Iowa Storm Water Management Manual

Design Standards Chapter 5- Infiltration Practices

A. Introduction

The emerging goal of urban stormwater management is to achieve effective control of pollutants in stormwater runoff and reduce the volume and rate of runoff to control downstream impacts from flooding and stream-channel erosion. Best management practices (BMPs) that mirror the natural process of infiltration found in undeveloped watersheds can effectively increase the volume of water returned to the soil and reduce the volume of direct runoff to streams and sewers. Increased infiltration will maintain pre-development baseflow in local streams, and also help reduce the frequency of bank-full flow in urban stream channels. Infiltration practices are the one group of BMPs that can effectively reduce the volume of net annual direct runoff to streams. When site conditions permit, a portion of urban stormwater runoff can be managed through infiltration. The water volume from infiltration is transferred to the soil-water system and released slowly over time through the local water table and into local and regional stream baseflow. Additional water is transferred back into the atmosphere through evapotranspiration.

B. Infiltration fundamentals

Infiltration is the downward movement of water from the land surface into the soil profile. Infiltration can occur naturally following precipitation, or can be induced artificially through structural modifications in the ground surface. Some water that infiltrates will remain in the shallow soil layer, where it will gradually move vertically and horizontally through the soil and subsurface material. Eventually, it might enter a stream by seepage into the stream bank. Some of the water may continue to move deeper (percolate), recharging the local groundwater aquifer. A dry soil has a defined capacity for infiltrating water. The capacity can be expressed as a depth of water that can be infiltrated per unit time, such as inches per hour. If rainfall supplies water at a rate that is greater than the infiltration capacity, water will infiltrate at the capacity rate, with the excess either being ponded, moved as surface runoff, or evaporated. If rainfall supplies water at a rate less than the infiltration capacity, all of the incoming water volume will infiltrate. In both cases, as water infiltrates into the soil, the capacity to infiltrate more water decreases and approaches a minimum capacity. When the supply rate is equal to or greater than the capacity to infiltrate, the minimum capacity will be approached more quickly than when the supply rate is much less than the infiltration capacity.

1. **Infiltration.** The downward entry of water into the immediate surface of soil or other materials.
2. **Infiltration capacity.** The maximum rate at which water can infiltrate into a soil under a given set of conditions.
3. **Infiltration rate.** The rate at which water penetrates the surface of the soil, expressed in cm/hr, mm/hr, or inches/hr. The rate of infiltration is limited by the capacity of the soil and the rate at which water is applied to the surface. This is a volume flux of water flowing into the profile per unit of soil surface area (expressed as velocity).
4. **Percolation.** Vertical and lateral movement of water through the soil by gravity.

As precipitation infiltrates into the subsurface soil, it generally forms an unsaturated (vadose) zone and a saturated (phreatic) zone. In the unsaturated zone, the voids (spaces between grains of gravel, sand, silt, clay, and cracks within rocks) contain both air and water. Although a lot of water can be present in the unsaturated zone, this water cannot be pumped by wells because it is held too tightly by capillary forces. The upper part of the unsaturated zone is the soil-water zone. The soil zone is crisscrossed by roots, openings left by decayed roots, and animal and worm burrows, which allow the precipitation to infiltrate into the soil zone. Water in the soil is used by plants in life functions and leaf transpiration, but it also can evaporate directly to the atmosphere. Below the unsaturated zone is a saturated zone where water completely fills the voids between rock and soil particles.

Water movement in the vadose zone is generally conceptualized as occurring in the three stages of infiltration, redistribution, and drainage or deep percolation, as illustrated in Figure C5-S1- 1. As described above, infiltration is defined as the initial process of water entering the soil resulting from application at the soil surface. Capillary forces or matric (negative pressure) potentials are dominant during this phase. Redistribution occurs in the next stage where the infiltrated water is redistributed within the soil profile after water application to the soil surface stops. During redistribution, both capillary and gravitational effects are important. Simultaneous drainage and wetting takes place during this stage. Evapotranspiration takes place concurrently during the redistribution stage, and will impact the amount of water available for deeper penetration within the soil profile. The final stage of water movement is termed deep percolation or recharge, which occurs when the wetting front reaches the water table. The term “infiltration” is typically used as a single terminology to describe all three stages of water movement through the vadose zone. The terms, “water flux", "infiltration rate", and "rate of water movement" are also used interchangeably.

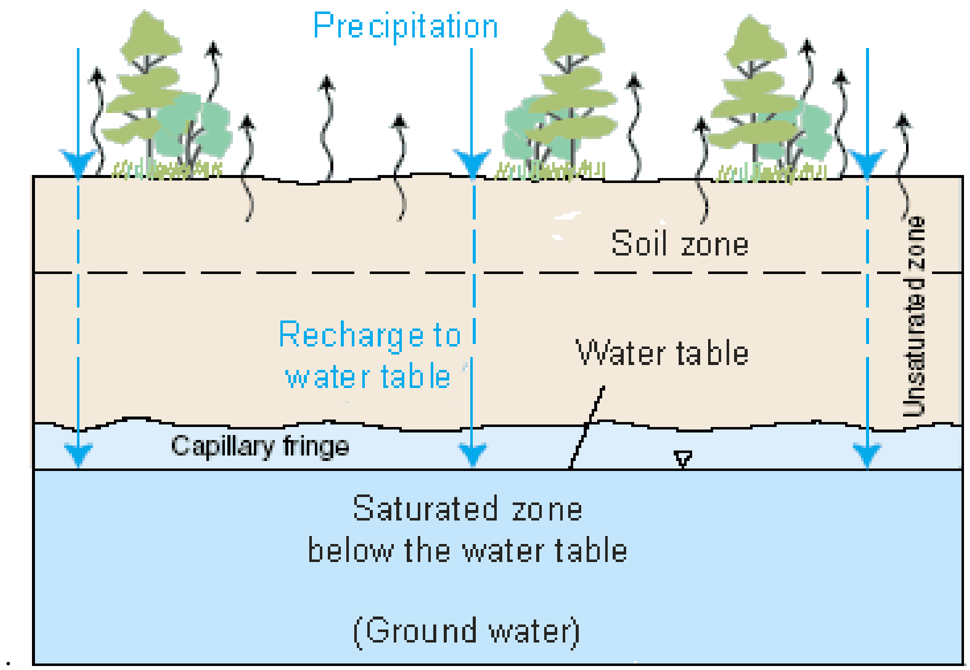


Figure C5-S1- 1: Water infiltration through the soil-water unsaturated zone and into the water table

The distribution of water during the infiltration process under ponded conditions is illustrated in Figure C5-S1- 2. In this idealized profile for soil-water distribution for a homogeneous soil, five zones are illustrated for the infiltration process.

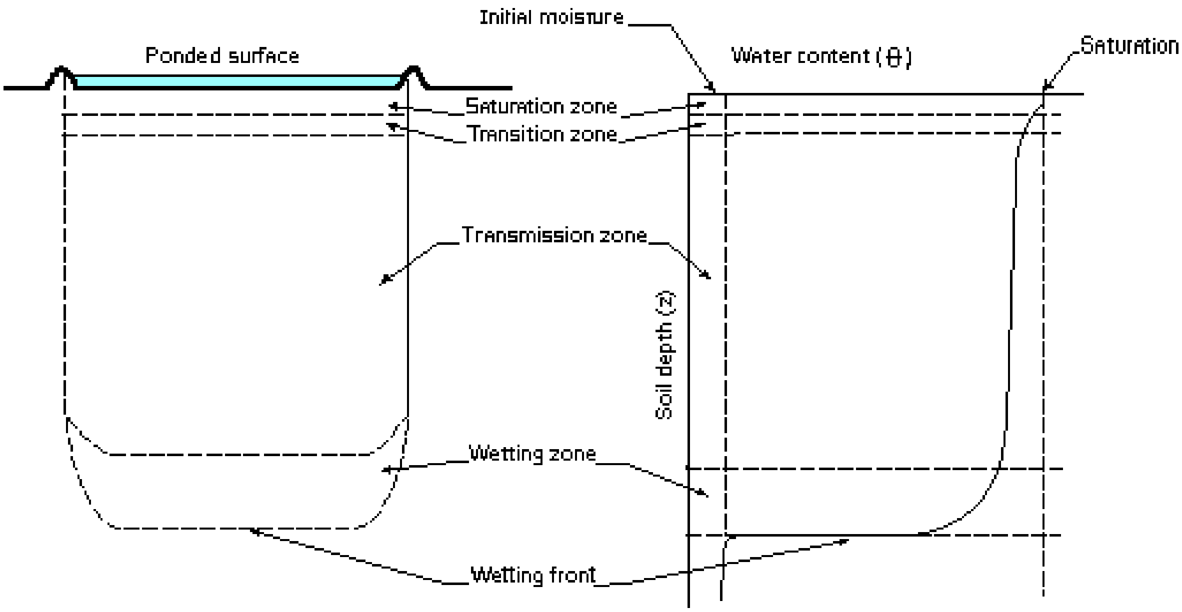


Figure C5-S1- 2: Zones of the infiltration process for the water content profile under ponded conditions

1. **Saturated zone.** The pore space in this zone is filled with water (saturated). Depending on the length of time elapsed from the initial application of water, this zone will generally extend only to a depth of a few millimeters.
2. **Transition zone.** This zone is characterized by a rapid decrease in water content with depth, and will extend a few centimeters.
3. **Transmission zone.** This zone is characterized by a small change in water content with depth. In general, the

transmission zone is a lengthening unsaturated zone with uniform water content. Gravity forces primarily drive hydraulic gradient in this zone.

