WQv into the practice wherever possible, and bypass the larger storms around the facility. The grass swale and dry and wet swales can receive runoff from concentrated sources (pipe outfalls), as well as from lateral sheet flow along the length of the practice. An isolation/diversion structure within the drainage network is the preferred method for diverting concentrated flows prior to entering these treatment practices. The filter strip receives runoff through sheet flow from impervious or pervious surfaces and is commonly designed as an online practice. It may be possible, through site grading and other design techniques, to provide an overflow diversion that bypasses larger flows around the facility. However, since the filter strip drainage area is often limited by the flow path, the volume of high-flow runoff will not generally be excessive, and there should be little need to design the system as an offline practice.

H. Pre-treatment

As with all other filtering practices, pre-treatment is necessary to extend the operational life of the practice, as well as increasing the pollutant removal capability. All four design variations have incorporated nominal pre-treatment as a component of the system design. The difference between these practices and other filtering practices is that the pre-treatment component is more qualitative in nature and is an integral part of the practice itself (e.g., the side slopes of the grass swale). The design components for pre-treatment which are specific to the four design variations are presented below (from Clayor and Schueler, CRC, 1996). With the exception of sizing a forebay at the initial inflow point, there are no specific, quantitative sizing criteria for these pre-treatment components.

1. **Grass swale, dry swale, and wet swale.**
   - A shallow forebay is provided at the initial inflow point of the channel. The volume of this forebay should equal approximately 0.05 inches per impervious acre of drainage.
   - A pea gravel diaphragm is recommended along the top of the channel to provide pre-treatment for lateral flows entering the practice.
   - Mild side slopes (≤3:1) provide additional pre-treatment for lateral flows.

2. **Filter strip.**
   - A pea gravel diaphragm is recommended along the top of the slope.
   - The uphill area, above the shallow ponding limit provides additional pre-treatment. A summary of the design criteria for vegetated open channel practices is provided in Table C9-S1- 4.

Table C9-S1- 4: Design criteria summary for vegetated open channel practices

Type of practice	Design Criteria by System Component			
	Flow regulation quantity/ method	**Pre-treatment quantity/method**	**Filter bed and media**	**Overflow**
Grass swale	Online, rate based on peak Q_{wq} for WQv, velocity <1.5 fpsec.	Pea gravel diaphragm and vegetated filter strip; forebay at inflow, no minimum volume.	Rate-based design, minimum residence time =10 min. Depending on slope, treatment area approx. =6.5% of impervious drainage area. Grass surface/soil interface.	Online flow, sized to treat WQv with velocity <1.5 fpsec, 2-year non-erosive velocities (<4.0-5.0 fpsec), adequate capacity for 10-year storm with 6-inch freeboard.
Dry swale	Online volume based on WQv.	Pea gravel diaphragm and vegetated filter strip; forebay at inflow, no minimum volume.	Volume-based design to retain WQv. Depending on slope and depth, treatment area approx. =16% of impervious drainage area. 30-inch thick planting soil bed, consisting of 50% soil/50% sand mix.	Online flow, sized to treat WQv, 2-year non-erosive velocities (<4.0-5.0 fps), adequate capacity for 10-year storm with 6-inch freeboard.
Wet swale	Online volume based on WQv.	Pea gravel diaphragm and vegetated filter strip, forebay at inflow, no minimum volume.	Volume-based design to retain WQv. Depending on slope and depth, treatment area approx. =16% of impervious drainage area. Grass/wetland vegetation surface/soil interface.	Online flow, sized to treat WQv, 2-year non-erosive velocities (<4.0-5.0 fps), adequate capacity for 10-year storm with 6-inch freeboard.
Filter strip	Online volume based on WQv.	Pea gravel diaphragm; no minimum volume.	Volume-based design to retain WQv. Depending on slope and depth, treatment area approx. =100% of impervious drainage area. Grass surface/soil interface.	Online flow, sized to treat WQv, all other flows overflow berm.

Grass swale (VA DCR, 1999)

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

Description: Grass swales are designed to convey stormwater runoff at a non-erosive velocity, as well as enhance its water quality through infiltration, sedimentation, and filtration. Check dams can be used within the swale to slow the flow rate, promote infiltration, and create small, temporary ponding areas. The vegetation covering the side slopes and channel bottom provide a filtration surface as the runoff is slowly conveyed to a downstream discharge location. The vegetation also serves to reduce flow velocities.

Typical uses:
- Manages runoff from residential sites, parking areas, and along perimeter of paved roadways.
- Located in a drainage easement at the rear of side of residential parcels.
- Used as a pre-treatment conveyance for other water quality BMPs.
- 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:
- Mitigates runoff from impervious surfaces; removes sediment and pollutants to improve water quality
- Reduce runoff rate and volume in highly impervious areas; reduced runoff velocity.
- Provides for groundwater recharge if design and site soils provide sufficient infiltration.
- Good option for small area retrofits for residential or institutional areas of low to moderate density; linear configuration works well with highway or residential street applications.

Disadvantages/limitations:
- Cannot alone achieve 80% goal reduction of TSS.
- Sediment and pollutant removal sensitive to proper design of slope and maintaining sufficient vegetation density; possible re-suspension of sediment.
- Limited to small areas (<5 acres); cannot be used on steep slopes (>6%).
- Higher maintenance than curb and gutter systems.

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

A. Description

Grass swales, also called biofilters, are typically designed to provide nominal treatment of runoff as well as meet runoff velocity targets for the water quality design storm. Grass swales are well-suited to a number of applications and land uses, including treating runoff from roads and highways and pervious surfaces.

Grass swales differ from the enhanced dry swale design in that they do not have an engineered filter media to enhance pollutant removal capabilities, and therefore have a lower pollutant removal rate than a dry or wet (enhanced) swale. Grass swales can partially infiltrate runoff from small storm events in areas with pervious soils. When properly incorporated into an overall site design, grass swales can reduce impervious cover, accent the natural landscape, and provide aesthetic benefits.

When designing a grass swale, the two primary considerations are channel capacity and minimization of erosion. Runoff velocity should not exceed 1 fps during the peak discharge associated with the water quality design rainfall event, and the total length of a grass swale should provide at least five minutes of residence time. To enhance water quality treatment, grass swales must have broader bottoms, lower slopes, and denser vegetation than most drainage channels. Additional treatment can be provided by placing check dams across the channel, below pipe inflows and at various other points along the channel.

B. Stormwater management suitability

Grass swales can provide effective control under light to moderate runoff conditions, but their ability to control large storms is limited. Therefore, they are most applicable in low- to moderately-sloped areas, or along highway medians as an alternative to ditches or curb and gutter drainage (Boutiette and Duerring, 1994). Their performance diminishes sharply in highly urbanized settings, and they are generally not effective enough to receive construction stage runoff where high sediment loads can overwhelm the system (Schueler et al., 1992). Grass swales are often used as a pre-treatment measure for other downstream BMPs, particularly infiltration devices (Driscoll and Mangarella, 1990). Grass swales are typically shallow, vegetated, man-made conveyance channels designed such that the bottom elevation is above the water table to facilitate the infiltration of runoff to the soil. The vegetation covering the side slopes and channel bottom provide a filtration surface as the runoff is collected and slowly conveyed to a downstream discharge location. Swales provide additional treatment of the stormwater runoff as water moves through a subsoil matrix and infiltrates into the underlying soils. The vegetation also serves to reduce flow velocities. Swales can be either dry or wet; dry swales are more desirable where standing water is not wanted, such as residential areas; wet swales can be used where standing water does not create a nuisance and where the groundwater is close enough to the surface to maintain a shallow permanent pool between storm events. An advantage of wet swales is the ability to include wetland vegetation to assist in pollutant removal (US EPA 1999b).

C. Pollutant removal capabilities

Pollutants are removed in swales by the filtering action of grass, deposition in low velocity areas, or by infiltration into the subsoil. The primary pollutant removal mechanism is through sedimentation of suspended materials. Therefore, TSS and adsorbed metals are most effectively removed through a grass swale. Removal efficiencies reported in the literature vary, but generally fall into the low to medium range, with some swale systems recording no water quality effects at all. Table C9-S2- 1 presents pollutant removal efficiencies for swale lengths of 200 feet and 100 feet. Although research results varied, these data clearly indicate greater pollutant removal at longer swale lengths. In general, the current literature reports that a well-designed, well-maintained swale system can be expected to remove 70% of TSS, 30% for total phosphorus (TP), 25% for total nitrogen (TN), and 50-90% for trace metals (Barret et al., 1993 and GKY and Associates, Inc., 1991). The nitrogen removals may be fairly optimistic, given that studies conducted by Yousef et al. (1985) and others produced negative nitrogen removal in many cases. It is theorized that the outwelling of nitrogen from grass clippings and other organic materials from the swale produced these results.

Seasonal differences in swale performance can be important. In temperate climates, fall and winter temperatures force vegetation into dormancy, thereby reducing uptake of runoff pollutants, and removing an important mechanism for flow reduction. Decomposition in the fall and the absence of grass cover in the winter can often produce an outwelling of nutrients, and exposes the swale to erosion during high flows, increasing sediment loads downstream. Pollutant removal efficiencies for many constituents can be markedly different during the growing and dormant periods (Driscoll and Mangarella, 1990).

Table C9-S2- 1: Swale pollutant removal efficiencies

Pollutant removal efficiencies (%)								
Design	**Solids**	**Nutrients**		**Metals**			**Other**	
	TSS	TN	TP	Zn	Pb	Cu	FOG	COD**
200-ft swale	83	25*	29	63	67	46	75	25
100-ft swale	60	*	45	16	15	2	49	25

*Some swales (100-ft systems) show negligible or negative removal for TN
**Limited data
Sources: Barret et al., 1993; Schueler et al, 1991; Yu, 1993; and Yousef et al., 1985

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment.

- Total suspended solids: 50%
- Total phosphorus: 25%
- Total nitrogen: 20%
- Fecal coliform: insufficient data
- Heavy metals: 30%

D. Application and feasibility

The grass swale consists of a broad, mildly-sloped open channel designed to maintain a minimum residence time of 10 minutes for the water quality storm (Figure C9-S2- 1). Grass swales have traditionally been utilized only for stormwater conveyance purposes. However, the design provides capacity to convey a larger storm (usually the 10-year frequency storm); as well as protection against erosion for smaller, more frequent storms (usually the 2-year event). Water quality treatment in standard grass swales is provided by managing the slope and vegetation in the channel to slow the velocity to ~1 fps for the water quality design storm (≤1.25 inches). The design for a grass swale is flow-rate based.

E. Grass swales for pre-treatment

A number of other structural controls, including bioretention areas and infiltration trenches, may utilize a grass swale as a pre-treatment measure. The length of the grass swale depends on the drainage area, land use, and channel slope. Table C9-S2- 2 provides sizing guidance for grass swales for a 1-acre drainage area. The minimum length of a grass swale should be 20 feet.

Table C9-S2- 2: Grass swale sizing guidance

Parameter	**Upstream imperviousness**					
	≤33%		*34-66 %*		*≥67%*	
Slope (max = 4%)	<2%	>2%	<2%	>2%	<2%	>2%
Grass swale minimum length (feet)*	25	40	30	45	35	50

*assumes 2-foot wide bottom width

Source: CRC, Claytor and Schuler, 1996

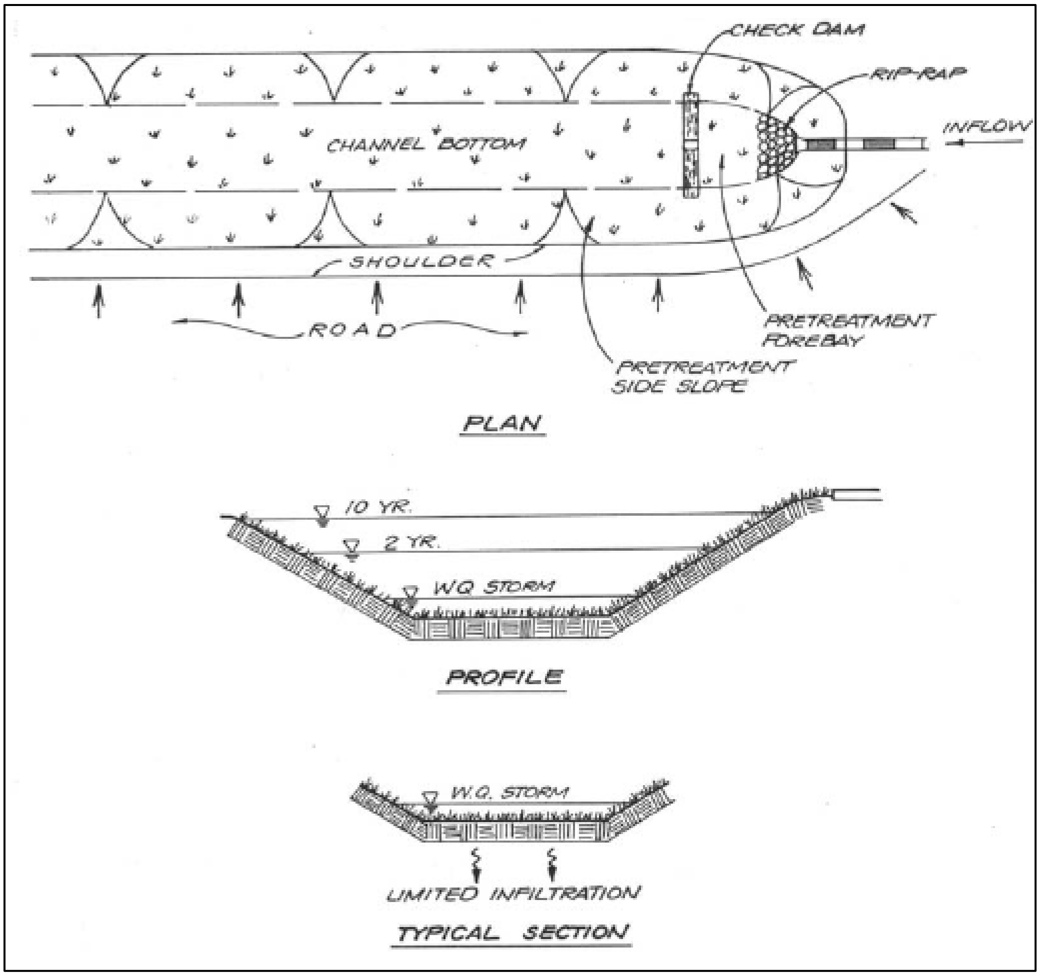


Figure C9-S2- 1: Configuration of grass swale
Source: CRC, Claytor and Schuler, 1996

F. Check dams

Check dams are used in swales for two reasons: to increase pollutant removal efficiency and/or to compensate for steep longitudinal slope. The dams should be installed perpendicular to the direction of flow and anchored into the slope of the channel. The side slopes of the check dams should be between 5:1 and 10:1 to facilitate mowing operations. The berm height should not exceed 2 feet, and water detained behind the berm should infiltrate into the soils within 24 hours (Colorado Department of Transportation, 1992). Figure C9-S2- 2 shows an example of check dams erected at regular intervals to maintain a shallower, uniform slope (VA DEC, 1999). With this configuration, energy-dissipating and flow-spreading riprap is often used across check dams and for a short distance downstream at the toe of the drops. Check dams should be spaced so that the toe of the upstream dam is at the same elevation as the top of the downstream dam. Check dams can be constructed using earth, riprap, gabions, railroad ties, or pressure-treated wood logs. Figure C9-S2- 3 provides typical check dam configurations for a riprap and a half-round corrugated metal pipe check dam (VA DEC, 1999). For best performance, check dams should have a level upper surface rather than the uneven surface of a riprap check dam. Earthen check dams are not recommended, due to erosion potential and high maintenance effort.

Figure C9-S2- 2: Grass swale with check dams (berms)
Source: VA DEC, 1999

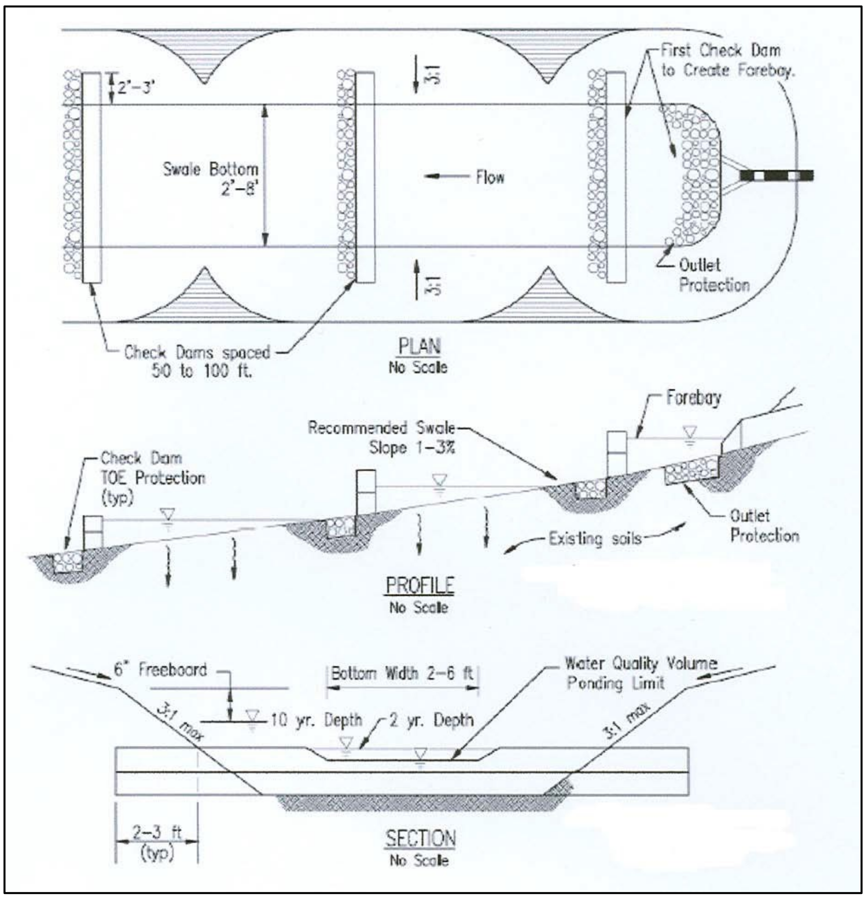


Figure C9-S2- 3: Typical swale with check dam configuration
Source: VA DEC, 1999

G. Channel design criteria

The design approach consists of three criteria for sizing grass swales for stormwater quality treatment, while also accommodating larger storms:

- The channel is initially designed, based on the treatment principles of small storm hydrology for the water quality storm (see Chapter 3, section 7).