4. **Wetting zone.** In this zone, the water content sharply decreases with depth from the water content of the transmission zone to near the initial water content of the soil.
5. **Wetting front.** This zone is characterized by a steep hydraulic gradient, and forms a sharp boundary between the wet and dry soil. The hydraulic gradient is characterized primarily by matric potentials.

Beyond the wetting front, there is no visible penetration of water. A comprehensive review of the principles governing the infiltration process has been published by Hillel (1982). Soil-water infiltration is controlled by the rate and duration of water application, soil physical properties, slope, vegetation, and surface roughness. Generally, whenever water is ponded over the soil surface, the rate of infiltration exceeds the soil infiltration capacity. On the other hand, if water is applied slowly, the infiltration rate may be slower than the soil infiltration capacity, and the supply rate becomes a determining factor for the infiltration rate. This type of infiltration process is termed “supply controlled” (Hillel, 1982). However, once the infiltration rate exceeds the soil infiltration capacity, it is the latter which determines the actual infiltration rate, and thus the process becomes profile-controlled. Generally, soil-water infiltration has a high rate in the beginning, decreases rapidly, and then slowly decreases until it approaches a constant rate. As shown in Figure C5-S1- 3, the infiltration rate will eventually become steady and approach the value of the saturated hydraulic conductivity.

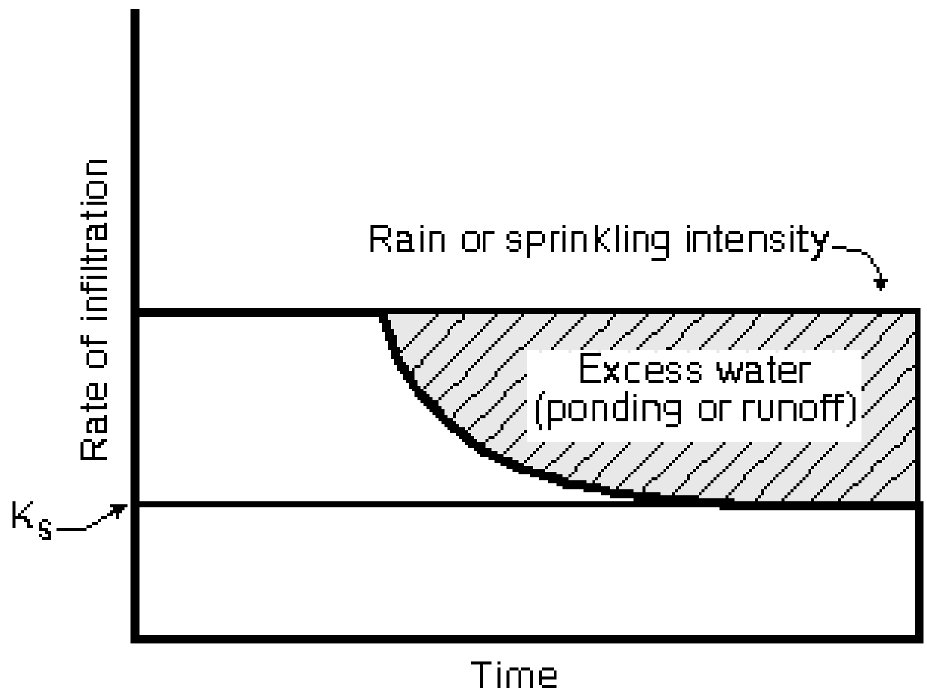


Figure C5-S1- 3: Decrease of infiltration rate
Source: Hillel, 1982

The slope of the land can also indirectly impact the infiltration rate. A steep slope will result in runoff, which will impact the amount of time the water will be available for infiltration. In contrast, gentle slopes will have less of an impact on the infiltration process due to decreased runoff. When compared to the bare soil surface, vegetation cover tends to increase infiltration by retarding surface flow, allowing time for water infiltration. Plant roots may also increase infiltration by increasing the hydraulic conductivity of the soil surface through the creation of additional pore space. Due to these impacts, infiltration may vary widely under different types of vegetation. The movement of water is described below.

1. **Groundwater.** Groundwater occupies the zone of saturation.
2. **Percolation.** Groundwater moves downward through the soil by percolation.
3. **Seepage.** Groundwater then moves toward a stream channel or large body of water as seepage.
4. **Water table.** The water table separates the zone of aeration (vadose zone) from the zone of saturation (phreatic zone). The water table fluctuates with moisture conditions; during wet times the water table will rise as more pore spaces are occupied with water.

5. **Aquifers.** Groundwater is found in these bodies of earth material that have the ability to hold and transmit water. Aquifers can be either unconfined or confined.
   a. **Unconfined.** Open aquifers connected to the surface above.
   b. **Confined.** Closed aquifers sandwiched between dense impermeable layers of earth material.

6. **Aquiclude.** Dense impermeable layers of earth material between which confined aquifers exist.
7. **Recharge zone.** Groundwater is replenished in the recharge zone of a confined aquifer, where the aquifer is exposed at the surface and water can enter it.

C. Infiltration systems

Surface infiltration can be achieved through the use of grass buffer strips, vegetated swales, and porous pavement systems. Infiltration systems such as infiltration trenches, infiltration basins, and bioretention areas (including rain gardens) are designed specifically to capture a defined volume of storm runoff and transfer it directly to the soil profile. Several integrated practices, such as soil quality restoration and native landscaping, can be used in conjunction with these practices to improve the infiltration capacity of compacted urban soils. An infiltration BMP is designed to capture a volume of stormwater runoff, retain it, and infiltrate all or part of that volume into the ground.

Infiltration of stormwater has a number of advantages and disadvantages. The advantages of infiltration include both water quantity control and water quality control.

1. Water quantity control can occur by capturing and retaining surface runoff and infiltrating the water into the underlying soil, reducing the volume of water discharged directly to receiving streams. Infiltration systems can be designed to capture the volume of stormwater from the smaller, more frequent storm events (water quality volume) and infiltrate this water into the ground over a period of several hours or days. Infiltration can provide a secondary benefit by increasing recharge of underlying aquifers and increasing baseflow levels of nearby streams.

2. Water quality treatment can be attained when pollutant removal occurs as water percolates through the various soil layers. As the water moves through the soil, particles can be filtered out. In addition, microorganisms in the soil can degrade organic pollutants that are contained in the infiltrated stormwater.

There are two general types of situations where infiltration practices may be used:

1. For determining the dimensions of an infiltration device that is required to provide storage of the WQv, Cpv, and/or Q_p.

2. Site conditions may dictate the layout and capacity of infiltration measures, and one might be interested in determining the level of control provided by such a layout. In the latter case, control may not be sufficient. Additional control, possibly from other BMPs, may be needed.

The same principles of design apply to both situations.

Although infiltration of stormwater has many benefits, it also has some drawbacks:

1. Infiltration may not be appropriate in areas where groundwater is a primary source of drinking water, due to the potential for contaminant migration. This is especially true if the runoff is from a commercial or industrial area where there may be contamination from organics or metals.

2. The performance of infiltration BMPs will also be limited in areas with low-permeability soils.

3. In addition, infiltration BMPs can experience reduced infiltrative capacity, and even clogging, due to excessive sediment accumulation. Frequent maintenance may be required to restore the infiltrative capacity of the system. Care must also be taken during construction to limit sediment generation and compaction of the soil layers underlying the BMP, to avoid reducing the infiltrative capacity.

D. Infiltration hydraulics and process

A fundamental principle for describing the flow of water in a homogeneous, porous media is given by Darcy's Law (Chow, Maidment, and Mays, 1988; McCuen, 1989):

Equation C5-S1- 1

$$Q = KA\Delta h/L$$

Where:
Q = flow (cfsec)
K = saturated hydraulic conductivity; characteristic of a specific porous medium when effectively saturated with water (fps)
A = cross-sectional area through the porous medium perpendicular to the flow (ft^2)
$\Delta h/L$ = hydraulic gradient, the difference in hydraulic head, Δh, per unit distance in the direction of flow, L ft/ft

The velocity of flow through the porous medium can be determined from Equation C5-S1- 1 by substituting the continuity equation Q = qA to obtain:

$$q = K\left(\frac{\Delta h}{L}\right)$$

q = velocity of water through a unit cross-section of the porous medium (fps)

The velocity of water through the pores of the medium is described by:

$$V = \frac{q}{\theta_s}$$

Where:
V = fluid velocity (in/hr)
θ_s = water content of the medium (in^3/in^3) equal to the medium's porosity less the volume of trapped air in the pore spaces

The infiltration rate is the flux of water into the soil in units of in/hr (Hillel, 1980). As shown in Figure C5-S1- 2 and Figure C5-S1- 3, infiltration downward into an initially dry soil occurs under the combined influence of ponding head and suction gradient (Hillel, 1980). As the water penetrates deeper and the transmission zone lengthens, the suction gradient decreases because the difference in matric suction between the saturated soil surface and the unwetted soil below the wetting front divides itself along an increasing distance (L). The suction gradient eventually becomes negligible and the gravity gradient becomes the remaining force pushing water downward. In vertical flow, each unit of decline in ponding depth (L) leads to an equal loss of gravity head (Δh), so the gravity gradient has a value of unity. Early in a ponding event, the total hydraulic gradient is higher than unity since the suction gradient and gravity gradient are both significant and acting together. Over time, the total hydraulic gradient declines approaching a lower limiting value of 1. As long as the vertical profile is homogeneous, the vertical movement of water settles down to a steady, gravity-induced rate approaching the hydraulic conductivity as a lower limiting value. Work done by Bouwer (1966) suggests a safety factor of 0.5 be applied to the measured soil hydraulic conductivity to account for any decrease in conductivity due to plugging of the soil interface and air trapped in the pore spaces.