- The channel design is then checked against the larger 2-year storm to ensure a non-erosive condition.
- Finally, the capacity for conveyance of the 10-year frequency storm is checked and a minimum freeboard is applied.

The design procedure is a rate-based sizing criteria which uses Manning's equation to compute velocities and depths, based on specified channel geometry and slope. Figure C9-S2- 4 illustrates the design components of the grass swale. The specific design considerations are presented below, and a summary is provided in Table C9-S2- 3.

1. **General design criteria.**
   a. Grass swales should generally be used to treat small drainage areas of less than 5 acres. If the practices are used on larger drainage areas, the flows and volumes through the channel become too large to allow for filtering and infiltration of runoff.
   b. Grass swales should be designed on relatively flat slopes of less than 4%, channel slopes between 1-2% are recommended.
   c. Grass swales can be used on most soils with some restrictions on the most impermeable soils. Grass swales should not be used on soils with infiltration rates less than 0.3 inches per hour if infiltration of small runoff flows is intended.
   d. A grass swale should accommodate the peak flow for the water quality design storm – Q_{wq} (see Chapter 3. section 7).
   e. Runoff velocities must be non-erosive. For the Q_{wq}, the velocity should be ≤ 1 fps. The full-channel design velocity will typically govern.
   f. A minimum five-minute residence time is recommended for the water quality peak flow. Residence time may be increased by reducing the slope of the channel, increasing the wetted perimeter, or planting a denser grass (raising the Manning's n).
   g. The depth from the bottom of the channel to the groundwater should be at least 2 feet to prevent a moist swale bottom or contamination of the groundwater.
   h. Check dams within the channel will maximize retention time (Figure C9-S2- 2).
   i. Select a grass that can withstand relatively high-velocity flows at the entrances, and both wet and dry periods. See SUDAS specifications for a list of appropriate grasses for use in Iowa.

2. **Shape.** The channel should be trapezoidal or parabolic in shape. The trapezoidal cross section is the easiest to construct and a more efficient hydraulic configuration. However, since channels tend to become parabolic in shape over time, a channel originally designed as a trapezoidal section should also be checked against parabolic sizing equations as a long-term functional assessment. The criteria presented in this section assume a trapezoidal cross section. Note that the same design principles will govern parabolic cross sections, except for the cross sectional geometry.

3. **Bottom width.** For a trapezoidal cross section, size the bottom width between 2 and 8 feet. The 2-foot minimum allows for construction considerations and ensures a minimum filtering surface for water quality treatment. The 8-foot maximum prevents shallow flows from concentrating and potentially eroding channels, thereby maximizing the filtering by vegetation. Widths up to 12 feet may be used if separated by a dividing berm or structure to avoid braiding. The bottom width is a dependent variable in the calculation of velocity based on Manning's equation. If a larger channel is needed, the use of a compound cross section is recommended.

4. **Manning's n value.** The roughness coefficient, n, varies with the type of vegetative cover and flow depth. At very shallow depths, where the vegetation height is equal to or greater than the flow depth, the n value should be approximately 0.15. This value is appropriate for flow depths up to 4 inches. For higher flow rates and flow depths, the n value decreases to a minimum of 0.03 for grass swales at a depth of approximately 12 inches. The n value must be adjusted for varying flow depths between 4 and 12 inches (see Figure C9-S2- 5 for variable n values with varying depths).

5. **Side slopes.** The side slopes should be flat as possible to aid in providing pre-treatment for lateral incoming flows and to maximize the channel filtering surface. Steeper side slopes are likely to have potential for erosion from incoming lateral flows. A maximum slope of 3:1 is recommended (33%); a 4:1 slope is preferred where space permits.

6. **Channel longitudinal slope.** The slope of the channel should be steep enough to ensure uniform flow and that which can be constructed using conventional construction equipment without ponding, but not steeper than 4%. A minimum slope of 1% is recommended.

7. **Flow depth.** Maximum depth of flow no greater than one-third of the vegetation height for infrequently mowed swales, or no greater than one-half of the vegetation height for regularly mowed swales, up to a maximum of 4 inches. The maximum flow depth for water quality treatment should be approximately the same as the height of the grass. Since most channels will be mowed relatively infrequently, the vegetation may reach heights of 6 inches or more. However, since higher grass will likely flatten during higher flows, a maximum flow depth of 4 inches is recommended for water quality design. The flow depth for the 2-year and 10-year storms will depend on the flow rate and channel geometry.

8. **Flow velocity.** The maximum flow velocity for water quality treatment should be sufficiently low to provide adequate residence time within the channel. A maximum flow velocity of 1 fps for water quality treatment is required. The maximum flow velocity for the 2-year storm should be non-erosive (a rate of 4-5 fps is generally recommended). The permissible velocities of several grass species are listed in Table C9-S2- 4. Velocity values are purely guidelines and may not always be representative of field conditions. The 10-year permissible velocity may be somewhat higher due to the low frequency of occurrence. A permissible maximum rate of approximately 7 fps for this event is recommended.

9. **Length of channel.** Generally grass swale length (for conveyance) is a function of site drainage constraints and a required length is not necessary. However, for water quality treatment, a minimum residence time of 10 minutes should be reached to facilitate filtering. The minimum length required for water quality treatment grass swales is equal to the velocity, in feet per second, multiplied by the minimum residence time of 600 seconds.

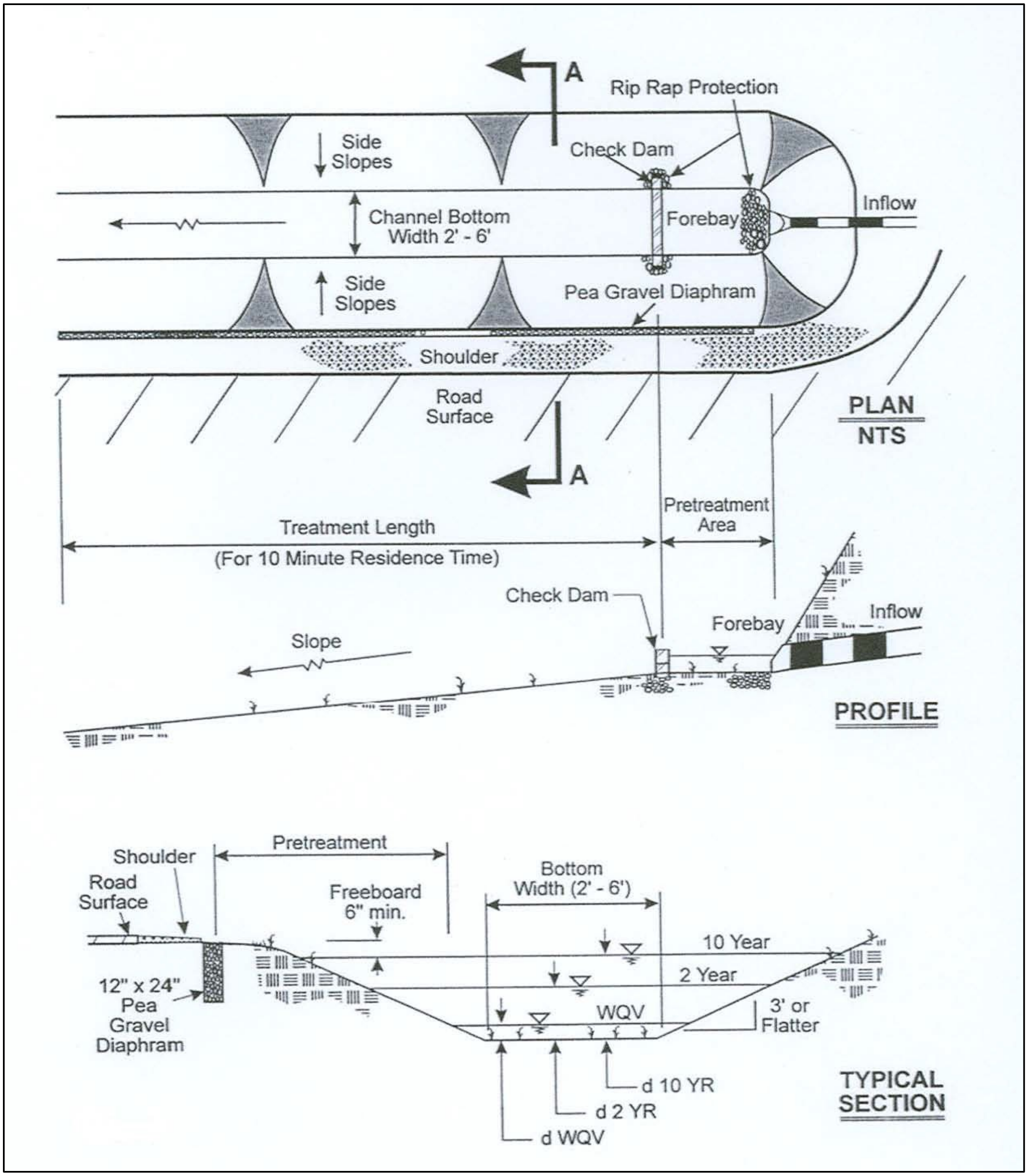


Figure C9-S2- 4: Configuration and design components of a grass swale for water quality treatment
Source: Claytor and Schuler, 1996

Table C9-S2- 3: Design criteria for trapezoidal grass swales for water quality treatment

Parameter	Design Criteria
Bottom width	2-ft minimum, 6-ft maximum*
Side slopes	3:1 or flatter
Channel longitudinal slope	1% minimum; 4% maximum
Flow depth	4 inches for water quality treatment
Manning's n value	0.15 for water quality treatment (depth <4 inches); varies from 0.15-0.03 for depths of 4-12 inches; 0.03 minimum for depths >12 inches (see Figure C9-S2- 5)
Flow velocity	1 fps for water quality treatment; 4-5 fps for 2-yr storm; 7 fps for 10-yr storm
Length	Length necessary for 10-minute residence time

*Widths up to 12 feet are allowable when using a division structure to avoid meandering concentrated flows

Source: Adapted from Claytor and Schuler, 1996

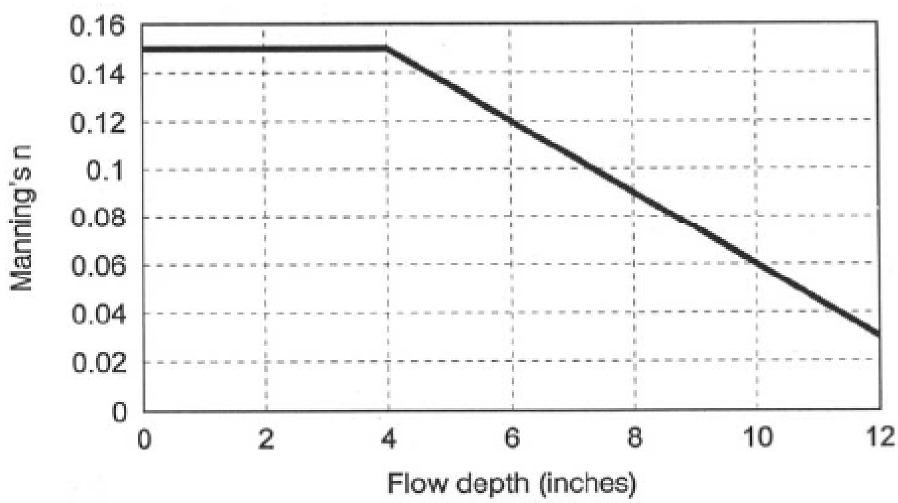


Figure C9-S2- 5: Variable Manning's n with flow depth
Source: Claytor and Schuler

Table C9-S2- 4: Selecting maximum permissible swale velocities for stability

		Maximum Velocity (fps)	
Cover Type	Slope (%)	Erosion-resistant soils	Easily-eroded soils
Kentucky blue grass; Tall fescue	0-5	6	5
Kentucky blue grass; Rye grasses	5-10	5	4
Grass – legume	0-5	5	4
Mixture	5-10	4	4
Red fescue	0-5	3	2.5

Source: Temple et al, 1987

H. Design procedure

The following steps are recommended for completing a grass swale design:

- Determine design flow rate to the system (Q_{wq})
- Determine the slope of the system
- Select a swale shape
- Determine required channel width
- Calculate the cross sectional area of flow
- Calculate the velocity of channel flow
- Calculate swale length
- Select swale location based on the design parameters
- Select a vegetation cover for the swale
- Check for swale stability

1. **Step 1: Determine design flow rate.**
   a. Determine the WQv using a design storm depth of 1.25 inches, or use the 90% rule to select rainfall depth for the water quality storm (refer to Chapter 2 and Chapter 3).
   b. Compute the peak rate of discharge (Q_{wq}) for the water quality storm, based on the procedures identified in Chapter 3, section 7. Note: This calculation can be done using WinTR-55 after a custom CN is computed using the water quality design storm depth (1.25 inches).
   c. The design storm is subject to local regulations, and thus may vary on a local basis.
   d. Unless runoff from larger events is designed to bypass the swale, consideration must be given to the control of channel erosion and destruction of vegetation. A stability analysis for larger flows (up to the 100-yr, 24-hour) must be performed. Runoff quantity and design flows can be estimated using a variety of mathematical, graphical, and computerized techniques.
   e. Use the Q_{wq} to size the channel, maintaining design criteria parameters noted in Table C9-S2- 3.
   f. Determine the velocity (fps) for the Q_{wq} and n = 0.15 for channel widths of 2 feet, 4 feet, and 6 feet, or use computer model which solves Manning's equation or other open channel flow equations.
   g. Compute 2-year and 10-year frequency storm event peak discharges using NRCS WinTR-55.
   h. Check 2-year velocity for erosive potential (adjust geometry if necessary, and re-evaluate WQv design parameters).
   i. Check 10-year depth and velocity for capacity (adjust geometry if necessary, and re-evaluate WQv and 2-year design parameters).
   j. Provide minimum freeboard above 10-year stormwater surface elevation (6 inches minimum, recommended).

2. **Step 2:** Determine the slope of the system. (See Table C9-S2- 3). The slope of the swale will be somewhat dependent on where the swale is placed, but should be between the stated criteria of 1-4%. An optimum slope of 1.5-2% is desired. With slopes less than 2%, the use of under drainage may be required. If the slope is between 4-6%, vertical drops of 6-12 inches will be required using check dams/berms at 50- to 100-foot intervals. Energy dissipating and flow spreading riprap will be needed across check dams and for a short distance downstream of the toe drops. If the slope is greater than 6%, the grade will need to be traversed to reduce the slope of any segment to below 4% preferably, or to below 6% with check dams.

3. **Step 3:** Select a swale shape. Normally, swales are designed and constructed in a trapezoidal shape, although alternative designs can be parabolic, rectangular, or triangular. Trapezoidal cross sections would be preferred because of relatively wider vegetative areas and ease of maintenance. This also avoids the sharp corners present in v-shaped and rectangular swales, and offer better stability than the vertical walls of rectangular swales. A parabolic shape is best for erosion control, but is hard to construct. Trapezoidal shapes tend to become parabolic over time, due to the growth of vegetation and settlement of solids (Horner, 1988). Unless space is a problem, the design process should begin assuming a trapezoidal shape. The remainder of the design process assumes that a trapezoidal shape has been selected. A minimum side slope of 3:1 or flatter should be used; a side slope of 4:1, or even 5:1, would be preferred. The wider the wetted area of the swale, the slower the flow.

4. **Step 4:** Determine required channel width. Estimates for channel width for the selected shape can be obtained by applying Manning's equation (Equation C9-S2- 2). Figure C9-S2- 6 presents channel geometry and equations for a trapezoidal swale, the most frequently-used shape. A Manning's n value of 0.15-0.2 is recommended for routine

swales that will be mowed with some regularity. For swales that are infrequently mowed, a Manning's n value of 0.24 is recommended. A higher n value can be selected if it is known that vegetation will be very dense. Figure C9-S2- 7 provides a range of n values.

a. **Continuity Equation.**

Equation C9-S2- 1

$$Q = VA$$

Where:
V = the mean velocity (fps)
A = the flow cross sectional area normal to the direction of the flow (ft^2).

The cross sectional area is the product of the channel width and the depth of flow in the channel. The depth of flow in the channel for a uniform discharge is the normal depth. At normal depth the slope of the invert (channel bottom), the slope of the HGL, and the slope of the EGL are equal and parallel to each other. Normal depth for a given discharge can be determined using the Manning equation. Velocities for grass swales are calculated with Manning’s equation, but the characteristic dimension now becomes the hydraulic radius.

b. **Manning’s Equation.**

Equation C9-S2- 2

$$V = \left(\frac{1.49}{n}\right) R^{2/3} S^{1/2}$$

Where:
V = the mean velocity (fps)
R = the hydraulic radius (ft)
S = the slope of the energy line (channel invert)
n = the coefficient of roughness.

Further, hydraulic radius, R, of the swale is defined as:

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

Where:
A = cross sectional area of swale (ft^2)
P = wetted perimeter of swale (ft).