E. Soils and infiltration

Factors that control infiltration rate and capacity:

- Vegetative cover, root development, and organic content
- Moisture content
- Soil structure and texture
- Porosity and permeability
- Soil bulk density and compaction
- Slope, landscape position, and topography

1. **Hydrologic soil group (HSG).** The HSG refers to the soil characteristics that tend to decrease or increase the amount of runoff produced from a precipitation event. The HSG is used in the determination of the runoff curve number (CN) developed by the Natural Resource Conservation Service (NRCS).

   a. Group A.
      1) Sand, loamy sand, or sandy loam soil types.
      2) Low runoff potential and high infiltration rates, even when thoroughly wetted.
      3) Includes deep and well- to excessively-drained sands and gravels.
      4) High rate of water transmission (hydraulic conductivity).

   b. Group B.
      1) Silt loam or loam.
      2) Moderate infiltration rate when thoroughly wetted.
      3) Includes moderately deep to deep, moderately well- to well-drained soils.
      4) Moderately fine to moderately coarse textures.

   c. Group C.
      1) Sandy clay loam.
      2) Low infiltration rates when thoroughly wetted.
      3) Consists primarily of soils with a layer that impedes downward movement of water.
      4) Moderately fine to fine structure.
      5) Perched water table at 40-60 inches; root-limiting at 20-40 inches.

   d. Group D.
      1) Clay loam, silty clay loam, sandy clay, silty clay, and clay.
      2) Very low infiltration rates when thoroughly wetted.
      3) Consists chiefly of clay soils with high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material.

2. **Soil texture.** The hydrologic design methods presented are based on the use of two hydrologic soil properties; the effective water capacity (C_w) and the minimum infiltration rate (*f*) of the specific soil textural groups, as shown in Table C5-S1- 1.

   a. **Effective water capacity.** The fraction of the void spaces available for water storage (in/in).
   b. **Minimum infiltration rate.** The final rate that water passes through the soil profile during saturated conditions (in/hr).

   The hydrologic soil properties are obtained by identifying the soil textures with a gradation test for each change in soil profile. The soil textures presented in Table C5-S1- 1 correspond to the soil textures of the US Department of Agriculture (USDA) Textural Triangle presented in Figure C5-S1- 4. The data presented in Table C5-S1- 1 are based on the analysis of over 5,000 soil samples by the USDA under carefully controlled procedures. The use of the soil properties established in the table for design and review procedures will offer two advantages. First, it provides for consistency of results in the design procedures. Second, it eliminates the need for the laborious and costly process of conducting field and laboratory infiltration and permeability tests.

   Based on the soil textural classes and the corresponding minimum infiltration rates, a restriction is established to eliminate unsuitable soil conditions. Soil textures that are recommended for infiltration systems include those soils with infiltration rates of 0.52 in/hr or greater, which include loam, sandy loam, loamy sand, and sand (soil clay content of less than 20% and a silt/clay content of less than 40%). Soil textures with minimum infiltration rates of less than 0.52 in/hr are not suitable for usage of infiltration practices. These include soils with more than 30% clay content, which are susceptible to frost heaving and therefore structurally unstable; in addition to having a poor capacity to percolate runoff.

3. **Suitability of soils.** As seen above, the HSG and soil texture at the site will have a direct impact on the suitability of the site soils for application of an infiltration practice. Other considerations such as the soil bulk density and

degree of compaction of the site soils should also be considered. The NRCS Soil Survey publications provide tables of physical and engineering soil properties for each of the soil series at a particular site. These tables can be useful for completing an initial screening of the site soils to determine if an infiltration system should be selected as a viable alternative.

Table C5-S1- 1: Hydrologic soil properties classified by soil texture

Soil texture class	Hydrologic soil group	Effective water capacity (C_w) (in/in)	Minimum infiltration rate (*f*) (in/hr)	Effective porosity, θe (in^3/in^3)
Sand	A	0.35	8.27	0.025 (0.022-0.029)
Loamy sand	A	0.31	2.41	0.024 (0.020-0.029)
Sandy loam	B	0.25	1.02	0.025 (0.017-0.033)
Loam	B	0.19	0.52**	0.026 (0.020-0.033)
Silt loam	C	0.17	0.27	0.300 (0.024-0.035)
Sandy clay loam	C	0.14	0.17	0.020 (0.014-0.026)
Clay loam	D	0.14	0.09	0.019 (0.017-0.031)
Silty clay loam	D	0.11	0.06	0.026 (0.021-0.032)
Sandy clay	D	0.09	0.05	0.200 (0.013-0.027)
Silty clay	D	0.09	0.04	0.026 (0.020-0.031)
Clay	D	0.08	0.02	0.023 (0.016-0.031)

**Minimum rate: soils with lower rates should not be considered for infiltration BMPs

Source: Rawls et al., 1982

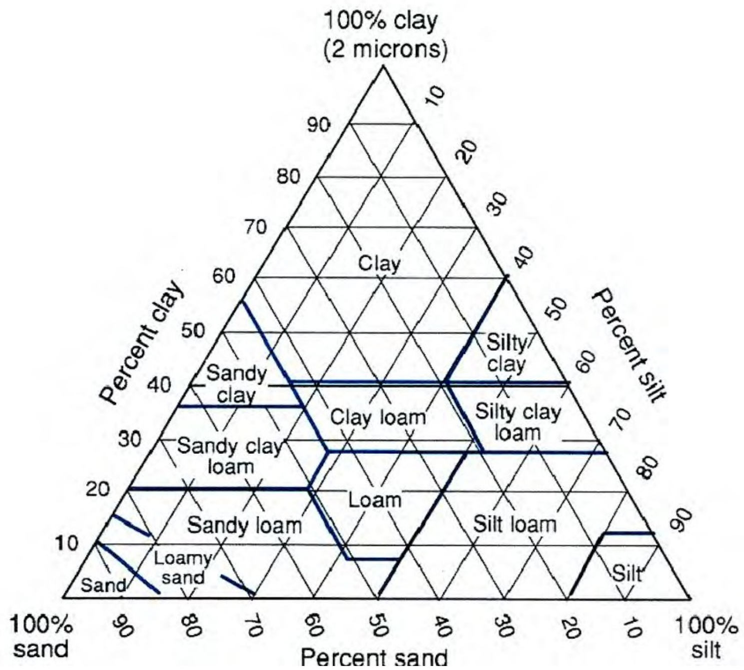


Figure C5-S1- 4: USDA Soil Textural Classification

F. Screening criteria for infiltration practices

1. Evaluation of the viability of a particular site includes:

   a. Determine soil type (consider NRCS Group A, B or C only) from mapping and consult USDA/NRCS soil survey tables to review other parameters such as the amount of silt and clay, presence of a restrictive layer or seasonal high water table, and estimated permeability. The soil should not have more than 30% clay or more than 40% clay and silt combined. Eliminate sites that are clearly unsuitable for infiltration. If the surface and underlying soils are NRCS Group D or the saturated infiltration rate is less than 0.52 in/hr, the site should not be used for infiltration.

   b. Groundwater separation should be at least 4 feet from the basin invert to the measured groundwater elevation. Seasonal high groundwater should be a minimum of 4 feet below the infiltration surface.

   c. Bedrock or impervious soils should be a minimum of 4 feet from the infiltrating surface (i.e. bottom of trench).

   d. Location should be the following distances away from structures:
      1) Buildings, slopes, and highway pavement: greater than 25 feet
      2) Wells and bridge structures: greater than 100 feet

   e. Sites that are constructed of fill and/or have a baseflow or slope greater than 15% should not be considered.

   f. Infiltration practices should not be placed in locations that cause water problems to downgrade properties. Infiltration facilities should be set back 25 feet (10 feet for dry wells) down-gradient from structures.

   g. Ensure that adequate head is available to operate flow splitter structures when the trench is operated as an offline structure. Hydraulic design should prevent ponding in the splitter structure or creation of backwater upstream of the splitter.

   h. For infiltration basins, at least three in-hole conductivity tests should be performed using USBR 7300-89 or Bouwer-Rice procedures (ASTM D5084-03 Standard Test Methods for Measurement of Hydraulic Conductivity of Saturated Porous Materials), the latter if groundwater is encountered within the boring; two tests at different locations within the proposed basin; and the third down-gradient by no more than 25 feet. The tests should measure permeability in the side slopes and the bed within a depth of 12 feet of the invert.

i. The minimum acceptable hydraulic conductivity as measured in any of the three required test holes is 0.5 in/hr. If any test hole shows less than the minimum value, the site should be disqualified from further consideration.

2. Should the initial site assessment described above not rule out infiltration as a BMP alternative, the point evaluation system described below can be used as the next level of site evaluation. The point system was developed by the Swedish Association of Water and Wastewater Works (1983) and was first recommended for use in the US by Urbonas and Stahre (1993). The protocol is based on evaluating various site conditions by assigning points for each category listed in Table C5-S1- 2. A site with fewer than 20 points is considered unsuitable. A site with more than 30 points is considered good. A site with 20-30 points is considered a suitable condition, with some occasional standing water on the infiltration surfaces possible.

3. These preliminary evaluation procedures should be coupled with a detailed site-specific engineering evaluation. This may include a standard series of soil borings at the proposed BMP locations to establish more definitive information on vertical soil textural/grain size classifications, as well as any restrictive layers in the soil profile. Direct in-situ measurement of soil infiltration rates can also be competed using a double-ring infiltrometer as described in the standard test method ASTM D3385-03, Standard Test Method for Infiltration Rate of Soils in Field Using Double-Ring Infiltrometer.

4. The recommended procedure for final site evaluation of soils for infiltration practices is provided in Chapter 5, section 7.

Table C5-S1- 2: Point system for the evaluation of potential infiltration sites

1 Ratio between tributary-connected impervious area (AIMP) and the infiltration area (AINF):	
• AINF > 2 AIMP	20 points
• AIMP ≤ AINF ≤ 2 AIMP	10 points
• 0.5 AIMP ≤ AINF ≤ AIMP	5 points
Urban catchments with pervious surfaces smaller than 0.5 AIMP should not be used for infiltration.	
2 Nature of surface soil layer:	
• Coarse soils with low ratio of organic material	7 points
• Normal humus soil	5 points
• Fine grained soils with high ratio of organic material	0 points
3 Underlying soils:	
• If the underlying soils are coarser than surface soils, assign the same number of points as for the surface layer under criterion #2.	
• If the underlying soils are finer-grained than the surface soils, use the following points:	
➢ Gravel, sand, or glacial till with gravel or sand	7 points
➢ Silty sand or loam	5 points
➢ Fine silt or clay	0 points
4 Slope (S) of the infiltration surface:	
• S < 7%	5 points
• 7% ≤ S ≤ 20%	3 points
• S > 20%	0 points
5 Vegetation cover:	
• Healthy, natural vegetation cover	5 points
• Lawn – well established	3 points
• Lawn – new	0 points
• No vegetation – bare ground	5 points
6 Degree of traffic on infiltration surface:	
• Little foot traffic	5 points
• Average foot traffic	3 points
• High foot traffic (i.e. playing/sports fields)	0 points

Source: Adapted from Urbonas and Stahre, 1993

G. Types of infiltration practices

Design methodologies are presented for three infiltration practices and two integrated (complementary) practices below:

- Infiltration trenches
- Infiltration basins
- Bioretention area (and rain gardens)
- Soil quality restoration
- Native landscaping

The design procedures are based on either intercepting the WQv from the area contributing runoff or using the truncated hydrograph method for control of the runoff from an area for either Cpv or Q_p. The design equations may be defined for either case of stormwater quality or quantity control because the volume of water (V_w) stored in the individual infiltration practice may be determined from the methods described in Chapter 2 and Chapter 3.

Infiltration trench and infiltration basin systems rely directly on the site soil conditions to infiltrate the design capture volume of stormwater. Infiltration trenches and basins can be used on single/multi-family residential sites of up to 10 acres and up to 5 acres for commercial sites. Bioretention BMPs use an additional prepared soil and vegetation layer on top of the infiltrating soil surface to provide an additional filtration process prior to infiltrating all or part of the filtered stormwater. Rain gardens are a smaller design variant of the class of BMPs called bioretention areas. Rain gardens are typically constructed on residential sites and use a shallow depression in the native soil profile supplemented with permeable upper soil, mixed with vegetation, to capture and treat the runoff. A rain garden is typically constructed without an aggregate subbase or subdrain system, and the captured runoff volume will be limited to that which can infiltrate into the local subsoil within 12-24 hours. The last two practices, soil quality restoration and native landscaping, are intended as complementary integrated practices that can be implemented to improve the infiltration capacity of compacted urban soils, and provide a vegetation system to maintain a healthy soil profile for infiltration.

An important consideration in the design and construction of infiltration systems is to understand that the primary cause of failure is clogging of the infiltrating soil interface. On development sites where construction will continue over an extended period of time, the final implementation of the infiltration BMP should be completed after the site is fully developed and the entire catchment area is stabilized for control of sediment from construction activity. All of the structural infiltration practices should be provided with an upstream pre-treatment BMP for removal of sediment (i.e. grass buffer strip, vegetated swale, sediment forebay, etc). While an infiltration trench or basin will provide removal of suspended solids, the primary functions will be removal of very small particulates and soluble pollutants in the soil profile, reduction of the volume of direct annual runoff to the storm sewer system and local streams, and increasing the volume of recharge to the local water table.

H. Design criteria for infiltration practices

1. **Infiltration conveyance criteria.** The design of all infiltration practices includes an analysis of the site runoff conveyance configuration to ensure that excess flow is discharged at non-erosive velocities.
   a. The overland flow path of surface runoff exceeding the capacity of the infiltration system is configured to preclude erosive concentrated flow. If computed flow velocities do not exceed the non-erosive threshold, overflow may be accommodated by natural topography. Critical erosive velocities for grass and soil are summarized in Chapter 9, section 2.
   b. Infiltration systems are designed to fully de-water the entire WQv within 48 hours after the storm event.
   c. If the infiltration practice is used to control the Cpv or Q_p, the truncated hydrograph method can be used to determine the required detention volume (see Chapter 3, section 7).
   d. If runoff is delivered by a storm drain pipe or along the main conveyance system, the infiltration practice should be designed as an offline practice (see Chapter 4 for an example of an offline infiltration practice).
   e. Stormwater outfalls with capacity for the overflow associated with the 10-year design storm event are included and configured to prevent non-erosive velocities on the downslope.

2. **Infiltration pre-treatment criteria.**
   a. **Pre-treatment techniques to prevent clogging.** The purpose of pre-treatment is to protect the long-term integrity of the infiltration rate. The following techniques, at least three for infiltration trenches and two for infiltration basins, are installed in every infiltration practice:
      - Grass channel (see Chapter 9, section 2 for design requirements and example computation)
      - Grass filter strip (minimum 20 feet and only if sheet flow is established and maintained); see Chapter 9, section 4 for design requirements and example computation
      - Bottom sand layer
      - Upper sand layer (6-inch minimum) with filter fabric at the sand/gravel interface
      - Use of washed bank run gravel as aggregate

   b. **Pre-treatment volume.** A minimum of 25% of the WQv is pre-treated prior to entry to the infiltration practice. If the infiltration rate for the underlying soils is greater than 2 inches per hour, 50% of the WQv is pre-treated prior to entry into an infiltration facility. This can be provided by a sedimentation basin, stilling basin, sump pit, or other acceptable measures. Exit velocities from pre-treatment should be non-erosive during the two-year design storm.

3. **Infiltration treatment criteria.**
    a. The infiltration practice is designed to exfiltrate the entire WQv less the pre-treatment volume through the floor of each practice, using the design methods outlined in the subsequent design procedures for each practice.
    b. Infiltration practices are best used in conjunction with other BMPs, and downstream detention is often still needed to meet the Cpv and Q_p sizing criteria.
    c. The construction sequence and specifications for each infiltration practice is outlined in the SUDAS Specifications manual. Experience has shown that the longevity of infiltration practices is strongly influenced by the care taken during construction.
    d. A porosity value, n ($n=V_v/V_t$), of 0.40 is used in the design of stone reservoirs for infiltration practices.

4. **Infiltration landscaping criteria.**
    a. A dense and vigorous vegetative cover is established over the contributing pervious drainage areas before runoff can be accepted into the facility.
    b. Infiltration trenches are not constructed until all of the contributing drainage area has been completely stabilized.

5. **Infiltration maintenance criteria.**
    a. Infiltration practices may not serve as a sediment control device during the site construction phase. In addition, the erosion and sediment control plan for the site must clearly indicate how sediment will be prevented from entering the infiltration site.
    b. An observation well for monitoring the water level is installed in the infiltration practice, consisting of an anchored six-inch diameter perforated PVC pipe with a lockable cap.
    c. Consideration should be given in the infiltration design to include a dewatering method in the event of failure. This can be done with subdrain pipe systems to provide drawdown capability.
    d. Direct access is provided to all infiltration practices for maintenance and rehabilitation.

Source: California Stormwater Manual

POLLUTANT REMOVAL Low = <30% Medium = 30-65% High = 65-100%			
	Low	Med	High
Suspended Solids			✓
Nitrogen		✓	✓
Phosphorous		✓	
Metals			✓
Bacteriological			✓
Hydrocarbons		✓	

Description: An infiltration trench is a long, narrow, rock-filled trench with no outlet that receives stormwater runoff. Runoff is stored in the void space between the stone aggregate and infiltrates through the bottom and into the soil matrix. Infiltration trenches can range from 3-12 feet deep, are backfilled with stone aggregate, and are lined with filter fabric. Underground trenches receive runoff through pipes or channels, whereas surface trenches collect sheet flow from the drainage area. Trenches should be designed to drain completely within 6-48 hours. Infiltration trenches perform well for removal of fine sediment and associated pollutants. Pre-treatment using buffer strips, swales, or detention basins is important for limiting amounts of coarse sediment entering the trench, which can clog and render the trench ineffective.

Typical uses: Residential subdivisions, high-density residential, ultra-urban areas, and parking lots.

Advantages:

- Appropriate for small sites with porous soils
- Infiltration trenches reduce runoff volume and filter pollutants
- Provide stream baseflow and recharge groundwater.
- As an underground BMP, trenches are unobtrusive and have little impact on site aesthetics

Limitations:

- Use should be restricted to small drainage areas – generally less than 5 acres
- Suitable for NRCS HSG-A/B soils; limited application in HSG-C soils; not recommended in HSG-D soils. Do not use with soil infiltration rates <0.5 inches/hour
- Seasonal high water table should be 4 feet below bottom of trench
- Susceptible to clogging by sediment – use upstream BMPs for sediment removal
- Restricted in karst areas
- Placement under paved surfaces or in industrial or commercial settings not recommended

Maintenance requirements:

- Remove sediment accumulation to ensure proper functioning
- Inspect for clogging – install an integrated observation well/piezometer to check water level
- Remove sediment from pre-treatment areas

A. Description

By diverting runoff into the soil, an infiltration trench not only treats the water quality volume, but also helps to preserve the natural water balance on a site and can recharge groundwater and preserve baseflow. Due to this fact, infiltration systems are limited to areas with highly porous soils where the water table and/or bedrock are located well below the bottom of the trench. In addition, infiltration trenches must be carefully sited to avoid the potential of groundwater contamination.