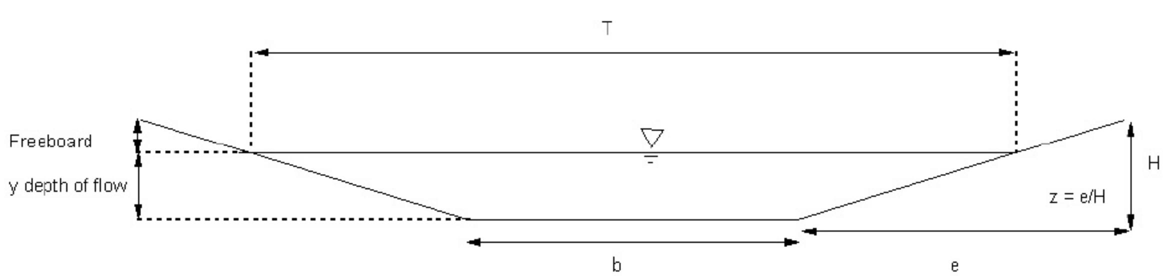


Figure C9-S2- 6: Channel and flow geometry for a trapezoid swale

c. **Side slope.**

Equation C9-S2- 3

$$Z = \frac{e}{H}$$

d. **Cross sectional area.**

Equation C9-S2- 4

$$A = by + zy^2$$

e. **Top width.**

Equation C9-S2- 5

$$T = b + 2Hz$$

f. **Wetted perimeter.**

Equation C9-S2- 6

$$P = b + 2y(1 + z^2)^{0.5}$$

g. **Hydraulic radius**

Equation C9-S2- 7

$$R = \frac{A}{P} = \frac{by + by^2}{[b + 2y(1 + z^2)^{0.5}]}$$

h. **Swale depth.**

Equation C9-S2- 8

$$H = y + freeboard$$

Where:
y = flow depth
b = bottom width
e = side width of trapezoidal channel.

Manning’s n values are not constant, but vary widely with depth of flow as shown in Figure C9-S2- 5. Vegetated channels are grouped into retardance classes A through E shown in Table C9-S2- 5. In each of these retardance classes, Manning’s n is shown as a function of product of velocity V in fps and hydraulic radius R in ft. Using these curves, Ree (1949) developed nomographs for solving Manning’s equation for each retardance class. An example is shown in Figure C9-S2- 8 for Retardance class C. Nomographs for other retardance classes are given in Haan et al. (1994).

i. Manning’s equation (Equation C9-S2- 2) can be solved for flow by combining with the continuity equation (Equation C9-S2- 1).

j. The bottom width of the trapezoid cannot be solved directly so the solution is iterative. However, the calculations can be solved fairly quickly using a spreadsheet with iterative capabilities and the ability to vary only certain variables.

k. Typically, flow depth, y, is set at 3-4 inches maximum (Table C9-S2- 3). Flow depth can also be estimated by subtracting 2 inches from the expected grass height, if the grass type and the height it will be maintained is known. Values lower than 3-4 inches can be used, but doing so will increase the computed width (T or b) of the swale. Flow depth is subject to a stability check as described below.

l. The computed bottom swale width should be between 2-8-feet. Relatively wide swales (those wider than 8 feet are more susceptible to flow channelization and are less likely to have uniform sheet flow across the swale bottom for the entire swale length. A practical minimum swale width for trapezoidal swales should also be established for ease of maintenance, e.g., to facilitate swale mowing with standard lawn mowers. Therefore, if b for a trapezoid swale is greater than 8 feet, investigate either (a) the probability for channelization given flow spreader device(s) to be used and swale maintenance practices, or (b) methods by which the design flow (Q) can be reduced. Since length may be used to compensate for width reduction (and vice versa) so that the area is maintained, the swale width can be arbitrarily set to 8 feet to continue with the analysis. If b <2 feet, set b = 2 feet and continue. Narrower widths can be used if space is very constrained.

5. **Step 5: Calculate cross sectional area of flow.** Compute the cross-sectional area (A) for the design flow, using Equation C9-S2- 4.

6. **Step 6: Calculate the velocity of the channel flow.**

   a. Using the continuity equation (Equation C9-S2- 1), the channel flow velocity can be calculated. The channel flow velocity should be less than 1 fps to prevent grasses from being flattened, which reduces filtration. A velocity lower than this maximum value is recommended to achieve the 10-minute hydraulic residence time criterion, particularly in shorter swales (at V=1 fps, a 600-foot swale is needed for a 10-minute hydraulic residence time and a 300-foot swale for a 5-minute residence time).

   b. If the value V suggests that a longer swale will be needed than space permits, investigate how the design flow Q can be reduced; or increase flow depth (y) and/or swale bottom width (b) up to the maximum allowable values and repeat the analysis.

7. **Step 7: Calculate swale length.**

   a. Compute the swale length (L) using the following equation:

Equation C9-S2- 9

$$\coloneqq V t_r \left(60 \frac{sec}{min}\right)$$

Where:
t_r = Hydraulic residence time (in minutes).

   b. Use t_r = 10 minutes for this calculation. Swale length may be a matter of local regulation, however length is directly related to achieving the goal of a 10-minute hydraulic residence time. This criterion has been shown to be the optimum value for good removal of particulates, oil, and grease. Performance data from research has indicated that shorter residence times cause a reduction in pollutant removal rates. Longer times may be required if expected pollutant removal efficiency for solids is to exceed 80%.

8. **Step 8: Select swale location.** Options for swale locations may be limited, or may be decided through processes outside the control of the designer. If this is the case, swale geometry should be maximized by the designer, using the above equations, and given the area to be utilized. If the location has not yet been chosen, it is advantageous to compute the required swale dimensions and then select a location where the calculated width and length will fit. If locations available cannot accommodate a linear swale, a wide-radius curved path can be used to gain length. Sharp bends should be avoided to reduce erosion potential. Regardless of when and how site selection is

performed, consideration should be given to the following site criteria:

a. **Soil type.** Soil characteristics in the swale bottom should be conducive to grass growth. Soils that contain large amounts of clay cause relatively low permeability and result in standing water, which may cause grass to die. Compacted soils will need to be tilled before seeding or planting. If topsoil is required to facilitate grass seeding and growth, use 6 inches of the following recommended topsoil mix: 50-80% sandy loam, 10-20% clay, and 10-20% composted organic matter (leaf compost).

b. **Slope.** The natural slope of the potential location will determine the nature and amount of re-grading, or if additional measures to reduce erosion and/or increase pollutant removal are required. Biofilters should be graded carefully to attain uniform longitudinal and lateral slopes, and to eliminate high and low spots. If needed, grade control checks should be provided to maintain the computed longitudinal slope and limit maximum flow velocity.

c. **Natural vegetation.** The presence and composition of existing vegetation can provide valuable information on soil and hydrology. If wetland vegetation is present, inundated conditions may exist at the site. The presence of larger plants, trees and shrubs may provide additional stabilization along the swale slopes, but also may shade any grass cover established. Most grasses grow best in full sunlight, and prolonged shading should be avoided. It is preferable that vegetation species be native to the region of application, where establishment and survival have been demonstrated.

9. **Step 9: Select vegetative cover.** A dense planting of grass provides the filtering mechanism responsible for water quality treatment in swales. In addition, grass has the ability to grow through thin deposits of sediment and sand, stabilizing the deposited sediment and preventing it from being re-suspended in runoff waters. Few other herbaceous plant species provide the same density and surface per unit area. Grass is by far the most effective choice of plant material in swales, however not all grass species provide optimum vegetative cover for use in swale systems. Dense turf grasses are best for vegetative cover. Table C9-S2- 6 is provided as an example of the variations in grass species. See the SUDAS specifications for information on the recommended or optimum turf grass species most suitable to the area, based on suitability in terms of cold tolerance, heat tolerance, mowing height adaptation, drought tolerance, and maintenance cost and effort.

   The type of grass cover can be selected at any earlier stage in the design process. Often if grass cover is known, optimum height can be established and flow depths can be set accordingly. In areas of poor drainage, wetlands species can be planted for increased vegetative cover. Use wetland species that are finely divided and relatively resilient, like grass. Use of invasive species should be avoided to eliminate proliferation in the swale and downstream.

10. **Step 10: Check swale stability.** The stability check is performed for the combination of highest expected flow and least vegetation coverage and height.

a. Compute 2-year and 10-year frequency storm event peak discharges using NRCS WinTR-55.

b. Check 2-year velocity for erosive potential (adjust geometry if necessary, and reevaluate WQv design parameters).

c. Check 10-year depth and velocity for capacity (adjust geometry if necessary, and reevaluate WQv and 2-year design parameters).

d. Provide minimum freeboard above 10-year stormwater surface elevation (6 inches minimum, recommended).

e. Stability is normally checked for flow rate (Q) for the 100-yr, 24-hour storm unless runoff from larger such events will bypass the swale. Q can be determined using the same methods mentioned for the initial design storm computation.

f. The maximum velocity, V_{max} (fps), that is permissible for the vegetation type, slope, and soil conditions should be obtained. Table C9-S2- 4 provides maximum velocity data for a variety of vegetative covers and slopes.

g. The estimated degree of retardance for different grass coverage (good or fair) should be obtained for the selected vegetation height. Estimation should be based on coverage and height that will first receive flow, or whenever coverage and height are at their lowest. Table C9-S2- 5 provides qualitative degrees of retardance for coverage and grass height.

Table C9-S2- 5: Guide for selecting degree of flow retardance

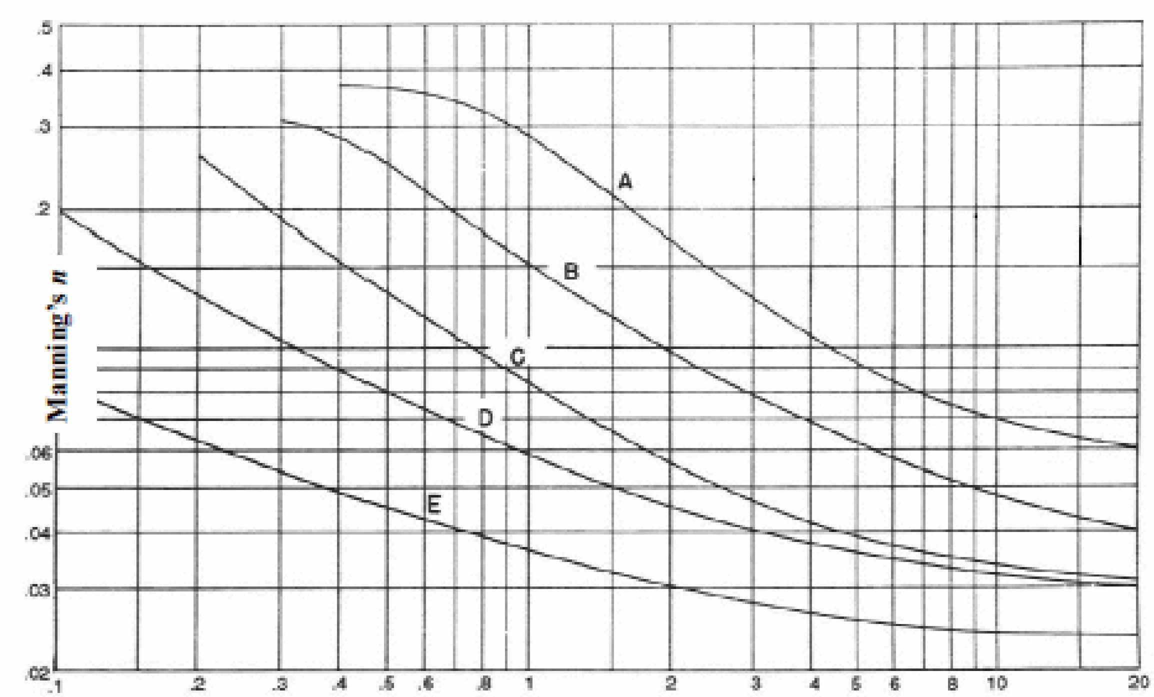
Average height of grass (inches)	Degree of vegetation coverage	
	Good	Fair
>25	A (very high)	B (high)
11-24	B (high)	C (moderate)
6-10	C (moderate)	D (low)
2-6	D (low)	D (low)
<2	E (very low)	E (very low)

Source: NRCS, 1954

Table C9-S2- 6: Manning roughness coefficients, n, for typical grasses for well-maintained straight channels without shrubbery or trees

Grass Type (1)	Depth of Flow	
	0.7-1.5 feet (2)	> 3.0 feet (3)
Bermuda grass, buffalo grass, Kentucky bluegrass		
a. Mowed to 2 inches	0.035	0.030
b. Length 4-6 inches	0.040	0.030
Good stand; any grass		
a. Length of 12 inches	0.070	0.035
b. Length of 24 inches	0.100	0.035
Fair stand; any grass		
a. Length of 12 inches	0.060	0.035
b. Length of 24 inches	0.070	0.035

Source: Chow, 1959



*VR: product of velocity and hydraulic radius
**Degree of flow retardance due to vegetation: A-very high, B-high, C-moderate, D-low, E-very low

Figure C9-S2- 7: Relation between Manning's n roughness coefficient VR* and degree of retardance**
Source: Haan et al., 1994

h. Select a trial Manning's n value for poor vegetation cover and low height. A good initial choice is n = 0.04. Using the alpha code assigned for the degree of retardance and the chosen n value, consult the graph in Figure C9-S2- 7 to obtain a first approximation for VR (velocity times hydraulic radius, ft^2/sec).
i. Compute the hydraulic radius, using the Vmax determined for vegetation type and slope, by applying the following equation:

Equation C9-S2- 10

$$R = \frac{VR}{V_{max}}$$

For precision, the VR value obtained from the graph, in units of ft^2/s, should be converted to metric units by multiplying by a factor of 0.09290 to obtain VR in m^2/s. From Manning's equation (metric):

$V = \left(\frac{1.0}{n}\right) R^{2/3} S^{1/2}$, then

Equation C9-S2- 11

$$VR = \frac{R^{1.67} S^{0.5}}{n}$$

Once the actual VR is determined, compare this value with the first approximation for VR obtained through Figure C9-S2- 7. If they do not agree within 5%, adjust Manning's n value and repeat the process until acceptable agreement is reached. If n <0.033 is needed to get agreement, set n = 0.033, solve VR again using Manning's equation above, and proceed. The actual velocity for the final design conditions should be computed using the following equation:

Equation C9-S2- 12

$$V = \frac{VR}{R}$$

The actual velocity V should be less than or equal to the maximum value obtained from Table C9-S2- 4. The area required for stability is computed using the continuity equation (Equation C9-S2- 1).

The area value obtained in this procedure should be compared with the area value obtained in the design flow analysis. If less area is required for stability than is provided for design flow, the design is acceptable. If more area is required for stability, use the area value obtained in the stability analysis to recalculate channel dimensions and recalculate the depth of flow, solving Equation 4 for y.

This stability flow depth, if needed, should be compared to the depth used in the design flow. The larger of the two values should be used, plus 12 inches (6 inches minimum) of freeboard, to obtain the channel depth (Equation C9-S2- 8).

A final check for capacity should be performed based on the stability check and the maximum vegetation height and cover to ensure that capacity is adequate if the largest expected event coincides with the greatest retardance. Use Manning's equation with Manning's n value used for design flow and the calculated channel dimension (including freeboard) to compute the flow capacity of the channel. If the flow capacity is less than the flow rate of the stability check, increase the channel cross sectional area as needed for this conveyance, and specify the new channel dimensions. Horner (1988) advocated using a parabolic shape for design even if a design for a trapezoidal shape is initially used in construction. A check using the parabolic shape may give an indication of performance at some later date. If there is insufficient space for the grass swale as designed, possibilities include dividing the flow among several swales, installing detention to control release rate upstream, increasing longitudinal slope, increasing side slopes, increasing vegetation height and design depth of flow (design should ensure vegetation remains standing during design flow), and reducing developed surface area to reduce runoff coefficient value and gain space for use of the grass swale.