Infiltration trenches are not intended to trap sediment. Due to their high potential for failure, these facilities must only be considered for sites where upstream sediment control can be ensured. Pre-treatment using buffer strips, swales, or detention basins is important for limiting amounts of coarse sediment from entering the trench because it can clog the trench and render it ineffective. An example infiltration trench system is shown in Figure C5-S2- 1.

B. Stormwater management suitability

Infiltration trenches are designed primarily for reduction in stormwater runoff volume, but when integrated with other BMPs, they can achieve significant water quality improvement. Runoff volume control can be achieved for the water quality volume for smaller storm events up to the limits of the local infiltration capacity of the local soils. The runoff volume gradually infiltrates through the bottom and sides of the trench and into the subsoil, eventually reaching the water table. By diverting runoff into the soil, an infiltration trench not only treats the water quality volume, but also helps to preserve the natural water balance on a site, recharge groundwater, and preserve baseflow.

An infiltration trench may also be designed to capture and infiltrate the entire channel protection volume, Cpv, in either an offline or online configuration. For larger sites, or where only the WQv is diverted to the trench, another structural control must be used to provide Cpv extended detention. Infiltration trenches must be used in conjunction with another best management practice to provide overbank and extreme flood protection, if required.

C. Pollutant removal capabilities

Infiltration trenches can remove a wide variety of pollutants from stormwater through sorption (the action of soaking up or attracting substances), precipitation, filtering, and bacterial and chemical degradation. Pre-treatment areas up-gradient of the infiltration site are provided to remove a larger portion of the TSS and overall sediment load. Examples of some pre-treatment areas include grit chambers, water quality inlets, sediment traps, swales, and vegetated filter strips (SEWRPC 1991; Harrington 1989).

When used with pre-treatment areas, infiltration trenches can remove up to 80 percent of sediments, metals, coliform bacteria, and organic matter; and up to 60% of phosphorus and nitrogen (Schueler 1992). Biochemical oxygen demand (BOD) removal is estimated to be between 70-80%. Lower removal rates for nitrate, chlorides, and soluble metals should be expected. Undersized or poorly-designed infiltration trenches can reduce TSS removal performance. An infiltration trench is presumed to be able to remove 80% of the TSS load in typical urban post- development runoff when sized, designed, constructed, and maintained in accordance with the recommended specifications. 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.

For additional information on monitoring BMP performance, see ASCE/EPA “Urban Stormwater BMP Performance Monitoring: A Guidance Manual for Meeting the National Stormwater BMP Database Requirements.”

Using washed aggregate and adding organic matter and loam to the subsoil may improve pollutant removal efficiencies. The addition of organic material and loam to the trench subsoil will enhance metals and nutrient removal through adsorption.

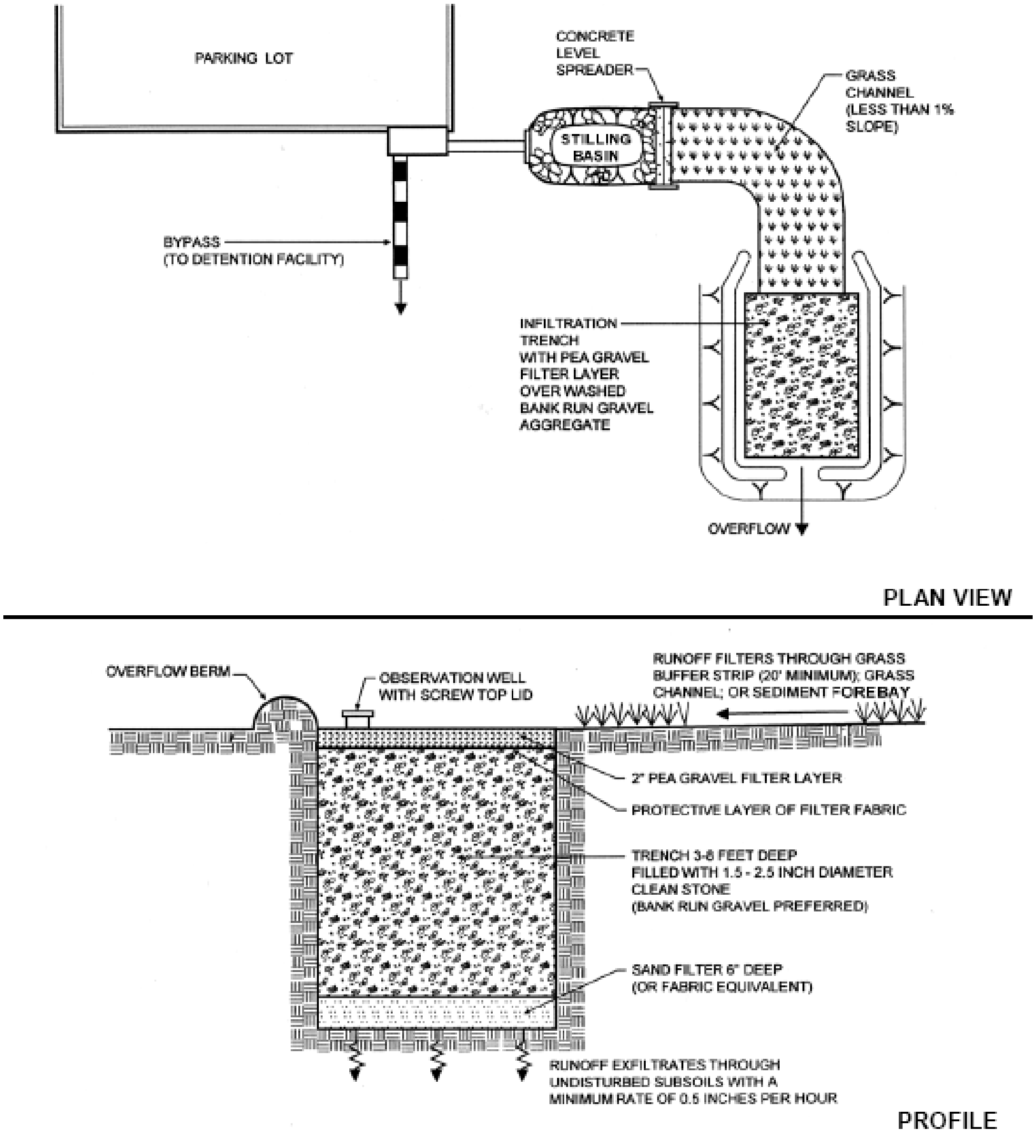


Figure C5-S2- 1: Example of an offline infiltration trench configuration
Source: Center for Watershed Protection

D. Application and feasibility

Infiltration trenches are generally suited for medium- to high-density residential, commercial, and institutional developments where the subsoil is sufficiently permeable to provide a reasonable infiltration rate and the water table is low enough to prevent groundwater contamination. They are applicable primarily for impervious areas where there are not high levels of fine particulates (clay/silt soils) in the runoff, and should only be considered for sites where the sediment load is relatively low.

Infiltration trenches can be used either to capture sheet flow from a drainage area or to function as an offline device. Due to the relatively narrow shape, infiltration trenches can be adapted to many different types of sites and can be utilized in retrofit situations. Unlike some other structural stormwater controls, they can easily fit into the margin, perimeter, or other unused areas of developed sites. Infiltration trenches capture and treat small amounts of runoff but do not control peak hydraulic flows. Infiltration trenches should be used in conjunction with another best management practice to provide both water quality control and peak flow control (Harrington 1989). Peak flow control is usually achieved with a slow release of the stormwater management volume through an orifice in the storage facility. As a result, the water quality volume will equal the stormwater detention area below the orifice, and must infiltrate to exit.

The applicability of infiltration trenches depends on native soils, slope, depth to water tables, depth to bedrock, size of drainage area, and proximity to wells, surface waters, and foundations. Trenches are generally suitable to sites with gentle slopes, permeable soils, deep bedrock, and deep groundwater. Excessive slope of the drainage area, fine-particle soil types, and proximate location of the water table and bedrock may prevent the use of infiltration trenches.

1. **General feasibility.**
   a. Suitable for Residential Subdivision Usage – yes
   b. Suitable for High Density/Ultra Urban Areas – yes
   c. Regional Stormwater Control – no

2. **Physical feasibility – physical constraints at project site.**
   a. **Drainage area.** 5 acres maximum.
   b. **Space required.** Will vary depending on the depth of the facility.
   c. **Site slope.** No more than 6% (for pre-construction facility footprint).
   d. **Minimum head.** Elevation difference needed at a site from the inflow to the outflow: 1 foot.
   e. **Minimum depth to water table.** Four feet recommended between the bottom of the infiltration trench and the elevation of the seasonally high water table.
   f. **Soils.** Infiltration rate greater than 0.5 in/hr required (typically HSG-A and B soils).
   g. **Other constraints/considerations.** Aquifer protection: no hotspot runoff allowed; meet setback requirements in design criteria.

E. Planning and design criteria

1. **Evaluation of the site.** Per the general criteria listed in C5-S1.

2. **Slope and drainage area.** The drainage area slope determines the velocity of the runoff and influences the amount of pollutants entrained in the runoff. Infiltration trenches work best when the up-gradient drainage area slope is less than 5% (Schueler, 1987). The down-gradient slope should be no greater than 15% to minimize slope failure and seepage. The slope of the surrounding area should be such that the runoff is evenly distributed as sheet flow as it enters the trench. Runoff can be captured by depressing the trench surface or by placing a berm at the down-gradient side of the trench. In general, infiltration trenches are suitable for drainage areas up to 5 acres. Supplemental BMPs should always be carefully considered. The drainage area must be fully developed and stabilized with vegetation before constructing an infiltration trench. High sediment loads from unstabilized areas will quickly clog the infiltration trench.

3. **Depth to water table and bedrock.** Land availability, the depth to bedrock, and the depth to the water table will determine whether the infiltration trench is located underground or at grade. Feasible sites should have a minimum of 4 feet to bedrock in order to reduce excavation costs. There should also be at least 4 feet below the trench to the water table to prevent potential groundwater problems.

4. **Minimum setbacks.** Stormwater easements may be necessary to accommodate setbacks. Recommended setbacks are as follows:
   a. Property line: 10 feet
   b. Building foundation: 25 feet
   c. Private well: 100 feet
   d. Public water supply well: 1,000 feet
   e. Septic system tank/leach field: 100 feet
   f. Surface waters: 100 feet

5. **Location and siting:**
   a. Infiltration trenches can be used to capture sheet flow or function as an offline device. They are suitable for medium- to high-density residential areas. Because of their narrow shape, they can be added to many different sites and retrofit situations. They easily fit into the margin, perimeter, or other unused areas of developed sites. Infiltration trenches are not suitable for sites that use or store chemicals or hazardous materials, unless diversion structures prevent hazards from entering the trench. The potential for spills can be minimized by

aggressive pollution prevention measures. Many municipalities and industries have developed comprehensive spill prevention control and countermeasure plans. These plans should be modified to include the infiltration trench and the contributing drainage area.

b. When used in an offline configuration, the WQv is diverted to the infiltration trench through the use of a flow splitter. Stormwater flows greater than the WQv are diverted to other controls or downstream, using a diversion structure or flow splitter.

c. To reduce the potential for costly maintenance and/or system reconstruction, it is strongly recommended that the trench be located in an open or lawn area, with the top of the structure as close to the ground surface as possible. Infiltration trenches should not be located beneath paved surfaces, such as parking lots.

d. The underlying soils must meet the soils screening criteria with an infiltration rate, *f*, of 0.5 in/hr or greater, as initially determined from NRCS soil textural classification; and subsequently confirmed by field geotechnical tests. The minimum geotechnical testing is one test hole per 5,000 square feet, with a minimum of two borings per facility (taken within the proposed limits of the facility). Infiltration trenches cannot be used in fill soils.

e. Infiltration trenches should have a contributing drainage area of 5 acres or less.

f. Infiltration trenches are designed for intermittent flow and must be allowed to drain and allow re-aeration of the surrounding soil between rainfall events. They must not be used on sites with a continuous flow from groundwater, sump pumps, or other sources. Trenches should be designed to drain completely within 6-72 hours. A shorter drain time of 48 hours is often used as a factor of safety in the design. A minimum drainage time of 6 hours will ensure satisfactory pollutant removal. The maximum drainage time is dependent on the precipitation zone. In Iowa, the average time between storm events is approximately 72 hours. Therefore, the trench should be designed to drain completely within a maximum of 72 hours.

g. The site assessment approach for stormwater infiltration sites is very similar to the site assessment used for the design of onsite wastewater treatment systems. In a soil/water infiltration system (SWIS) for septic tank effluent, the same concerns are considered as with seasonal high groundwater, depth to water table, and soil permeability. The typical loading rates for septic tank effluent in B soils would be on the order of 0.5-1.0 gal/day/ft^2 of infiltration surface. These are long-term loading rates and are based on the assumption that a bio-mat will eventually form at the soil/water interface.

6. **Cold weather considerations.** Climate can limit infiltration trench use. Winter sanding can clog an infiltration trench, and winter salting can increase the potential for chloride contamination of groundwater. Additionally, the trench surface may freeze, thereby preventing the runoff from entering the trench and allowing the untreated runoff to enter surface water. However, recent studies indicate that if properly designed and maintained, infiltration trenches can operate effectively in colder climates. By keeping the trench surface free of compacted snow and ice, and by ensuring that part of the trench is constructed below the frost line, the performance of the infiltration trench during cold weather will be greatly improved.

F. Physical specifications, geometry, and volume

1. A well-designed infiltration trench consists of:
   a. Excavated shallow trench backfilled with sand, coarse stone, and pea gravel; and lined with a filter fabric.
   b. Appropriate pre-treatment measures.
   c. One or more observation wells to show how quickly the trench dewaters or to determine if the device is clogged.
   d. A plan view and profile schematic for the design of an offline infiltration trench is shown in Figure C5-S2- 1. A schematic for an online infiltration facility is shown in Figure C5-S2- 2.

2. Physical specifications of an infiltration trench include:
   a. The required trench storage volume is equal to the WQv. For smaller sites, an infiltration trench can be designed with a larger storage volume to include the Cpv.

b. A trench must be designed to fully dewater the entire WQv within 24-48 hours after a rainfall event. The slowest infiltration rate obtained from tests performed at the site should be used in the design calculations.
c. Trench depths should be between 3 and 8 feet, to provide for easier maintenance. The width of a trench is usually less than 25 feet.
d. Broader, shallow trenches reduce the risk of clogging by spreading the flow over a larger area for infiltration.
e. The surface area required is calculated based on the trench depth, soil infiltration rate, aggregate void space, and fill time (assume a fill time of 2 hours for most designs).
f. The bottom slope of a trench should be flat across its length and width to evenly distribute flows, encourage uniform infiltration through the bottom, and reduce the risk of clogging.

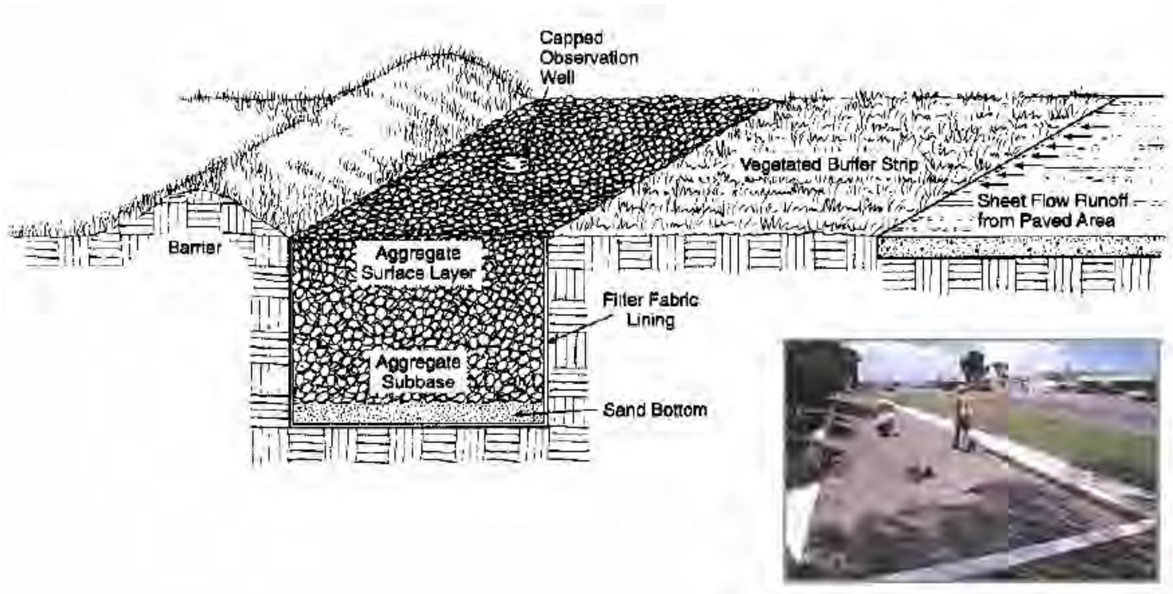


Figure C5-S2- 2: Example of an online infiltration trench configuration
Source: Georgia Stormwater Manual, 2000

3. Components of an infiltration trench (See Figure C5-S2- 3):
   a. **Aggregate.** The basic infiltration trench uses stone aggregate in the top of the trench to promote filtration. The stone aggregate is normally 1-3 inches in diameter, which provides a void space of ~35 percent (SEWRPC 1991; Harrington 1989; Schueler 1987). The stone aggregate should be washed to remove dirt and fines before placement in the trench. The aggregate should be non-crushed limestone or a river-run washed stone (often referred to as septic rock). A 4-6 inch layer of clean, washed, ASTM C33 medium aggregate concrete sand is placed on the bottom of the trench to encourage drainage and prevent soil compaction when the stone aggregate is added. The design can be modified by substituting pea gravel for stone aggregate in the top foot of the trench. The pea gravel should be #8 to ⅜-inch. The pea gravel improves sediment filtering and maximizes the pollutant removal in the top of the trench. When the modified trenches become clogged, they can generally be restored to full performance by removing and replacing only the pea gravel layer, without replacing the lower stone aggregate layers. Infiltration trenches can also be modified by adding a layer of organic material (peat) or loam to the trench subsoil. This modification appears to enhance the removal of metals and nutrients through adsorption.

   b. **Sheet flow.** The trench surface may consist of stone or vegetation with inlets to evenly distribute the runoff entering the trench (SEWRPC 1991; Harrington 1989). A level spreader can be installed to create sheet flow (Harrington 1989).

   c. **Filter fabric.** The sides and bottom of the infiltration trench should be lined with filter fabric. The fabric should be placed around the walls and bottom of the trench, and 1 foot below the trench surface. The filter fabric should overlap each side of the trench in order to cover the top of the stone aggregate layer. The filter fabric prevents sediment in the runoff and soil particles from the sides of the trench from clogging the aggregate. Filter fabric placed 1 foot below the trench surface will maximize pollutant removal within the top layer of the trench and decrease the pollutant loading to the trench bottom, reducing the required frequency of maintenance.

d. **Observation well.** (See example schematics). An observation well allows monitoring of drainage. The observation well can be 4 to 6-inch diameter PVC pipe with a lockable cap. The well can either be 6 inches above-ground or flush with the ground, depending on the trench surface. It is anchored to a footplate at the bottom of the trench, and should be located near the longitudinal center of the infiltration trench. A visible floating marker should be provided to indicate the water level. The pipe should have a plastic collar with ribs to prevent rotation when removing the cap. The screw-top lid should be a cleanout with a locking mechanism or special bolt to discourage vandalism. The depth to the invert should be marked on the lid.

e. **Filter strip.** A vegetated buffer strip, 20-25 feet wide, is established adjacent to the infiltration trench to capture large sediment particles in the runoff. The buffer strip is installed immediately after trench construction using sod instead of hydroseeding (Schueler 1987). The buffer strip should be graded with a slope between 0.5 and 15% so that runoff enters the trench as sheet flow.