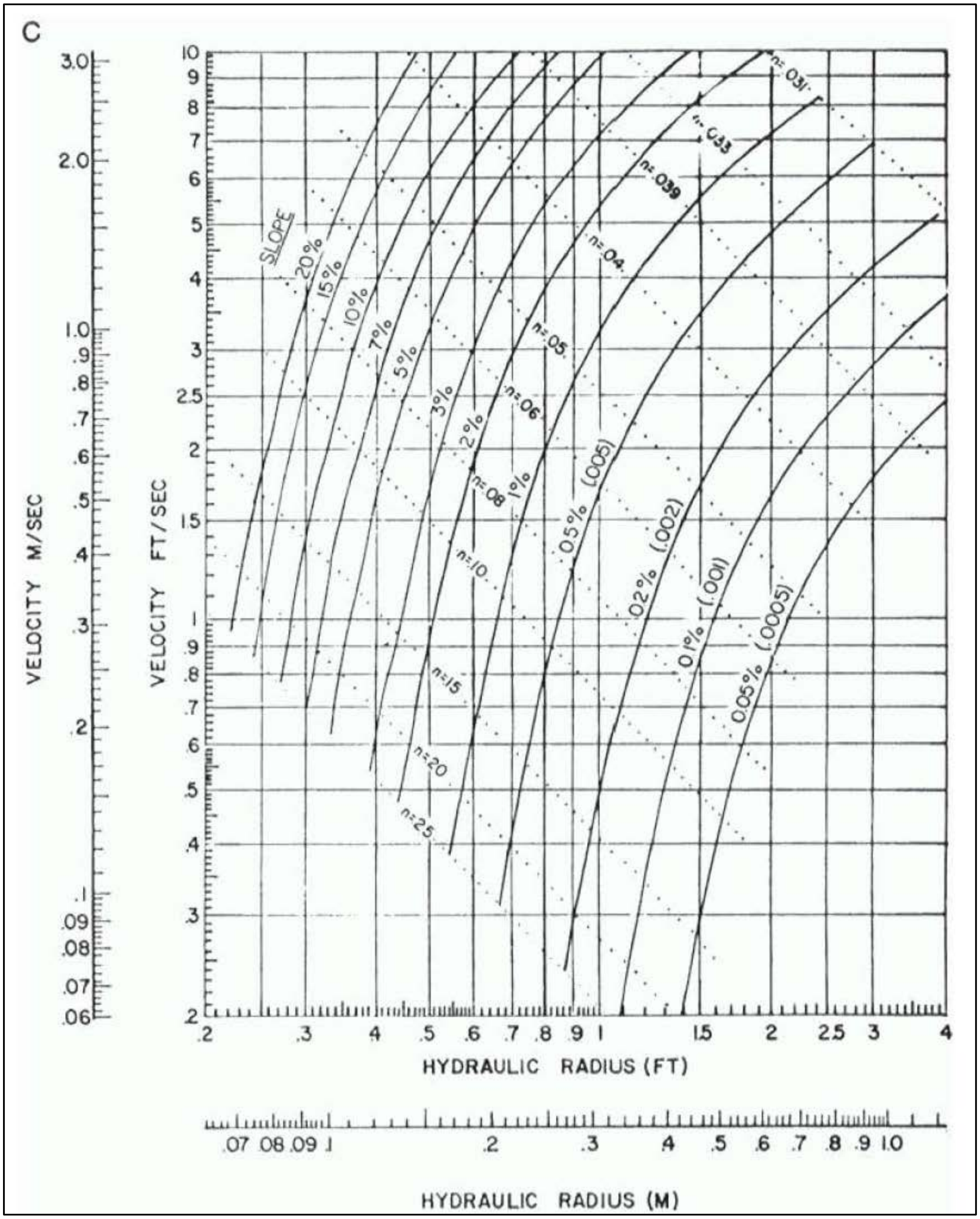


Figure C9-S2- 8: Solution to Manning's equation for retardance class C
Source: Haan et al., 1994

I. Inspection and maintenance requirements

Table C9-S2- 7: Typical maintenance activities for grass swales

Activity	Schedule
Mow grass to maintain a height of 3-6 inches.	As needed (frequently/seasonally)
Remove sediment buildup in the bottom of the grass swale once it has accumulated to 25% of the original design volume.	As needed (infrequently)
Inspect grass along side-slopes for erosion and formation of rills or gullies and correct.	Annually (semi-annually the first year)
Remove trash and debris accumulated in the channel.	
Based on inspection, plant an alternative grass species if the original grass cover has not been successfully established.	

Source: Claytor and Schuler, 1996

Table C9-S2- 8: Example criteria for turf grass cover

	Cold Tolerance	Heat Tolerance	Mowing Height	Drought Tolerance	Maintenance
High	Kentucky bluegrass Red fescue		Tall fescue		
			Red fescue		
		Tall fescue	Kentucky bluegrass Perennial ryegrass		
					Kentucky bluegrass
		Kentucky bluegrass Perennial ryegrass		Tall fescue	Perennial ryegrass
	Tall fescue			Red fescue	
		Red fescue		Kentucky bluegrass Perennial ryegrass	Tall fescue
Low					

Source: Adapted from Young et al., 1996

J. Design example

Trapezoidal Grass Swale.

1. **Basic data.** Small commercial lot 300 feet deep x 145 feet wide located in Des Moines, IA.
   a. Drainage area (A) = 1 acre
   b. Impervious percentage (I) = 70%
   c. $Rv = 0.05 + 0.009I = 0.68$

2. **Water quality peak flow.** (See Chapter 3, section 7 for details).
   a. Compute the water quality volume in inches:

$$WQv = 1.2(0.05 + 0.009 \times 70) = 0.82 inches$$

b. Compute modified CN for 1.25-inch rainfall (P=1.25 inches)

$$CN = \frac{1000}{[10 + 5P + 10Q - 10(Q^2 + 1.25QP)^{1/2}]}$$

$$CN = \frac{1000}{[10 + 5 \times 1.25 + 10 \times 0.82 - 10(0.822 + 1.25 \times 0.82 \times 1.25)^{1/2}]}$$

$$CN = 96.49\ (Use\ CN = 96)$$

c. For CN = 96 and an estimated time of concentration (tc) of 8 minutes (0.13 hours), compute the Q_{wq} for a 1.25-inch storm.

Compute Q_{wq} using NRCS WinTR-55:

Q_{wq} = 1.24 cfs

d. Compute Q_2 and Q_{10} using CN = 87 for this site (70% impervious urban commercial site with B soils) and t_c = 0.13 hr: WinTR-55 results:

Q_2 = 2.42 cfs Q_{10} = 4.14 cfs Q_{100} = 7.17 cfs

3. **Use Q_{wq} to size the channel.** The maximum flow depth for water quality treatment should be approximately the same height of the grass. A maximum flow depth of 4 inches is allowed for water quality design. A maximum flow velocity of 1 fps for water quality treatment is required. For Manning's n, use 0.15 for medium grass, 0.25 for dense grass. Longitudinal slope is 2%. Grass will need to be maintained at a 6-inch height. Trapezoidal channel with side slope of 4:1 (z = 4.0). Tall fescue will be used as the grass type.

a. Input variables: n = 0.15
S = 0.02 ft/ft
D = 4/12 = 0.33 ft

b. Then:

$$Q_{wq} = Q = VA = \left[\left(\frac{1.49}{n}\right) D^{2/3} S^{1/2}\right] \times DW$$

Where:
Q = peak flow (cfs)
V = velocity (ft/s)
A = flow area (ft^2) = WD
W = channel bottom width (ft)
D = flow depth (ft)
S = slope (ft/ft)

Note: D approximates hydraulic radius for shallow flows.

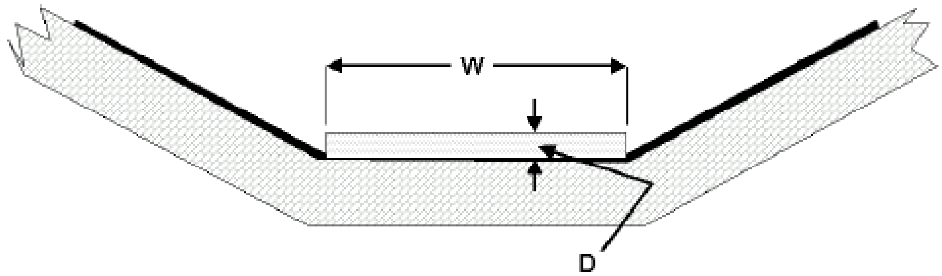


Figure C9-S2- 9: Using Q_{wq} to size a channel

c. Then for a known n, Q, D, and S minimum width can be calculated.

$$\frac{nQ}{1.49D^{5/3}S^{1/2}} = W = \frac{0.15 \times 1.24}{1.49 \times 0.33^{5/3} \times 0.02^{1/2}} = 5.6ft\,\min(use\ 6\ ft)$$

$$V = \frac{Q}{WD} = \frac{1.24\,ft^3/sec}{6ft \times \frac{4}{12}ft} = 0.62fps\ (OK\!:\ < 1fps)$$

Note: WD approximates flow area for shallow flows.

Minimum length for 5-minute residence time, $L = V(5 \times 60) = 186ft(\sim 372ft\ for\ t = 10\text{min})$

d. Depending on the site geometry; the width, slope, or density of grass (Manning's n value) might be adjusted to slow the velocity and shorten the channel in the next design iteration. For example, using an 8-foot bottom width* of flow and a Manning's n of 0.25, solve for new depth and length.

$$Q = VA = \left(\frac{1.49}{n}\right) D^{5/3}S^{1/2}W$$

$$D = \left[\frac{Qn}{1.49S^{1/2}W}\right]^{3/5}$$

$$D = \left[\frac{1.24 \times 0.25}{1.49 \times 0.02^{1/2} \times 8.0}\right]^{3/5} = 0.36ft\ (OK\!: < 4in$$

$$V = \frac{Q}{WD} = \frac{1.24}{8.0 \times 0.36} = 0.43fps$$

$$L = 0.43fps \times 5min \times 60\ sec/min = 129ft$$

For a velocity of 0.62 fps, a channel bottom width of 6 feet, flow depth of 4 inches, and Q = 1.24 cfs

$$A = \frac{1.24\,ft^3/sec}{0.62fps} = 2ft^2$$

4. **Check for stability and capacity at the computed dimensions.**
   a. Q_{10} = 4.14 cfs and Q_{100} = 7.17 cfs. From design flow, width of channel bottom is 6 feet. Base the check on a grass height of 6 inches and with fair coverage. From Table C9-S2- 5, the degree of retardance is category D. The soils are HSG B soils and will be erosion-resistant. The maximum velocity (V_{max}) is 6 fps (1.80 m/sec) from Table C9-S2- 4. Select a trial Manning's n value of 0.04, which corresponds to a VR value (velocity x hydraulic radius) of 3 ft^2/sec using Figure C9-S2- 7. Convert the VR value to metric units:

$$VR_{metric} = VR_{english} \times 0.0929 = 3\,ft^2/sec \times 0.0929 = 0.28\,m^2/sec$$

b. Calculate the hydraulic radius, R, using Equation C9-S2- 11:

$$R = \frac{0.28\, m^2/sec}{1.80\, m/sec} = 0.15m(0.47ft)$$

c. Using the computed hydraulic radius, calculate the actual VR using Equation C9-S2- 11:

$$VR = 0.15m^{1.67} \times \frac{0.02^{0.5}}{0.04} = 0.16\, m^2/sec\, (1.68\, ft^2/sec)$$

The estimated VR value, 3 ft^2/sec, is not within 5% of the computed VR value, 1.68 ft^2/sec. using a new Manning's n value of 0.036, from Figure C9-S2- 5, the new estimated VR is 6 ft^2/sec (0.56 m^2/sec). The recalculated R from Equation C9-S2- 9 is 0.31 m (0.98 feet) and the recalculated VR from Equation C9-S2- 11 is 0.55 m^2/sec (5.92 ft^2/sec). The new value is within 5% of the estimated value of 0.56 m^2/sec, so proceed with stability check.

d. The actual velocity for the new design is recomputed using Equation C9-S2- 12:

$$V = \frac{0.56\, m^2/sec}{0.31m} = 1.80\, m/sec\, (5.91fps)$$

The actual velocity is less than the estimated maximum velocity of 6 fps from Table C9-S2- 5, and the stability check can proceed.

e. Calculate the X-section area to test stability using the continuity equation:

$$A = \frac{Q_{100}}{V} = \frac{7.17\, ft^3/sec}{5.91fps} = 1.21ft^2$$

The stability area of 1.21 ft^2 is less than the original calculated flow area of 2 ft^2, so can proceed to the capacity check. If the stability area was larger, then would need to select a new trial size and flow depth and recalculate the X-section area of flow until this condition is met.

f. The channel dimensions, including freeboard, are used to compute the flow capacity of the channel. The greater of the two flow depths from the design flow or stability check should be used. In this example, the stability check flow depth of 0.98 feet is greater than the design flow depth of 0.33 feet (4 inches). Using Equation C9-S2- 8:

$$H = y + freeboard = 0.98ft + 1ft = 1.98ft$$

g. Using Manning's equation, the Manning's n value selected in the design flow (0.15) and the channel dimensions, recompute the flow capacity for the channel. Using Equation C9-S2- 4, and using H for Y:

$$A = by + zy^2 = (6.0ft)(1.98ft) + (4)(1.98ft)^2 = 27.56ft^2$$

Substituting Equation C9-S2- 6 into Equation C9-S2- 7 (using H for y):

$$R = \frac{A}{P} = \frac{27.56ft^2}{[6ft + (2)(1.98ft)(1 + 4^2)^{0.5}]} = 1.54ft$$

Using Equation C9-S2- 1 and Equation C9-S2- 2 (n = 0.15, S = 0.02):

$$Q = 27.56ft^2 \times (1.49)(1.54ft)^{0.667} \times \frac{0.02^{0.5}}{0.15} = 51.64\, ft^3/sec$$

The flow capacity of 51.6 ft3/sec for the swale is greater than the stability check flow rate of 7.17 ft^3/sec for the 100-yr storm, provided in the example site data.

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

Description: Dry swales (also called bio-swales) are a type of open vegetated channel used to treat and attenuate the WQv of stormwater runoff. The swale also serves as a conveyance to move excess stormwater to a downstream discharge point. In dry swales, the entire WQv 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. The wet swale is a vegetated channel designed to retain water or marshy conditions that support wetland vegetation. A high water table or poorly-drained soil is necessary to retain water.

Typical uses:
- Manage runoff from residential sites, parking areas, and along perimeter of paved roadways.
- Located in a drainage easement at the rear of 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:
- 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 maintaining sufficient vegetation density.
- Limited to small areas (<5 acres); cannot be used on steep slopes (>6%).
- Higher maintenance than curb and gutter systems.
- Possible re-suspension of sediment.

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

A. Description

Dry and wet swales (also referred to as vegetated open channels, water quality swales, or enhanced swales) 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. Dry and wet swales 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. Brief descriptions of these two designs are listed below (Claytor and Schuler, 1996):

1. **Dry swale.** The dry swale 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. The dry swale uses volume-based sizing criteria. The dry swale 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 is likely to be more limited since only a grass cover crop is available for nutrient uptake. Dry swales 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 C9-S3- 3 illustrates the configuration and design components of the dry swale.

Figure C9-S3- 1: Dry swale
Source: Georgia Stormwater Manual, 2001

2. **Wet swale (wetland channel).** The wet swale also consists of a broad open channel capable of temporarily storing the WQv (also a volume-based sizing criteria), but does not have an underlying filtering bed. The wet swale is constructed directly within existing soils and may or may not intercept the water table. Like the dry swale, the WQv within the wet swale should be stored for approximately 24 hours. The wet swale has water quality treatment mechanisms similar to stormwater wetlands, which rely primarily on settling of suspended solids, adsorption, and uptake of pollutants by vegetative root systems. Figure C9-S3- 4 illustrates the configuration and design components of the wet swale.

Figure C9-S3- 2: Wet swale
Source: Georgia Stormwater Manual, 2001

Dry and wet swales are not to be confused with a filter strip or grass swale, which are limited- application structural controls, and not considered acceptable for meeting the TSS removal performance goal when used alone. Ordinary grass swales are not engineered to provide the same treatment capability as a well-designed dry swale with filter media. Filter strips are designed to accommodate overland flow rather than channelized flow, and can be used as stormwater credits to help reduce the total water quality treatment volume for a site. Both of these practices may be used for pre-treatment or included in a treatment train approach where redundant treatment is provided. For a further discussion of these limited application structural controls, see Chapter 9, section 2 and section 4.

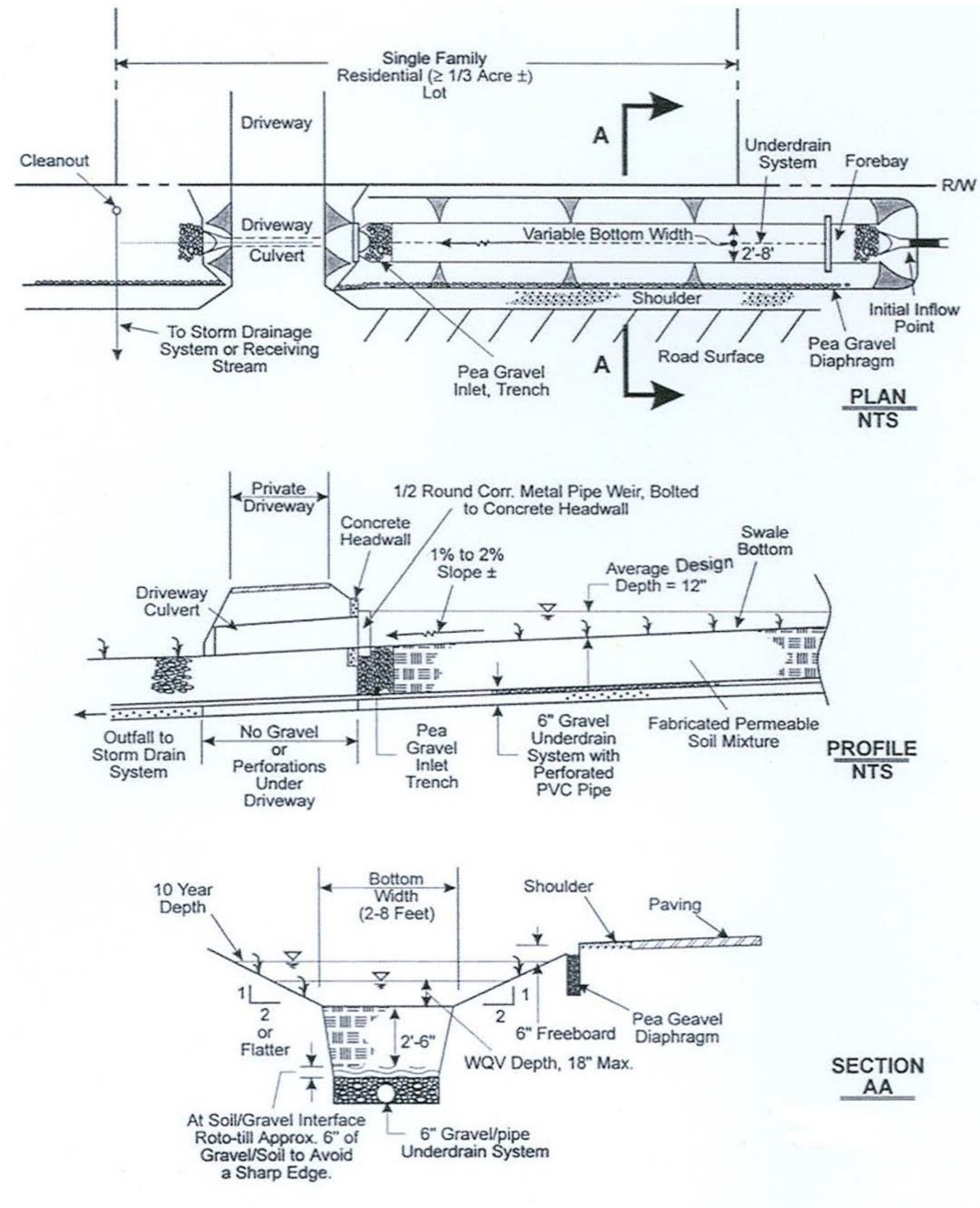


Figure C9-S3- 3: Configuration and design components of a dry swale BMP
Source: Claytor and Schuler, 1996