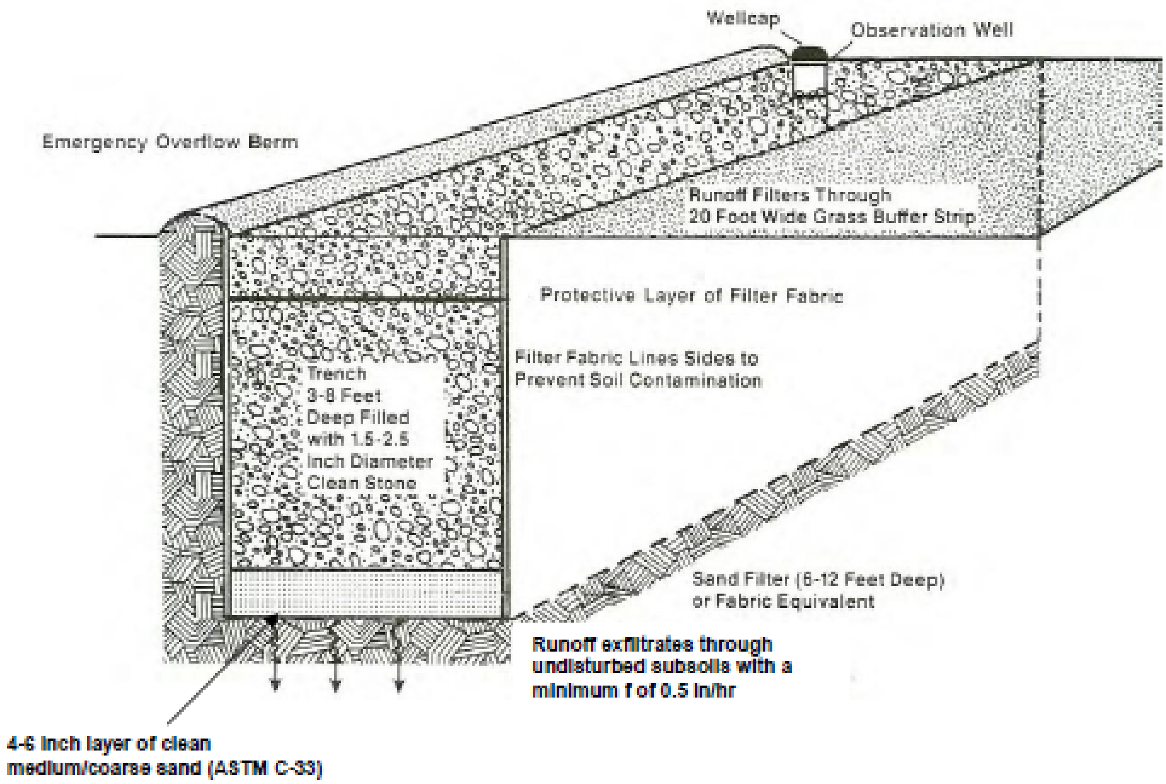


Figure C5-S2- 3: Components and configuration of an infiltration trench
Source: Adapted from Schueler, 1987

G. Design of infiltration trenches

The design of an infiltration trench is based on the textural class and nominal infiltration rate of the soils underlying the trench such that a feasible design is possible. The design of an infiltration trench is also based on the maximum allowable depth of the trench (d_{max} - ft). The maximum allowable depth should meet the following criteria:

Equation C5-S2- 1

$$d\_max = ((fT\_s)/n)/12$$

Where:
f = final infiltration rate of the trench area (in/hr)

T_s = maximum allowable storage time (hr)
n = porosity, volume voids/total volume (V_v/V_t) of the aggregate reservoir.

A nominal value for n of 0.32-0.35 is typical. This can be adjusted based on specific measurement for the aggregate specified. The maximum allowable storage time should be no greater than 72 hours. The maximum allowable depth for a site may also be limited by the depth to the water table.

The infiltration trench is sized to accept the design volume that enters the trench (V_w) plus the volume of rain that falls on the surface of the trench (PA_t) minus the exfiltration volume (fTA_t) out of the bottom of the trench. Based on NRCS hydrograph analysis, the effective filling time for most infiltration trenches (T) will generally be less than two hours. The volume of water that must be stored in the trench (V_s) is defined as:

Equation C5-S2- 2

$$V_s = V_w + \left(\frac{P}{12}\right)(A_t) - \left(\frac{f}{12}\right)TA_t$$

Where:
V_w = water quality volume (WQv) or total runoff volume to be infiltrated (ft^3)
P = design rainfall event (in)
A_t = trench surface area (ft^2)
f = infiltration rate (in/hr)
T = fill time (hr)

For most design storm events, the volume of water due to rainfall on the surface area of the trench (PA_t) is small when compared to the design volume (V_w) of the trench, and may be ignored with little loss in accuracy to the final design. The volume of rainfall and runoff entering the trench can be defined in terms of trench geometry. The gross volume of the trench (V_t) is equal to the ratio of the volume of water that must be stored (V_w) to the porosity (n) of the stone reservoir in the trench.

V_s is also equal to the product of the depth (d_t-ft), the surface area (A_t-ft^2), and the porosity (n).

Equation C5-S2- 3

$$V_s = d_t \times A_t \times n$$

Combining Equation C5-S2- 1 and Equation C5-S2- 2 provides the following expression:

$$d_t \times A_t \times n = V_w + \left(\frac{P}{12}\right)(A_t) - \left(\frac{f}{12}\right)TA_t$$

Assuming the volume of water falling directly onto the trench area is negligible, then:

Equation C5-S2- 4

$$d_t \times A_t \times n = V_w - \left(\frac{f}{12}\right)TA_t$$

Because both dimensions, A_t and d_t, of the trench are unknown, the equation may be rearranged to determine the area of the trench (At) if the value of d_t is determined, based on either the location of the water table, or the maximum allowable depth of the trench (d_{max}):

Equation C5-S2- 5

$$A_t = \frac{V_w}{nd_t + \frac{fT}{12}}$$

H. Design procedures

Design plans should include a geotechnical evaluation that determines the feasibility of using an infiltration trench at the site (See Chapter 5, section 1).

1. **Step 1**. Compute runoff control volumes. Calculate the WQv, Cpv, Q_p, and the 100-year Q_f. See Chapter 3 for calculations.

2. **Step 2.** Determine if the development site and conditions are appropriate for the use of an infiltration trench. Confirm any local design criteria and check with local agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply. Consider any special site-specific design conditions, including:
   - Soil:
     - Soil type (USDA classifications)
     - Percent clay
     - Permeability
     - Assessment procedures
   - Depth to water table
   - Slope
   - Drainage area

3. **Step 3.** Compute the peak discharge rate for the water quality volume event. The peak rate of discharge for water quality design is needed for sizing of the offline diversion structure and piping.

   a. Using WQv (or total volume to be infiltrated), compute CN.
   b. Compute time of concentration using WinTR-55 method.
   c. Determine appropriate unit peak discharge from time of concentration.
   d. Compute Qwq from unit peak discharge, drainage area, and WQv.

4. **Step 4.** Size the flow diversion structure, if needed. A flow regulator (or flow splitter diversion structure) should be supplied to divert the WQv to the infiltration trench. Size low-flow orifice, weir, or other device to pass the Qwq.

5. **Step 5.** Size the infiltration trench:

   a. Determine the trench volume by assuming the WQv will fill the void space based on the computed porosity of the stone aggregate backfill (normally about 35%).

   b. A site-specific trench depth is calculated based on the soil infiltration rate, aggregate void space, and the trench storage time as described above (Harrington 1989). Compute the maximum allowable trench depth (d_{max}) from Equation C5-S2- 1. Select the trench design depth (d_t) based on the depth that is the required depth above the seasonal groundwater table, or a depth less than or equal to d_{max}, whichever results in the smaller depth. Trench depths are usually between 3-12 feet (SEWRPC 1991; Harrington 1989). However, a depth of 8 feet is most commonly used (Schueler 1987).

   c. Compute the trench surface area (A_t) for the particular soil type using Equation C5-S2- 4:

$$A_t = \frac{WQv}{nd_t + \frac{fT}{12}}$$

Where:
A_t = trench surface area, ft^2
WQv = water quality volume (or total volume to be infiltrated), ft^3
f = infiltration rate, in/hr
T = drain time (maximum time to dewater the entire WQv), hours

d. A minimum drainage time of 6 hours should be provided to ensure satisfactory pollutant removal in the infiltration trench (Schueler 1987). Although trenches are designed to provide temporary storage of stormwater, the trench should drain prior to the next storm event. For Iowa, the mean time between storm events is about 72 hours. Using a shorter drain time of 48 hours would provide a more conservative design.

e. In the event that the sidewalls of the trench must be sloped for stability during construction, the surface dimensions of the trench should be based on the following equation:

Equation C5-S2- 6

$$A_t = (L - Zd_t)(W - Zd_t)$$

Where L and W are the top length and width, and Z:1 is the trench side-slope ratio. The design procedure would begin by selecting a top width (W) that is greater than 2×*Zdt* for a specified slope (Z). The side slope ratio value will depend on the soil type and the depth of the trench. The top length (L) may then be determined as:

Equation C5-S2- 7

$$L = Zd_t + \frac{A_t}{W - Zd_t}$$

6. **Step 6**. Determine pre-treatment volume and design pre-treatment measures. Size pre-treatment facility to treat 25% of the water quality protection volume for offline configurations.