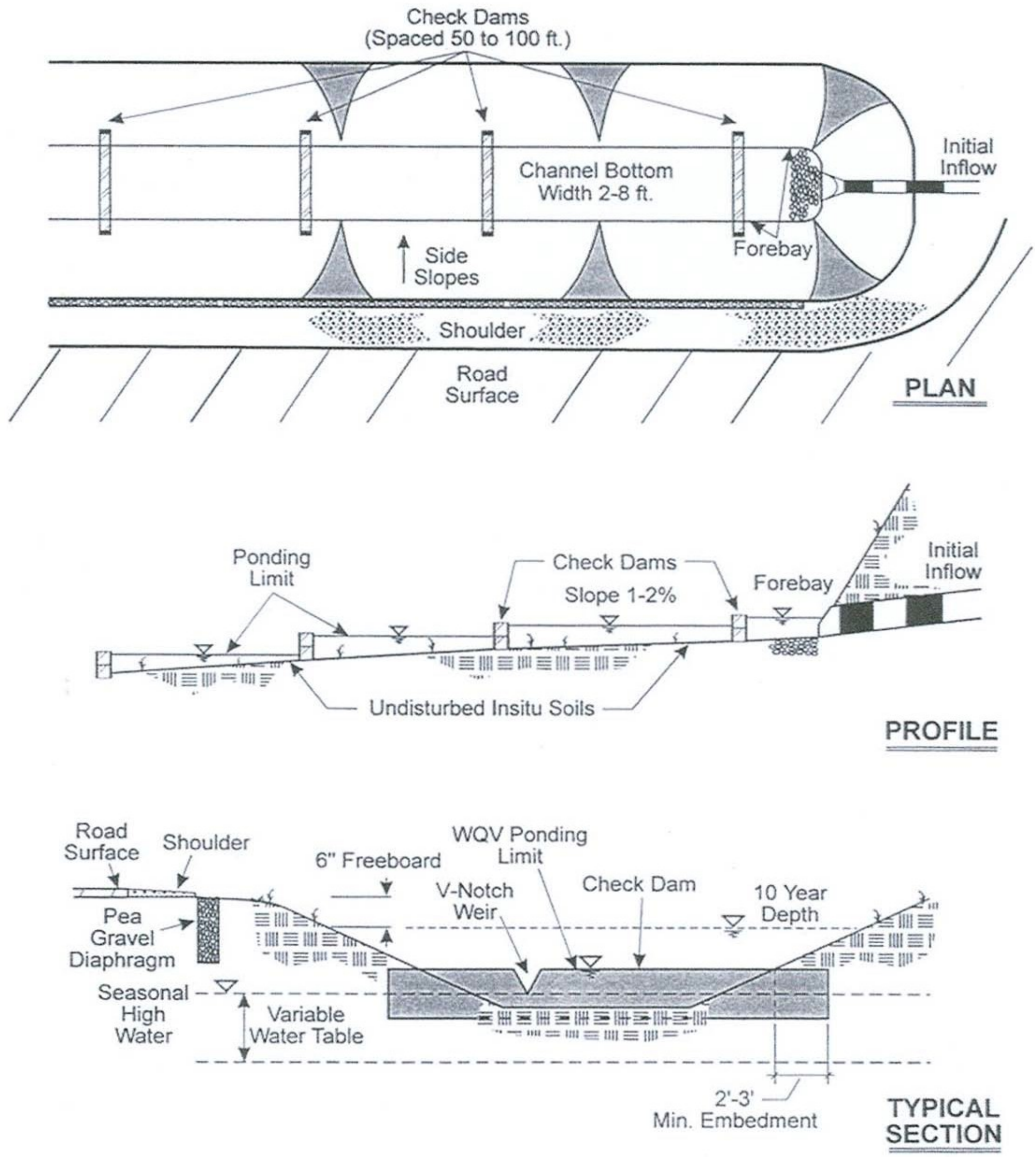


Figure C9-S3- 4: Configuration and design components of a wet swale BMP
Source: Claytor and Schuler, 1996

B. Stormwater management suitability

Dry and wet swale systems are designed primarily for stormwater quality and have only a limited ability to provide channel protection or to convey higher flows to other controls.

1. **Water quality.** Dry swale systems rely primarily on filtration through an engineered media to provide removal of stormwater contaminants. Wet swales achieve pollutant removal both from sediment accumulation and biological removal. Chapter 9, section 1 provides median pollutant removal efficiencies that can be used for planning and design purposes.

2. **Channel protection.** Generally only the WQv is treated by a dry or wet swale, and another structural control must be used to provide Cpv extended detention. However, for some smaller sites, a swale may be designed to capture and detain the full Cpv.

3. **Overbank flood protection.** Enhanced swales must provide flow diversion and/or be designed to safely pass overbank flood flows. Another structural control must be used in conjunction with an enhanced swale system to

reduce the post-development peak flow of the 25-year storm (Q_{p25}) to pre-development levels (detention).

4. **Extreme flood protection.** Enhanced swales must provide flow diversion and/or be designed to safely pass extreme storm flows. Another structural control must be used in conjunction with an enhanced swale system to reduce the post-development peak flow of the 100-year storm (Q_f) if necessary.

C. Pollutant removal capabilities

Both the dry and wet enhanced swale are presumed to be able to remove 80% of the total suspended solids load in typical urban post-development runoff when sized, designed, constructed, and maintained in accordance with the recommended specifications. Undersized or poorly-designed swales can reduce TSS removal performance. The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment. In a situation where a removal rate is not deemed sufficient, additional controls may be put in place at the given site in a series or treatment train approach.

- Total suspended solids – 80%
- Total phosphorus – dry swale 50%, wet swale 25%
- Total nitrogen – dry swale 50%, wet swale 40%
- Fecal coliform – insufficient data
- Heavy metals – dry swale 40%, wet swale 20%

For additional information and data on pollutant removal capabilities for enhanced dry and wet swales, see the National Pollutant Removal Performance Database (2nd Edition) available at http://www.cwp.org/ and the International Stormwater Best Management Practices (BMP) Database at http://www.bmpdatabase.org/.

D. Application and feasibility

Dry and wet swales can be used in a variety of development types; however, they are primarily applicable to residential and institutional areas of low to moderate density where the impervious cover in the contributing drainage area is relatively small, and along roads and highways. Dry swales are mainly used in moderate- to large-lot residential developments, small impervious areas (parking lots and rooftops), and along rural highways. Wet swales tend to be used for highway runoff applications, small parking areas, and in commercial developments as part of a landscaped area.

Because of their relatively large land requirements, enhanced swales are generally not used in higher- density areas. In addition, wet swales may not be desirable for some residential applications, due to the presence of standing and stagnant water, which may create nuisance odor or mosquito problems.

The topography and soils of a site will determine the applicability of the use of one of the two swale designs. Overall, the topography should allow for the design of a swale with sufficient slope and cross sectional area to maintain non-erosive velocities. The following criteria should be evaluated to ensure the suitability of a stormwater pond for meeting stormwater management objectives on a site or development.

1. **General feasibility.**
   a. Suitable for residential subdivision usage – yes
   b. Suitable for high-density/ultra-urban areas – no
   c. Regional stormwater control – no

2. **Physical feasibility – physical constraints at project site.**
   a. **Drainage area.** 5 acres maximum
   b. **Space required.** Approximately 10-20% of the tributary impervious area
   c. **Site slope.** Typically no more than 4% channel slope
   d. **Minimum head.** Elevation difference needed at a site from the inflow to the outflow: 3-5 feet for dry swale; 1 foot for wet swale
   e. **Minimum depth to water table.** 2 feet required between the bottom of a dry swale and the elevation of the seasonally high water table if an aquifer or treating a hotspot; wet swale is below water table or placed in poorly drained soils.

f. **Soils.** Engineered media for dry swale
g. The WQv for high-density residential, commercial, and industrial land uses will most likely be too great to be accommodated with most swale designs. However, swales may be appropriate for pre-treatment in association with other practices for these higher-density land uses, or may be acceptable solutions for watershed retrofit projects.

3. **Other constraints/considerations.** Aquifer protection: exfiltration should not be allowed for hotspots.

E. Planning and design criteria

The following criteria are considered minimum criteria for the design of dry and wet swale system.

1. **Location and siting.**
   a. A dry or wet swale 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 non-erosive velocities.
   b. Dry and wet swale systems should have a contributing drainage area of 5 acres or less.
   c. Swale siting should also take into account the location and use of other site features, such as buffers and undisturbed natural areas, and should attempt to aesthetically fit the facility into the landscape.
   d. A wet swale can be used where the water table is at or near the soil surface, or where there is a sufficient water balance in poorly drained soils to support a wetland plant community.

2. **General design.**
   a. Both dry and wet swales are designed to treat the WQv through a volume-based design and to safely pass larger storm flows. Flow enters the channel through a pre-treatment forebay. Runoff can also enter along the sides of the channel as sheet flow through the use of a pea gravel flow spreader trench along the top of the bank.
   b. **Dry swale.** A dry swale 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. Runoff is collected and conveyed by a perforated pipe and gravel underdrain system to the outlet. Figure C9-S3- 3 provides a plan view and profile schematic for the design of a dry swale system.
   c. **Wet swale.** A wet swale or wetland channel consists of an open conveyance channel which has been excavated to the water table or to poorly drained soils. Check dams are used to create multiple wetland cells, which act as miniature shallow marshes. Figure C9-S3- 4 provides a plan view and profile schematic for the design of a wet swale system.

3. **Physical specifications and geometry.**
   a. Channel slopes between 1-2% are recommended unless topography necessitates a steeper slope, in which case 6- to 12-inch drop structures can be placed to limit the energy slope to within the recommended 1-2% range. Energy dissipation will be required below the drops. Spacing between the drops should not be closer than 50 feet. Depth of the WQv at the downstream end should not exceed 18 inches.
   b. Dry and wet swales should have a bottom width of 2-8 feet to ensure adequate filtration. Wider channels can be designed but should contain berms, walls, or a multi-level cross section to prevent channel braiding or uncontrolled sub-channel formation.
   c. Dry and wet swales are parabolic or trapezoidal in cross section, and are typically designed with moderate side slopes no greater than 2:1 for ease of maintenance and side inflow by sheet flow (4:1 or flatter recommended).
   d. Dry and wet swales should maintain a maximum WQv ponding depth of 18 inches at the end point of the channel. A 12-inch average depth should be maintained.
   e. The peak velocity for the 2-year storm must be nonerosive for the soil and vegetative cover provided.
   f. If the system is online, channels should be sized to convey runoff from the overbank flood event (Q_{p25}) safely with a minimum of 6 inches of freeboard and without damage to adjacent property.

4. **Dry swale.**
   a. Dry swale channels are sized to store and infiltrate the entire WQv with less than 18 inches of ponding, and

allow for full filtering through the permeable soil layer. The maximum ponding time is 48 hours, though a 24-hour ponding time is more desirable.

b. The bed of the dry swale consists of a permeable soil layer of at least 30 inches in depth, above a 4-inch diameter perforated PVC pipe (AASHTO M 252) longitudinal underdrain in a 6-inch gravel layer. The soil media should have an infiltration rate of at least 1 foot per day (1.5 feet per day maximum) and contain a high level of organic material to facilitate pollutant removal. A permeable filter fabric is placed between the gravel layer and the overlying soil.

c. The channel and underdrain excavation should be limited to the width and depth specified in the design. The bottom of the excavated trench shall not be loaded in a way that causes soil compaction, and scarified prior to placement of gravel and permeable soil. The sides of the channel shall be trimmed of all large roots. The sidewalls shall be uniform with no voids and scarified prior to backfilling.

5. **Wet swale.**
   a. Wet swale channels are sized to retain the entire WQv with less than 18 inches of ponding at the maximum depth point.
   b. For wet swales, the WQv volume is still retained for 24 hours, but ponding may continue indefinitely depending on the depth and elevation to the water table.
   c. Check dams can be used to achieve multiple wetland cells. V-notch weirs in the check dams can be utilized to direct low-flow volumes.

6. **Pre-treatment inlets.**
   a. Inlets to dry and wet swales must be provided with energy dissipators such as riprap or rock-lines stilling basins.
   b. Pre-treatment of runoff in both a dry and wet swale system is typically provided by a sediment forebay located at the inlet. The pre-treatment volume should be equal to 0.1 inches per impervious acre. This storage is typically obtained by providing check dams at pipe inlets and/or driveway crossings.
   c. Dry and wet swale systems that receive direct concentrated runoff have a 6-inch drop to a pea gravel diaphragm flow spreader at the upstream end of the control.
   d. A pea gravel diaphragm and gentle side slope is provided along the top of channels to provide pre-treatment for lateral sheet flows.

7. **Outlet structures.**
   a. **Dry swale.** The underdrain system should discharge to the storm drainage infrastructure or a stable outfall.
   b. **Wet swale.** Outlet protection must be used at any discharge point from a wet swale to prevent scour and downstream erosion.

8. **Emergency spillway/overflow.** Dry and wet swales must be adequately designed to safely pass flows that exceed the design storm flows.

9. **Maintenance access.** Adequate access is to be provided for all dry and wet swale systems for inspection and maintenance.

10. **Safety features.** Ponding depths should be limited to a maximum of 18 inches.

11. **Landscaping.** Landscape design should specify proper grass species and wetland plants based on specific site, soils, and hydric conditions present along the channel. Below is some specific guidance for dry and wet swales:
    a. **Dry swale.** Information on appropriate turf grass species for Iowa can be found in the SUDAS Specifications Manual Section 9010.
    b. **Wet swale.**
       1) Emergent vegetation should be planted or wetland soils may be spread on the swale bottom for seed stock.
       2) Information on establishing wetland vegetation and appropriate wetland species for Iowa can be found in the SUDAS Specifications Manual, Section 9010.
       3) Where wet swales do not intercept the groundwater table, a water balance calculation should be performed to ensure an adequate water budget to support the specified wetland species. See Chapter 3, section 10 for guidance on water balance calculations.

12. **Additional site-specific design criteria and issues.**
    a. Physiographic factors (local terrain design constraints).
        1) Low relief: reduced need for use of check dams.
        2) High relief: often infeasible if slopes are greater than 4%.
        3) Karst: no exfiltration of hotspot runoff from dry swales; use impermeable liner.

    b. **Soils.** No additional criteria.
    c. **Special downstream watershed considerations.** Aquifer protection: no exfiltration of hotspot runoff from dry swales; use impermeable liner.

Table C9-S3- 1: Summary design criteria for dry and wet swale systems

Parameter	Swale Design Criteria
Pre-treatment volume	0.05-0.10 inches per impervious acre at initial inflow point
Preferred shape	Trapezoidal or parabolic
Bottom width	2-foot minimum, 8-foot maximum widths; up to 16-foot width if a dividing berm is installed to reduce concentrated flow channels.
Side slopes	2:1 maximum; 4:1 is optimum
Longitudinal slope	1-2% without check dams. On greater slopes, check dams added to achieve ponding of the WQv.
Sizing criteria	Length, width, depth, and slope needed to provide surface storage for WQv. Outlet structures sized to release WQv over 24 hours.
Hydraulic residence time	Minimum: 5 minutes; Optimum: 10-minutes
Average flow velocity	0.9 fps
Length	Minimum: 100 feet; Optimum: 200 feet
Underlying soil bed	Equal to swale width *Dry swale*: Moderately-permeable soils (USC ML, SM, or SC); soil mix 30 inches deep with gravel/pipe underdrain. *Wet swale*: Undisturbed soils, no underdrain system
Depth and capacity	Surface storage of WQv with a maximum depth of 18-inches for water quality treatment (12-inch average depth); safely convey 2-yr storm peak discharge with non-erosive velocity (4-5 fps); adequate conveyance capacity for the 10-yr storm peak discharge with 6 inches of freeboard.

Source: Adapted from Claytor and Schuler, 1996

F. Design procedure

1. **Step 1.** Compute runoff control volumes from the unified stormwater sizing criteria. Calculate the WQv, Cpv, Q_p, and the Q_f. (See Chapter 2).

2. **Step 2**. Determine if the development site and conditions are appropriate for the use of a dry or wet swale system. Consider the application and site feasibility criteria, location, and siting guidelines.

3. **Step 3.** Confirm local design criteria and applicability.
    a. Consider any special site-specific design conditions/criteria from Chapter 4, section 1.
    b. Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply.

4. **Step 4.** Determine pre-treatment volume.
    a. The forebay should be sized to contain 0.1 inches per impervious acre of contributing drainage.
    b. The forebay storage volume counts toward the total WQv requirement, and should be subtracted from the

WQv for subsequent calculations.

5. **Step 5.** Determine swale dimensions.
   a. Size bottom width, depth, length, and slope necessary to store WQv with less than 18 inches of ponding at the downstream end.
   b. Slope cannot exceed 4% (1-2% recommended).
   c. Bottom width should range from 2-8 feet.
   d. Ensure that side slopes are no greater than 2:1 (4:1 recommended).

6. **Step 6.** Compute the number of check dams (or similar structures) required to detain WQv.

7. **Step 7.** Calculate draw-down time.
   a. **Dry swale.** Planting soil should pass a maximum rate of 1.5 feet in 24 hours, and must completely filter WQv within 48 hours.
   b. **Wet swale.** Must hold the WQv.

8. **Step 8.** Check 2-year and 25-year velocity erosion potential and freeboard.
   a. Use NRCS WinTR-55 to compute estimates of the peak flow rates for the 2-yr through 100-yr storm events.
   b. Check for erosive velocities and modify design as appropriate.
   c. Provide a minimum of 6 inches freeboard for the 10-year discharge.
   d. Check stability for larger storm events and overland flow pathways for the larger storms.

9. **Step 9.** Design low-flow orifice at downstream headwalls and check dams. Design orifice to pass WQv in 6 hours, using orifice equation.

10. **Step 10.** Design inlets, sediment forebay(s), and underdrain system (dry swale).

11. **Step 11**. Prepare vegetation and landscaping plan. A landscaping plan for a dry or wet swale should be prepared to indicate how the dry or wet swale system will be stabilized and established with vegetation.

G. Design example

Dry swale.

1. **Site description.** Bucketsville, IA Recreation Center (Story County, IA).
   a. Site area = total drainage area (A) = 3.4 acres
   b. Total impervious area (building, parking, and driveway) = 1.9 acres I = 56%
   c. Soils: HSG – C
   d. Pre-development: meadow in good condition

Table 2I-3- 1: Data for Story County example

Rainfall Data for Story County Example (24-hr duration)	
Return period	Rainfall, P (inches)
0.3-yr (WQ event)	1.25
1-yr	2.38
2-yr	2.91
5-yr	3.64
10-yr	4.27
25-yr	5.15
100-yr	6.61

Hydrologic Data		
	Pre	Post
CN	71	87
t_c	0.34	0.22

WinTR-55 Storm Summary – Pre-development				
Storm	**P (inches)**	**Runoff Q_a (inches)**	**Q_p, Peak Discharge, cfs**	**Total Runoff Volume (ft^3)**
1	2.38	0.38	1.27	4,690
2	2.91	0.71	2.34	8,763
5	3.64	1.15	4.05	14,193
10	4.27	1.58	5.70	19,500
25	5.15	2.22	8.18	27,399
100	6.61	3.39	12.57	41,839
WinTR-55 Storm Summary – Post-development				
Storm	**P (inches)**	**Runoff Q_a (inches)**	**Q_p, Peak Discharge, cfs**	**Total Runoff Volume (ft^3)**
WQ storm	1.25	0.79	3.7	*9,750*
1	2.38	1.20	5.32	14,810
2	2.91	1.65	7.27	20,364
5	3.64	2.29	10.04	28,263
10	4.27	2.87	12.46	35,422
25	5.15	3.70	15.85	45,665
100	6.61	5.09	21.48	62,870

This example focuses on the design of a dry swale (with media) to meet the water quality treatment requirements of the site. Channel protection and overbank flood control is not addressed in this example other than quantification of preliminary storage volume and peak discharge requirements. In general, the primary function of dry swales is to provide water quality treatment and groundwater recharge, and not large storm attenuation. Flows in excess of the WQv are typically routed to bypass the facility. Where quantity control is required, the bypassed flows can be routed to conventional detention basins (or some other facility such as underground storage vaults).

Table 2I-3- 2: WinTR-55 current data description

User:	SEJ	Date:	12/10/2006
Project:	Bucketsville Rec Center	Units:	English
SubTitle:	Dry Swale **(pre-development)**	Area Units:	Acres
State:	Iowa	County:	Story
Filename:	C:\Documents and Settings\Stephen\Application Data\WinTR-55\Dry Swale_pre.w55		

--- Sub-Area Data ---

Name	Description	Reach	Area (ac)	RCN	T_c
DA-1		Outlet	3.4	71	.341
		Total area:	3.40(ac)		

-----Storm Data------

Rainfall Depth by Rainfall Return Period

0.3-yr (in)	1-Yr (in)	2-Yr (in)	5-Yr (in)	10-Yr (in)	25-Yr (in)	100-Yr (in)
1.25	2.38	2.91	3.64	4.27	5.15	6.61

Storm Data Source: User-provided custom storm data (MRA – Bulletin 71)

Rainfall Distribution Type: Type II

Dimensionless Unit Hydrograph: <standard>

-------Sub-Area Land Use and Curve Number Details------

Sub-Area Identifier	Land Use	Hydrologic Soil Group	Sub-Area Area (ac)	Curve Number
DA-1	Meadow -cont. grass (non-grazed)	C	3.4	71
	Total Area / Weighted Curve Number		3.4	71

-------Sub-Area Time of Concentration Details--------

Sub-Area Identifier	Flow Length (ft)	Slope (ft/ft)	Mannings's n	End Area (sq ft)	Wetted Perimeter (ft)	Velocity (ft/sec)	Travel Time (hr)
DA-1							
Sheet	100	0.050	0.240				0.280
Shallow	500	0.0200	0.050				0.061
					Time of Concentration:		**.341**

------Hydrograph Peak/Peak Time Table------

Sub-Area or Reach Identifier	Peak Flow 1-Yr (cfs)	Peak Flow 2-Yr (cfs)	Peak Flow 5-Yr (cfs)	Peak Flow 10-Yr (cfs)	Peak Flow 20-Yr (cfs)	Peak Flow 100-Yr (cfs)
	(hr)	(hr)	(hr)	(hr)	(hr)	(hr)
Sub-Areas						
DA-1	1.27	2.34	4.05	5.70	8.18	12.57
	12.13	12.13	12.12	12.11	12.10	12.10
Outlet	1.27	2.34	4.05	5.70	8.18	12.57

----- Identification Data -----

User:	SEJ	Date:	12/10/2006
Project:	Bucketsville Rec Center	Units:	English
SubTitle:	Dry Swale **(post-development)**	Area Units:	Acres

State: Iowa County: Story

Filename: C:\Documents and Settings\Stephen\Application Data\WinTR-55\Dry Swale_pre.w55

--- Sub-Area Data ---

Name	Description	Reach	Area (ac)	RCN	T_c
DA-1		Outlet	3.4	87	0.222
		Total area:	3.40(ac)		

-------Sub-Area Land Use and Curve Number Details------

Sub-Area Identifier	Land Use	Hydrologic Soil Group	Sub-Area Area (ac)	Curve Number
DA-1	Open space; grass cover >75% (good)	C	1.5	74
	Paved parking lots, roofs, driveways	C	1.9	98
	Total Area / Weighted Curve Number		**3.4**	**87**

-------Sub-Area Time of Concentration Details--------

Sub-Area Identifier	Flow Length (ft)	Slope (ft/ft)	Mannings's n	End Area (sq ft)	Wetted Perimeter (ft)	Velocity (ft/sec)	Travel Time (hr)
DA-1							
Sheet	50	0.0150	0.240				0.161
Shallow	600	0.0200	2.91				0.058
Channel	50	0.0200	0.024	4.5	10.3	4.630	0.003
					Time of Concentration:		**.222**

------Hydrograph Peak/Peak Time Table------

Sub-Area or Reach Identifier	Peak Flow 0.3-Yr (cfs)	Peak Flow 1-Yr (cfs)	Peak Flow 2-Yr (cfs)	Peak Flow 5-Yr (cfs)	Peak Flow 10-Yr (cfs)	Peak Flow 20-Yr (cfs)	Peak Flow 100-Yr (cfs)
	(hr)	(hr)	(hr)	(hr)	(hr)	(hr)	
Sub-Areas							
DA-1	1.55	5.32	7.27	10.04	12.46	15.85	21.48
	12.05	12.03	12.02	12.02	12.02	12.02	12.02
Reaches							
Outlet	**1.55**	**5.32**	**7.27**	**10.04**	**12.46**	**15.85**	**21.48**

WinTR-20 summary data for the 1-yr post-development scenario (From WinTR-55 analysis)

Area or Reach Identifier	Drainage Area (sq mi)	***STORM 1-yr*** Rain Gage ID or Location	Runoff Amount (in)	Elevation (ft)	-----Peak Flow----- Time (hr)	Rate (cfs)	Rate (csm)	
DA-1		0.005		1.200		12.03	5.32	1002.36

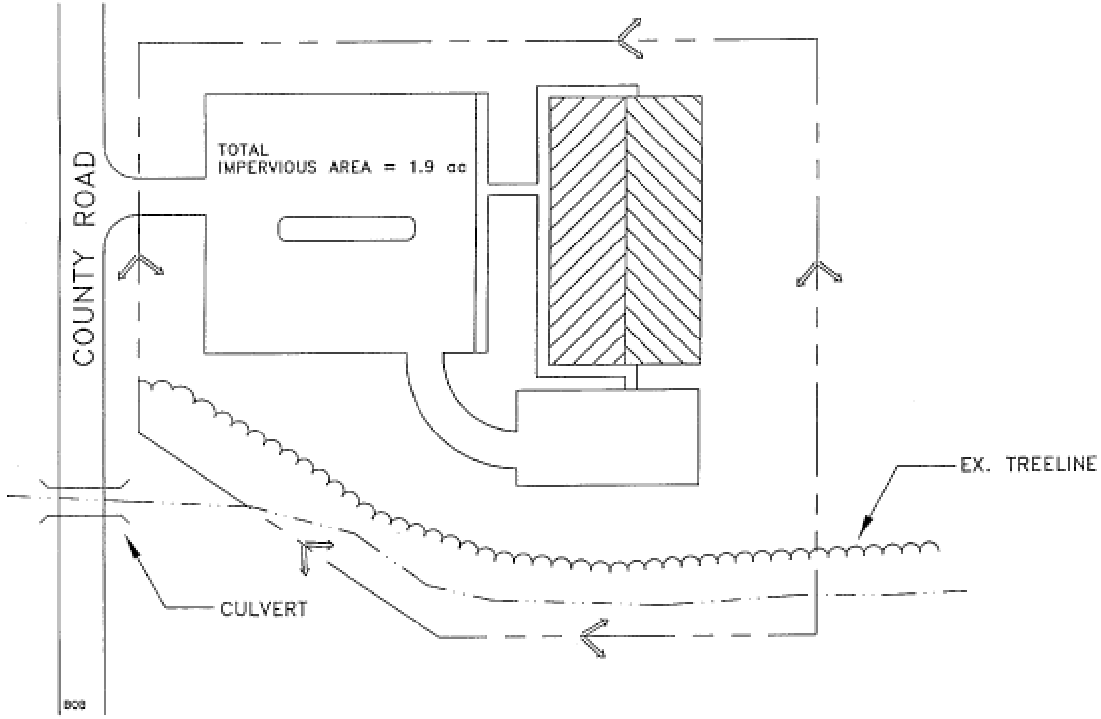


Figure C9-S3- 5: Site plan for dry swale design example

2. **Compute water quality volume (WQv) and WQv peak discharge (Q_{wq}).**
   a. $Rv = 0.05 + (0.009)(56) = 0.55$
   b. $P = 1.25 inches\ (WQv\ design\ storm)$
   c. $WQv = (0.55)(1.25) = 0.69 inches\ (Q)$
   d. $WQv = 0.69 inches \times 3.4 acres \times \frac{1}{12} \times 43{,}560\, ft^2/ac = 8{,}516 ft^3$

3. **Compute Q_{wq}.**
   a. Compute modified CN for 1.25-inch rainfall (P=1.25) (see Chapter 3, section 6):

$$CN = \frac{1000}{10 + 5P + 10Q - 10(Q^2 + 1.25QP)^{0.5}}$$

$$CN = \frac{1000}{[10 + (5)(1.25) + (10)(0.69) - 10(0.74^2 + (1.25)(0.69)(1.25)^{0.5})]}$$

$$CN = 96.15\ (Use\ CN = 96)$$

   b. For CN = 94 and an estimated time of concentration (t_c) of 13 minutes (0.22 hours) compute the Q_{wq} for the 1.25-inch storm. Results from WinTR-55 are:

$$Q_{wq} = 3.7\, ft^3/sec$$

4. **Compute channel protection volume (Cpv).** The criteria for stream channel protection, (Chapter 2), is 24-hour extended detention for the 1-year, 24-hour event. The Cpv is calculated here even though it will be provided downstream of the dry swale. The methodology for calculating the Cpv is summarized in Chapter 3, section 6. The WinTR-55 analysis performed above for the pre/post development scenarios provides data for this analysis.
   a. For the post-development scenario, CN = 87 and T_c = 0.22 hour, the peak discharge, Q_p, for the 1-year storm event is 5.32 cfs, or 1002.36 csm (See WinTR-55 result summary).

$$Q_p = q_u A_m Q F$$

b. The unit peak rate, q_u, is 835.3 csm/in (1002.26 csm/1.2 inches).

c. Using q_u and extended detention time, T, of 24 hours, find q_o/q_i for a Type II rainfall distribution. (See Chapter 3, section 6).

$$\frac{q_o}{q_i} = 0.03$$

d. For Type II rainfall distribution:

Equation C9-S3- 1

$$\frac{V_s}{V_r} = 0.683 - 1.43\left(\frac{q_o}{q_i}\right) + 1.64\left(\frac{q_o}{q_i}\right)^2 - 0.804\left(\frac{q_o}{q_i}\right)^3$$

$$\frac{V_s}{V_r} = 0.64$$

$$Cpv = V_s = \left(\frac{V_s}{V_r}\right)V_r = \frac{\frac{V_s}{V_r}Q_aA}{12}$$

Note: $V_r = Q_a$ (runoff in inches for developed 1-year storm from WinTR-55 analysis)

$$Cpv = \frac{(0.64)(1.2in)(3.4ac)}{12} = 0.218ac - ft = 9{,}479ft^3$$

5. **Determine overbank flood protection volume (Q_{25}).** For this site, the post-developed Q_{25} (15.85 cfs) must be controlled to the pre-development rate for the 5-year storm, $Q_{5\text{-}pre}$ (4.05 cfs).

From WinTR-55 analysis:

Area or Reach Identifier	STORM 25-YR		Runoff Amount (in)	Elevation (ft)	Time (hr)	Peak Flow	
	Drainage Area (sq mi)	Rain Gage ID or Location				Rate (cfs)	Rate (csm)
DA-1	0.005		3.697		12.02	15.85	2985.75

q_u = 807.6 csm/in q_o = 4.05 cfs q_i = 15.85 cfs q_o/q_i = 0.26

From Chapter 3, section 6 and a Type II rainfall distribution:

$$\frac{V_s}{V_r} = 0.405$$

$$V_s = \frac{(0.405)(3.697in)(3.40ac)}{12} = 0.424ac - ft = 18{,}480ft^3$$

Note: The channel protection volume (Cpv) and the detention storage for larger storm peak flow control will be provided by separate downstream detention storage for this site; no additional design details will be provided in this example.

6. **Determine if the development site and conditions are appropriate for the use of dry swale system.** Existing ground elevation at the facility location is 984 feet, mean sea level. Soil boring observations reveal that the seasonally high water table is at 976 feet, and underlying soils are silt loams (ML). Adjacent creek invert is at 972 feet.

7. **Confirm local design criteria and applicability.** There is a local requirement that the 25-year storm is contained within the top of banks of all channels, including the dry swale BMP controls. No additional local criteria are applicable.

8. **Determine pretreatment volume.** For this site, we will use two dry swale systems (DS-1 and DS-2) indicated in Figure C9-S3- 6. Provide two shallow forebays at the head of the swales equal to 0.05 inches per impervious acre of drainage (each). Note: total recommended pre-treatment requirement is 0.1 inches per impervious acre. For this site, the impervious area is 1.9 acres.

   $$(0.05in)(\frac{1ft}{12in})(43560\, ft^2/ac)(1.9ac) = 345ft^3$$

   Use a 2-foot deep pea gravel drain at the head of the swale to provide erosion protection, and to act as a level spreader to distribute the inflow. Any side flow at this site will be minimal, and will enter the swales as sheet flow.

9. **Determine swale dimensions.** Required: bottom width, depth, length, and slope necessary to store WQv with less than 18 inches of ponding. Use a trapezoidal channel with a measured maximum WQv depth of 18 inches. A shallow concrete wall configured with a low-flow orifice, with an integral trash rack will provide outlet flow control (see Figure C9-S3- 6 for location). A schematic of the outlet is illustrated in Figure C9-S3- 7.

   The total swale length of 1500 feet will be divided into two branches, as shown in Figure C9-S3- 6.

   Dry swale #1: 1000 feet
   Dry swale #2: 500 feet

   The outlet structure will be set at the existing invert minus 3 feet at (984 - 3 = 981 feet). The existing upstream invert for dry swale #1 is 992 (length of 1000 feet), and the existing invert for dry swale #1 is 988 (length of 500 feet).

   Slope of dry swale #1: (992 - 981)/1000 feet = 0.011 = 1.1%
   Slope of dry swale #2: (988 - 981)/500 feet = 0.014 = 1.4%

   Minimum slope is 1% (OK)

   For a trapezoidal cross section with a bottom width of 6 feet, a WQv average depth of 9 inches, 3:1 side slopes, compute the cross-sectional area:

   $Area = (6ft)(0.75ft) + (0.75ft)(2.25ft) = 6.2ft^2$ (See Figure C9-S3- 8)

   Calculate volume provided:

   $Swale\ volume = (6.2ft^2)(1500ft) = 9300ft^3$ $[9.300ft^3 > WQv\ of\ 8516ft^3]$ (OK)

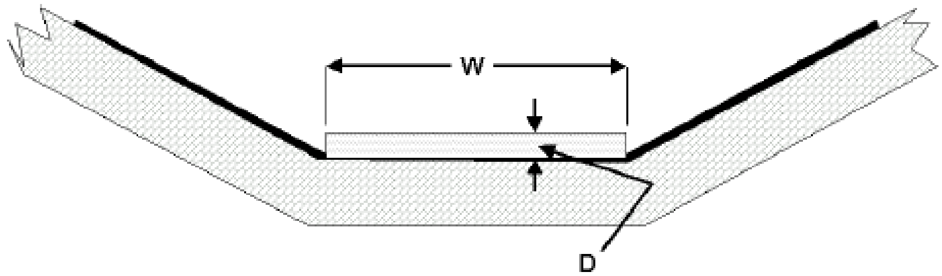
10. **Determine the number of check dams to be installed along the length of the swale to detain WQv (See Figure C9-S3- 9).** For dry swale #1, 1000 feet @1/1% slope and maximum depth of 18 inches, set check dams at spacing of $\frac{1.5ft}{0.011} = 136ft$; place at 135 ft ---- > 8 required.

    For dry swale #2, 500 feet @1.4% slope and maximum depth at 18 inches, set check dams at spacing of $\frac{1.5ft}{0.014} = 107ft$; place at 107-ft ----> 5 required.

11. **Determine the drain-down time.** To ensure the swales will drain down within the 24-hour criteria, the planting media will need to pass a maximum rate of 1.5 ft (18 in) in 24 hours (k = 1.5 ft/day). Provide a 6-inch perforated subdrain pipe and gravel system below the soil bed (see Figure C9-S3- 8).

12. **Check 2-year and 25-year flow for velocity erosion potential and freeboard.** Plan to provide 6 inches of

freeboard above the 25-year discharge water level. The 25-year peak discharge is 15.9 cfs; assume 70% (11.13 cfs) is routed through swale #1 and 30% (4.77 cfs) through swale #2.



From Manning's equation and trapezoidal channel: (for swale #1)

$$Q = VA = \left(\frac{1.49}{n}\right) D^{5/3} S^{1/2} W$$

Use n = 0.08 for swale surface condition (See Chapter 9, section 2):

$$D = \left[\frac{Qn}{1.49 S^{1/2} W}\right]^{3/5}$$

$$D = \left[\frac{(11.13cfs)(0.08)}{(1.49)(0.011^{1/2})(6ft)}\right]^{3/5} = 0.97ft \left(depth@Q_{p25}\right)$$

$$V = \frac{Q}{WD} = \frac{11.13cfs}{(6ft)(0.97ft)} = 1.91fps$$

Add 0.5 feet of freeboard above the top of the check dams, and add 25-year flow depth to determine the total swale depth:

$$1.5ft\ (berm) + 0.5ft\ (freeboard) + 0.97ft\ (25 - year\ flow\ depth) = 2.97ft\ (total\ depth\ of\ swale)$$

Use 3 feet total depth.

Determine required 25-year overflow weir length:

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

$$C = 3.1$$
$$Q_{25} = 15.9cfs$$
$$H = 0.97ft + 0.5ft = 1.47ft$$

$$L = \frac{15.0cfs}{[(3.1)(1.47^{1.5})]} = 2.88ft\ (Use\ 3\ feet)$$

13. **Design low-flow orifice at downstream headwall and checkdam (See Figure C9-S3- 7).** Design orifice to pass 8,516 cfs (WQv) in 6 hours:

$$\frac{8416ft^3}{(6hr)(3600\,sec/hr)} = 0.39cfs = 4cfs$$

Use orifice equation: $Q = CA(2gh)^{1/2}$

Assume H = 1.5 ft (18 inches)

$$A = \frac{0.4cfs}{\left[(0.6)\left((2)(32.2\,ft/sec^2)(1.5ft)\right)^{0.5}\right]} = 0.068ft^2$$

Diameter = $\left(\frac{0.068ft^2}{0.785}\right)^{0.5} = 0.29ft = 3.5inches\ (Use\ 4inch\ orifice)$

Provide a 3-inch V-notch slot in each check dam.

14. **Design inlets, sediment forebay, and subdrain piping.** Use minimum 4-inch Schedule 40 PVC perforated pipe (see Figure C9-S3- 9).

15. **Prepare vegetation, seeding, and landscaping plan.**

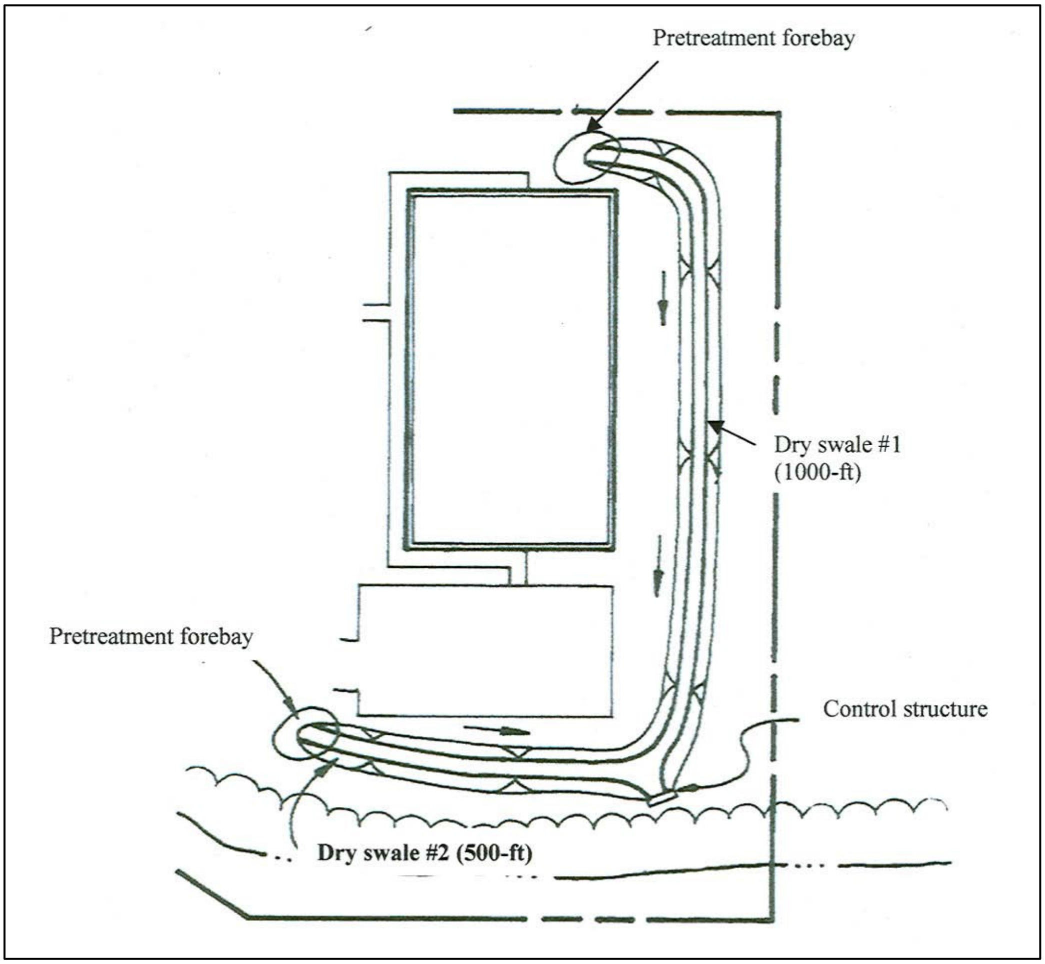


Figure C9-S3- 6: Site plan for dry swales

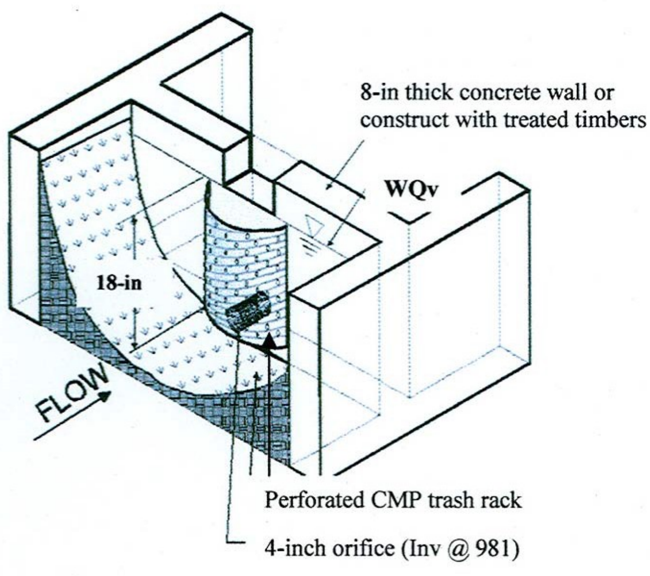


Figure C9-S3- 7: Control structure at swale outlet

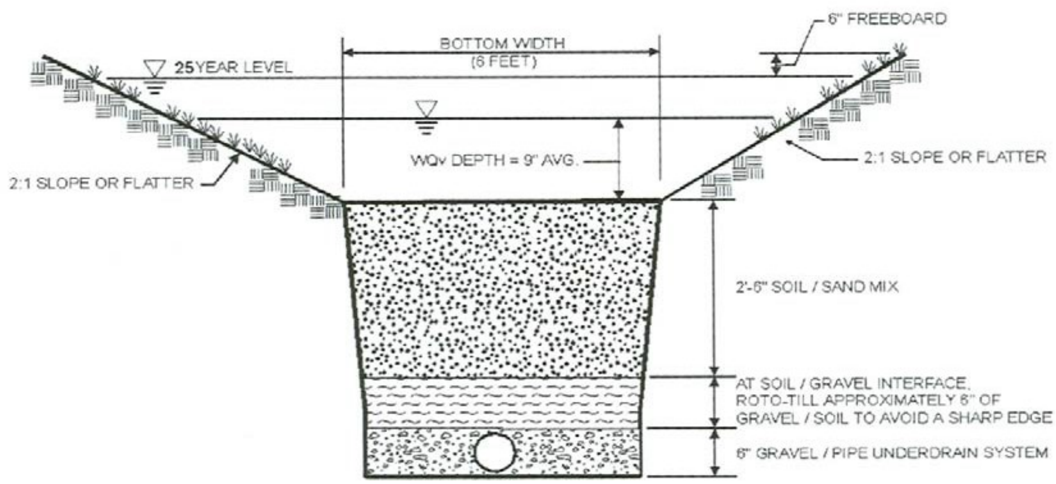


Figure C9-S3- 8: Trapezoidal dry swale section
Source: Adapted from Claytor and Schuler

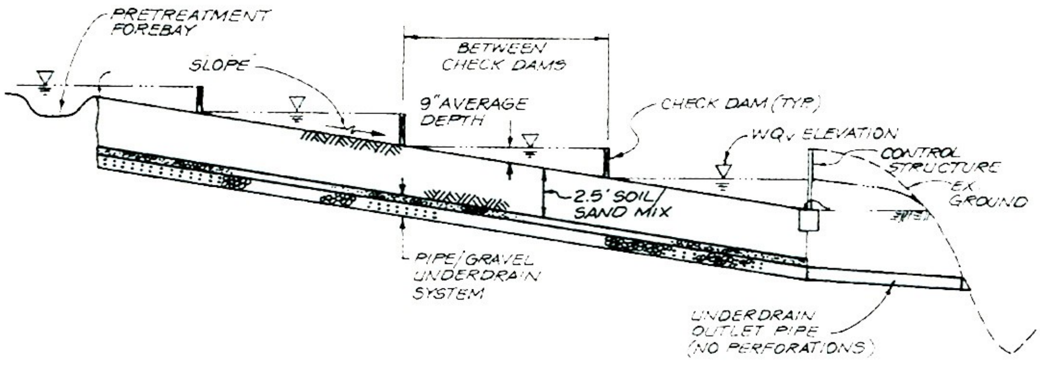


Figure C9-S3- 9: Dry swale #1 profile
Source: Adapted from Claytor and Schuler

Source: UDFCD, 1999

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

Description: Vegetated filter strips (VFS) are zones of vegetation through which sediment and pollutant-laden flow are directed before being discharged to a concentrated flow channel. They may closely resemble many natural ecological communities such as grassy meadows or riparian forests. Dense vegetative cover facilitates sediment attenuation and pollutant removal. VFS provide little treatment for concentrated flows. Grading and level spreaders are often used to create a uniformly sloping area to distribute the runoff evenly across the filter strip.

Typical uses:

- Manage runoff from residential sites, parking areas, and along perimeter of paved roadways.
- Located in a drainage easement at the rear of 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:

- Mitigates 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 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 and pollutant removal sensitive to proper design of slope and maintaining sufficient vegetation density.
- Limited to small areas (<5 acres); cannot be used on steep slopes (>6%).
- Possible re-suspension of sediment.

Maintenance requirements:

- Needs routine landscape maintenance; maintain grass height of approximately 2-4 inches.
- Inspect annually for erosion problems; seed or sod bare areas.

A. Description

Vegetated filter strips (VFS) are uniformly-graded and densely-vegetated sections of land engineered and designed to treat runoff and remove pollutants through vegetative filtering and infiltration. The primary purpose of a VFS is either to enhance the quality of stormwater runoff on small sites in a treatment system approach, or as a pre-treatment device for another BMP. The dense vegetative cover facilitates conventional pollutant removal through detention, filtration by vegetation, sediment deposition, and infiltration and adsorption in the soil (Yu and Kaighn, 1992). VFS can also be used as a pre-treatment BMP in conjunction with a primary BMP. This reduces the sediment and particulate pollutant load reaching the primary BMP, which in turn reduces the BMP maintenance costs and enhances its pollutant removal capabilities. Filter strips are located adjacent to impervious areas and can be used in residential and commercial areas and along highways and roads. Because their effectiveness depends on having an evenly-distributed sheet flow over their surface, the size of the contributing area and the associated volume of runoff have to be limited. Flow can be directly accepted from a parking lot, roadway, or building roof, provided the flow is distributed uniformly over the strip. They are ideal components of the outer zone of a stream buffer, or as pre-treatment for another structural stormwater control. Filter strips can serve as a buffer between incompatible land uses, landscaped to be aesthetically pleasing, and provide groundwater recharge in areas with pervious soils.

VFS are generally grouped into three categories:

- **Constructed filter strips.** Constructed filter strips are filter strips that are constructed and maintained to allow for overland sheet flow through the vegetation, primarily grass-like plants with density approaching that of tall lawn grass.
- **Natural vegetative strips.** Natural vegetative strips are any natural vegetative area through which sediment-laden flow is directed, including riparian vegetation around drainage channels. Flow is typically not broad overland sheet flow, but occurs in small concentrated flow channels or flow zones. Vegetation can range from grass-like plants to brush or trees with ground litter.
- **Riparian vegetative buffer strips.** Riparian vegetative buffer strips are strips of vegetation that grow along stream and concentrated flow channels. The vegetation may be constructed or natural. To be effective, the VFS will normally be located on the contour perpendicular to the general direction of flow.

Filter strips are often used as a stormwater site design credit when used in conjunction with other structural practices. Filter strips rely on the use of vegetation to slow runoff velocities and filter out sediment and other pollutants from urban stormwater. There can also be a significant reduction in runoff volume for smaller flows that infiltrate pervious soils while contained within the filter strip. To be effective, sheet flow must be maintained across the entire filter strip. Once runoff flow concentrates, it effectively short-circuits the filter strip and reduces the water quality performance. Therefore, a flow spreader must normally be included in the filter strip design.

There are two different filter strip designs:

1. A filter strip design that includes a permeable berm at the bottom (Figure C9-S4- 1). The presence of the berm increases the contact time with the runoff, thus reducing the overall width of the filter strip required to treat stormwater runoff.
2. A simple filter strip (See Figure C9-S4- 2).

Filter strips are typically an online practice, so they must be designed to withstand the full range of storm events without eroding. The design approach for filter strips involves site design techniques to maintain prescribed maximum sheet flow distances, as well as checking to ensure adequate temporary storage for the WQv for a 24-hour period. Filter strips are also designed using volume-based sizing criteria.

Therefore, the use of filter strips to treat stormwater runoff is primarily a function of limiting the flow path to the filter. One of the main abuses of the past has been draining too much area through the filter strip. In most cases the sheet flow distance limitations will be the controlling factor. Figure C9-S4- 1 and Figure C9-S4- 2 illustrate the primary design components of the two filter strip design variants.

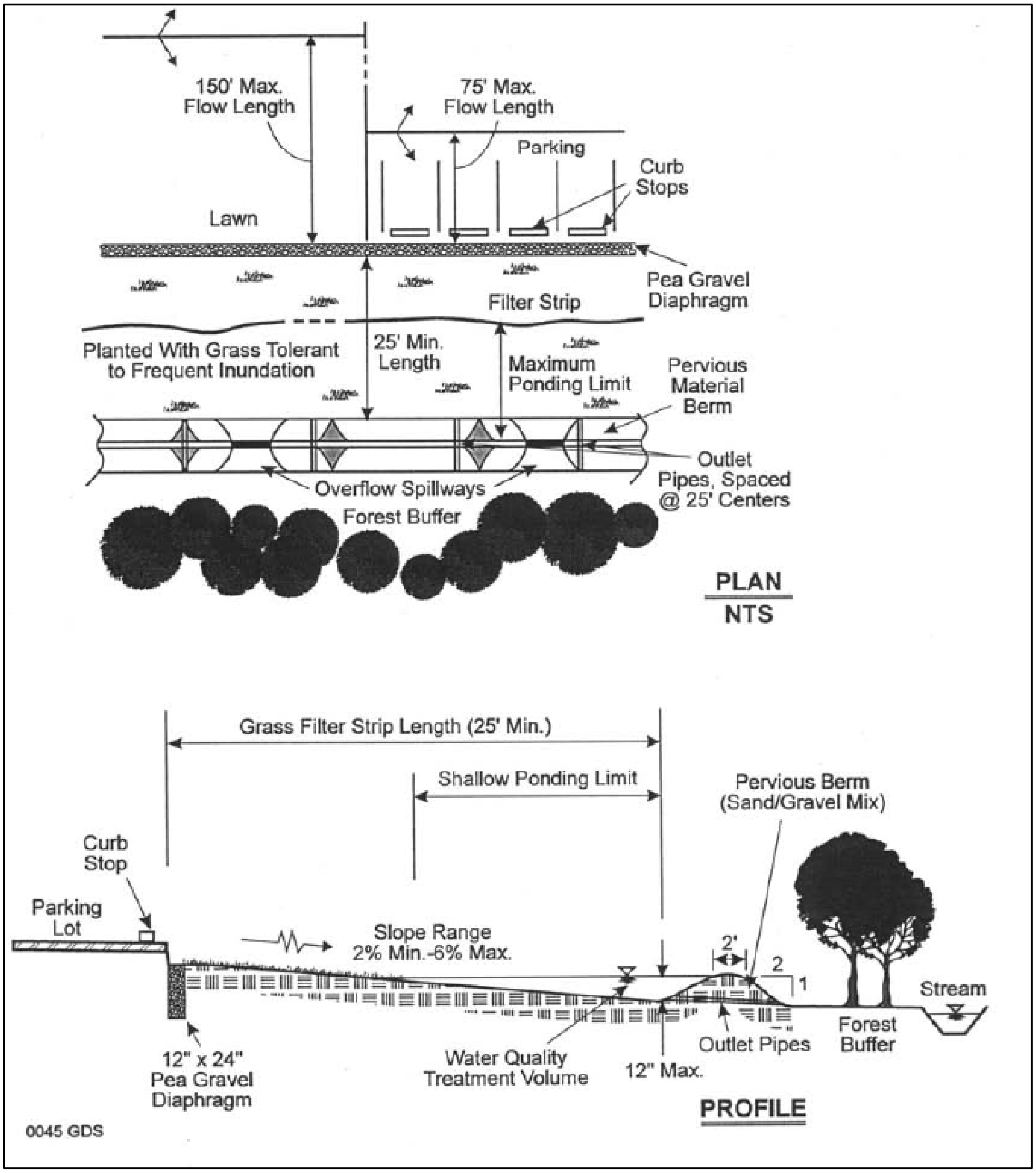


Figure C9-S4- 1: Vegetated filter strip (with berm)
Source: Claytor and Schueler, CRC, 1996

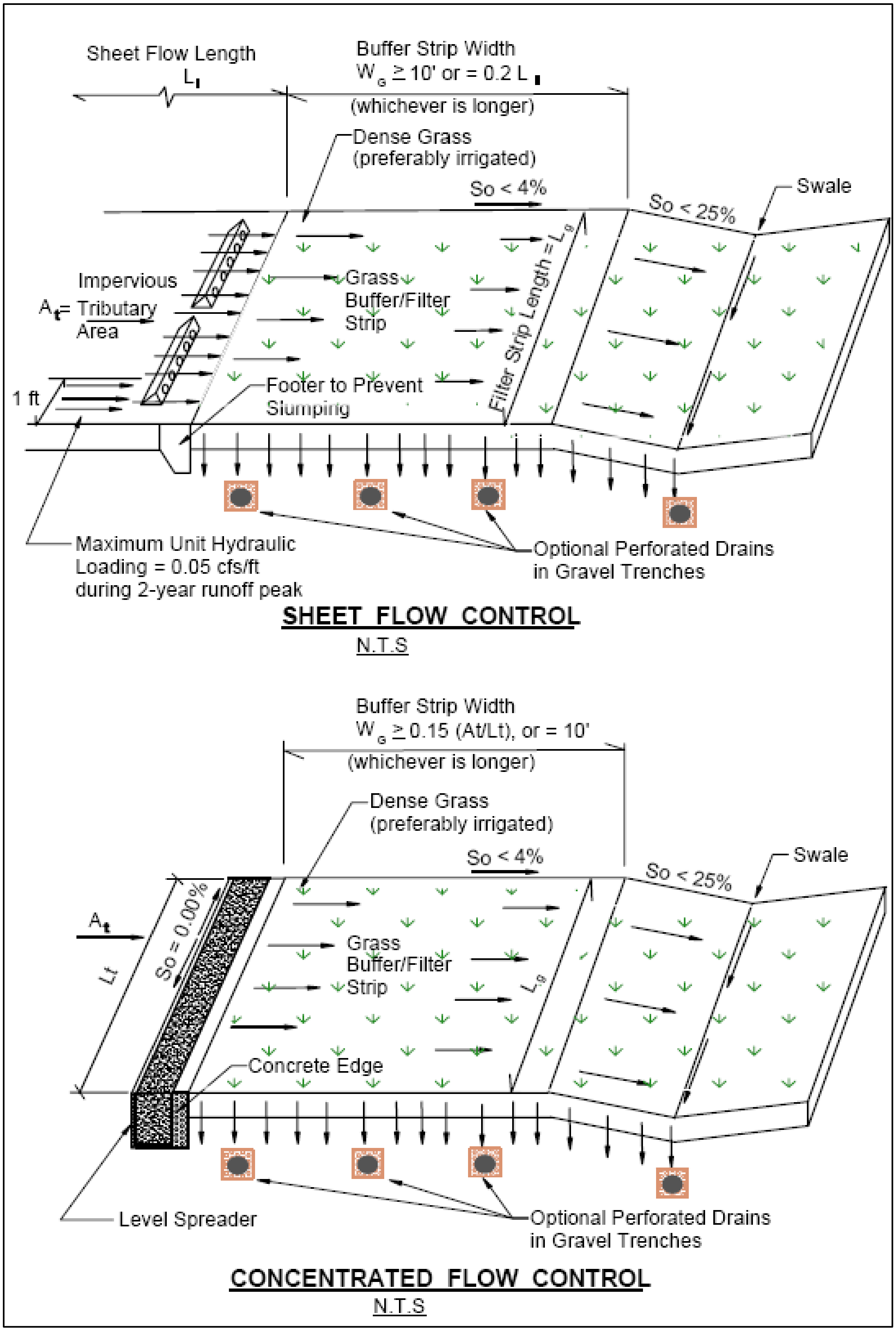


Figure C9-S4- 2: Grass buffer filter strip (without berm)
Source: UDFCD, 1999

B. Pollutant removal capabilities

Filter strips provide only marginal pollutant removal and require that follow-up structural BMPs be provided. Pollutant removal from filter strips is highly variable and depends primarily on density of vegetation and contact time for filtration and infiltration. These in turn depend on soil and vegetation type, slope, and presence of sheet flow. The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment.

- Total suspended solids – 50%
- Total phosphorus – 20%
- Total nitrogen – 20%
- Fecal coliform – insufficient data
- Heavy metals – 40%

C. Design criteria

1. General criteria.

a. Filter strips should be used to treat small drainage areas. Flow must enter the filter strip as sheet flow spread out over the width (long dimension normal to flow) of the strip, generally no deeper than 1-2 inches. As a rule, flow concentrates within a maximum of 75 feet for impervious surfaces, and 150 feet for pervious surfaces (Claytor and Schueler, 1996). For longer flow paths, special provisions must be made to ensure design flows spread evenly across the filter strip.

b. Filter strips should be integrated within site designs.

c. Filter strips should be constructed outside the natural stream buffer area whenever possible to maintain a more natural buffer along the streambank.

d. Filter strips should be designed for slopes between 2-6%. Greater slopes than this would encourage the formation of concentrated flow. Flatter slopes would encourage standing water.

e. Filter strips should not be used on soils that cannot sustain a dense grass cover with high retardance. Designers should choose a grass that can withstand relatively high-velocity flows at the entrances, and both wet and dry periods. See SUDAS Specifications Manual Section 9010 for a list of appropriate grasses for use in Iowa.

f. The filter strip should be at least 15 feet long to provide filtration and contact time for water quality treatment. Twenty-five feet is preferred (where available), though length will normally be dictated by design method.

g. Both the top and toe of the slope should be as flat as possible to encourage sheet flow and prevent erosion.

h. An effective flow spreader is to use a pea gravel diaphragm (a small trench running along the top of the filter strip) at the top of the slope (ASTM D 448 size no. 6, ⅛-⅜-inch). The pea gravel diaphragm serves two purposes. First, it acts as a pre-treatment device, settling out sediment particles before they reach the practice. Second it acts as a level spreader, maintaining sheet flow as runoff flows over the filter strip. Other types of flow spreaders include a concrete sill, curb stops, or curb and gutter with saw-teeth cut into it.

i. Ensure that flows in excess of design flow move across or around the strip without damaging it. Often a bypass channel or overflow spillway with protected channel section is designed to handle higher flows.

j. Pedestrian traffic across the filter strip should be limited by providing a designated walkway.

k. Maximum discharge loading per foot of filter strip width (perpendicular to flow path) is found using Manning's equation:

Equation C9-S4- 1

$$q = \left(\frac{0.00236}{n}\right) Y^{5/3} S^{1/2}$$

Where:
q = discharge per foot of width of filter strip (ft^3/sec-ft)
Y = allowable depth of flow (inches)
S = slope of filter strip (%)
n = Manning's roughness coefficient ($s/ft^{1/3}$)

Use 0.15 for medium grass, 0.25 for dense grass, and 0.35 for very dense grass. (See Table C9-S4- 1).

l. The minimum width of a filter strip is:

Equation C9-S4- 2

$$W_{f-MIN} = \frac{Q}{q}$$

Where:
$W_{f\text{-}MIN}$ = minimum filter strip width perpendicular to flow (feet)

2. **Filter strip without berm.**
   a. Size filter strip (parallel to flow path) for a contact time of 5 minutes minimum.
   b. Equation for filter length is based on the SCS TR-55 travel time equation (SCS, 1986):

Equation C9-S4- 3

$$L_f = \frac{{T_t}^{1.25}{P_{2-24}}^{0.625}S^{0.5}}{3.34n}$$

Where:
L_f = length of filter strip parallel to flow path (ft)
T_t = travel time through filter strip (minutes)
$P_{2\text{-}24}$ = 2-yr, 24-hr rainfall depth (inches)
S = slope of filter strip (%)
n = Manning's roughness coefficient

Use 0.15 for medium grass, 0.25 for dense grass, and 0.35 for very dense grass. (See Table C9-S4- 1).

3. **Filter strip with berm.**
   a. Size outlet pipes to ensure that the water detained by the berm drains within 24 hours.
   b. Specify grasses resistant to frequent inundation within the shallow ponding limit.
   c. Berm material should be of sand, gravel and sandy loam to encourage grass cover (Sand: ASTM C-33 fine aggregate concrete sand 0.02-inch to 0.04-inch; Gravel: ½-inch to 1-inch: use IA DOT #3-coarse PCC aggregate or IA DOT #29-porous backfill).
   d. Size filter strip to contain the WQv within the wedge of water backed up behind the berm.
   e. Maximum berm height is 12 inches.

4. **Filter strips for pre-treatment.**
   a. A number of other structural controls, including bioretention areas and infiltration trenches, may utilize a filter strip as a pre-treatment measure. The required length of the filter strip depends on the drainage area, imperviousness, and the filter strip slope. Table C9-S4- 1 provides sizing guidance for bioretention filter strips for pre-treatment.

Table C9-S4- 1: Pre-treatment filter strip sizing guidance

Parameter	Impervious Areas				Pervious Areas (i.e. Lawns)			
Maximum inflow approach length (feet)	35		75		75		100	
Filter strip slope (maximum = 6%)	<2%	>2%	<2%	>2%	<2%	>2%	<2%	>2%
Filter strip minimum length (feet)	10	15	20	25	10	12	15	18

Source: Claytor and Schueler, 1996

D. Inspection and maintenance requirements

Filter strips require similar maintenance to other vegetative practices. Maintenance is very important for filter strips, particularly in terms of ensuring that flow does not short-circuit the practice.

Table C9-S4- 2: Typical maintenance activities for vegetated filter strips

Activity	Schedule
Mow grass to maintain a 2-4 inch height.	Regularly (as required seasonally)
Inspect pea gravel diaphragm for clogging and remove accumulated sediment.	Annual inspection (semi-annual first year)
Inspect vegetation for rill and gully erosion. Seed or sod bare areas.	
Inspect to ensure that grass has established. If not, replace with alternate species.	

Source: Claytor and Schueler, CRC, 1996

E. Design example

Filter strip.

1. **Basic site data.** Small commercial parcel located in Ankeny, IA
   a. Parcel size: 240 feet deep x 200 feet wide
   b. Drainage area (A) = 1.1 acres
   c. Soils – HSG-B
   d. Roof and paved parking: 28,800-ft2 (0.66 acres)
   e. Impervious percentage (I) = 60%
   f. Slope = 3.6%, Manning's n = 0.25
   g. Design for allowable flow depth of 1 inch

Rainfall Data for Polk County Example (24-hr duration)	
Return period	Rainfall, P (inches)
0.3-yr (WQ event)	1.25
1-yr	2.38
2-yr	3.2
5-yr	4.1
10-yr	4.7
25-yr	5.5
100-yr	6.7

Figure C9-S4- 3: Data for Polk County example

2. **Calculate maximum discharge loading per foot of filter strip width.** Use Equation C9-S4- 1:

$$q = \left(\frac{0.00236}{0.25}\right)\left(1.0^{5/3}\right)\left(3.6^{0.5}\right) = 0.018\,ft^3/sec - ft$$

3. **Compute WQv and WQv peak flow (Q_{wq}).**

   a. Compute water quality volume in inches:
      1) $P = 1.25 inches$
      2) $Rv = 0.05 + 0.009(6) = 0.59$
      3) $WQv = PRv = 1.25 inches(0.59) = 0.74 inches$

   b. Compute modified CN for 1.25-inch rainfall (P=1.25). (See Chapter 3, section 7).

$$CN = \frac{1000}{[10 + 5P + 10Q - 10(Q^2 + 1.25QP)^{0.5}]}$$

$$CN = \frac{1000}{[10 + 5(1.25) + 10(0.74) - 10(0.74^2 + 1.25(0.74)(1.25))^{0.5}]}$$

$$CN = 94.36\ (Use\ CN = 94)$$

   c. For CN = 94 and an estimated time of concentration (T_c) of 8 minutes (0.13 hours) compute the Q_{wq} for the 1.25-inch storm. Results from WinTR-55 are:

      Q_{wq} = 1.21 ft^3/sec

   d. For standard CN based on actual impervious and pervious area on B soils: CN=83. Compute Q_{p2} and Q_{p10} for developed condition. Results from WinTR-55 are:

      Q_{p2} = 2.73 cfs Q_{p10} = 4.9 cfs

4. **Compute minimum filter width. Use Equation C9-S4- 2:**

$$W_{f-MIN} = \frac{Q}{q} = \frac{1.21\,ft^3/sec}{0.018\,ft^3/sec - ft} = 67ft$$

Since the width of the parcel is 200 feet, the actual width of the filter strip can depend on site grading and the flow path configuration to the filter strip in sheet flow through a pea gravel level spreader.

5. **Filter without a berm.**
   a. 2-yr, 24-hr storm for Polk County = 3.2 inches
   b. Use 5-minute travel contact time
   c. Use Equation C9-S4- 3:

$$L_f = \frac{5^{1.25} 3.2^{0.625} 3.6^{0.5}}{3.34(0.25)} = 35ft$$

Note: Reducing the filter strip slope to 2% and planting a denser grass (raising the Manning n to 0.35) would reduce the filter strip length to 22 feet. Sensitivity to slope and Manning's n changes are illustrated for this example in Figure C9-S4- 4.

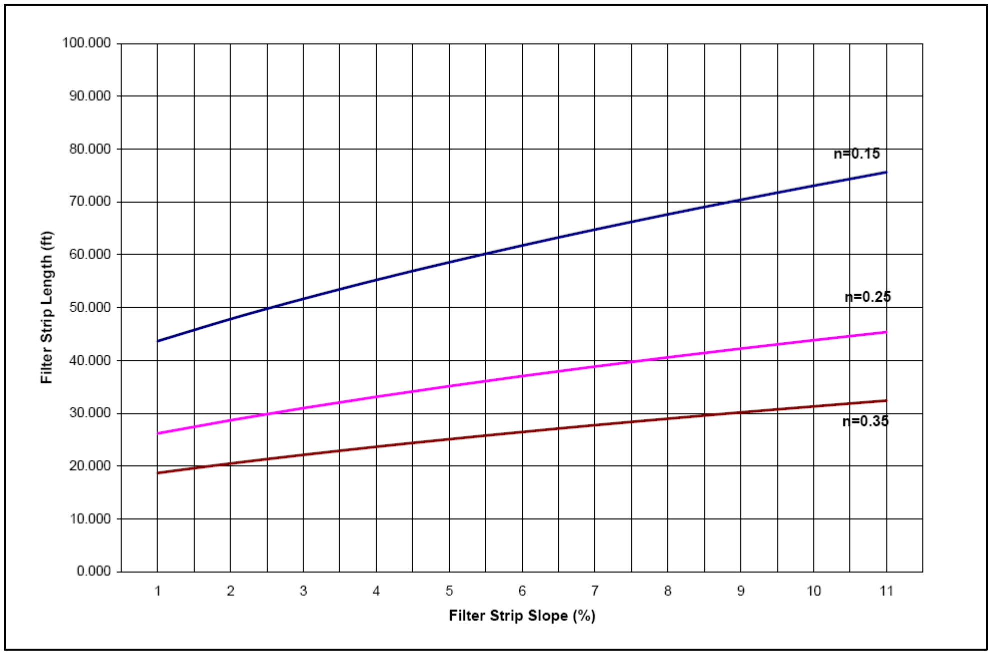


Figure C9-S4- 4: Example problem sensitivity of filter strip length to slope and Manning's n

6. **Filter with berm.**
   a. Pervious berm height is 8 inches
   b. Compute WQv in cubic feet: (Use WQv result in Step 3.a)

$$WQv = (0.74inches)(1.1acres)\left(\frac{1ft}{12in}\right)(43460\,ft^2/acre) = 2955ft^3$$

   c. For a berm height of 8 inches, the volume of the wedge of water captured by the filter strip is:

$$Volume = W_f L_f \left(\frac{1}{2}\right)(0.67ft) = 0.335 W_f L_f = 2955ft^3$$

   d. For a maximum width of the filter of 200 feet, the length of the filter would then be 44 feet.
   e. For a 1-foot berm height, the length of the filter would be 30 feet.