7. **Step 7.** Design spillway(s): Adequate stormwater outfalls should be provided for the overflow exceeding the capacity of the trench, ensuring non-erosive velocities on the downslope.

I. Inspection and maintenance requirements

Infiltration trenches, as with all BMPs, must have routine inspection and maintenance designed into the life performance of the facility. The principal maintenance objectives are to prevent clogging and groundwater contamination. Maintenance and inspection plans should be identified prior to establishment. Infiltration trenches and any pre-treatment BMPs should be inspected after large storm events to remove any accumulated debris or material. A more thorough inspection of the trench should be conducted annually. A summary of inspection and maintenance activities is provided in Table C5-S2- 1.

A record should be maintained of the dewatering time of an infiltration trench to determine if maintenance is needed. (Ponded water lasting more than 24 hours usually indicates that the trench is clogged). When vegetated buffer strips are used, they should be inspected for erosion or other damage after each major storm event. Trees and other large vegetation adjacent to the trench should also be removed to prevent damage to the trench.

Maintenance responsibility for an infiltration trench should be assigned to a responsible jurisdiction or authority through a legally binding and enforceable maintenance agreement completed as a condition of the site plan approval.

Table C5-S2- 1: Typical maintenance activities for infiltration trenches

Activity	Schedule
• Ensure the contributing drainage area, facility, and inlets are clear of debris. • Ensure that the contributing area is stabilized. • Remove sediment and oil/grease from pre-treatment devices, as well as overflow structures. • Mow grass filter strips as necessary. Remove grass clippings.	Monthly
• Check observation wells following three days of dry weather. Failure to percolate within this time period indicates clogging. • Inspect pre-treatment devices and diversion structures for sediment buildup and structural damage. • Remove trees that start to grow in the vicinity of the trench.	Semi-annual
• Replace pea gravel/topsoil and top surface filter fabric (when clogged).	As needed
• Perform total rehabilitation of the trench to maintain design storage capacity. • Excavate trench walls to expose clean soil.	Upon failure

Source: US EPA, 1999

J. Example schematics

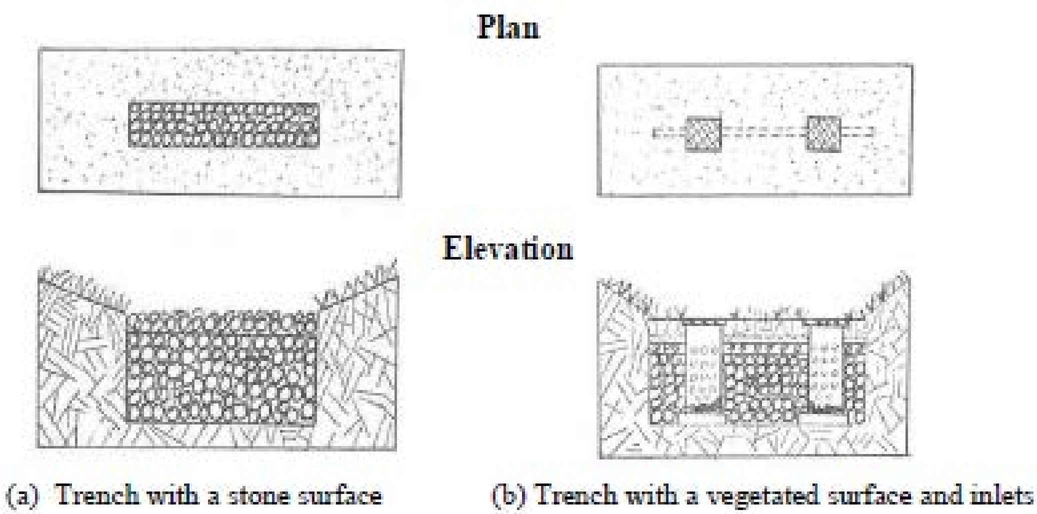


Figure C5-S2- 4: Types of infiltration trenches
Source: Schueler, 1987

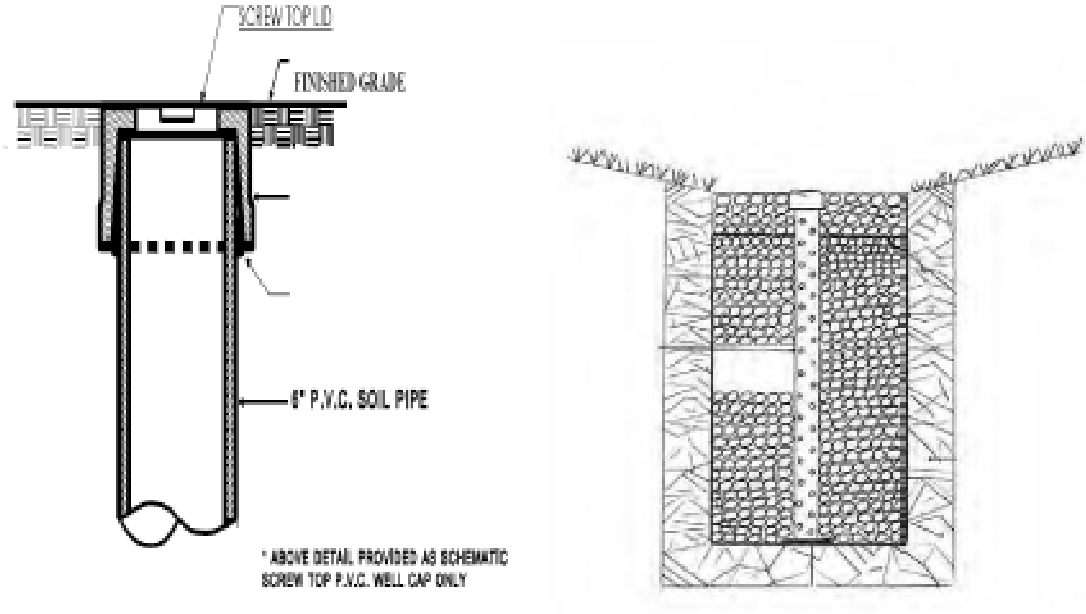


Figure C5-S2- 5: Example observation well and access lid
Source: Adapted from Maryland WRA

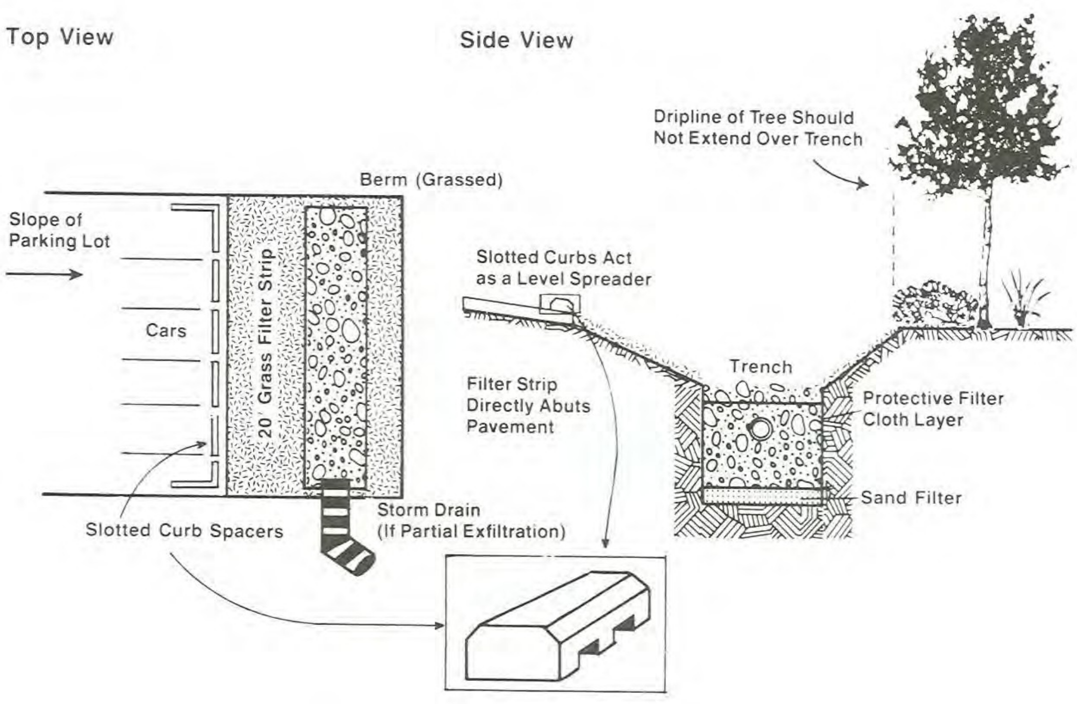


Figure C5-S2- 6: Commercial trench design – parking lot
Source: MWCOG, 1987

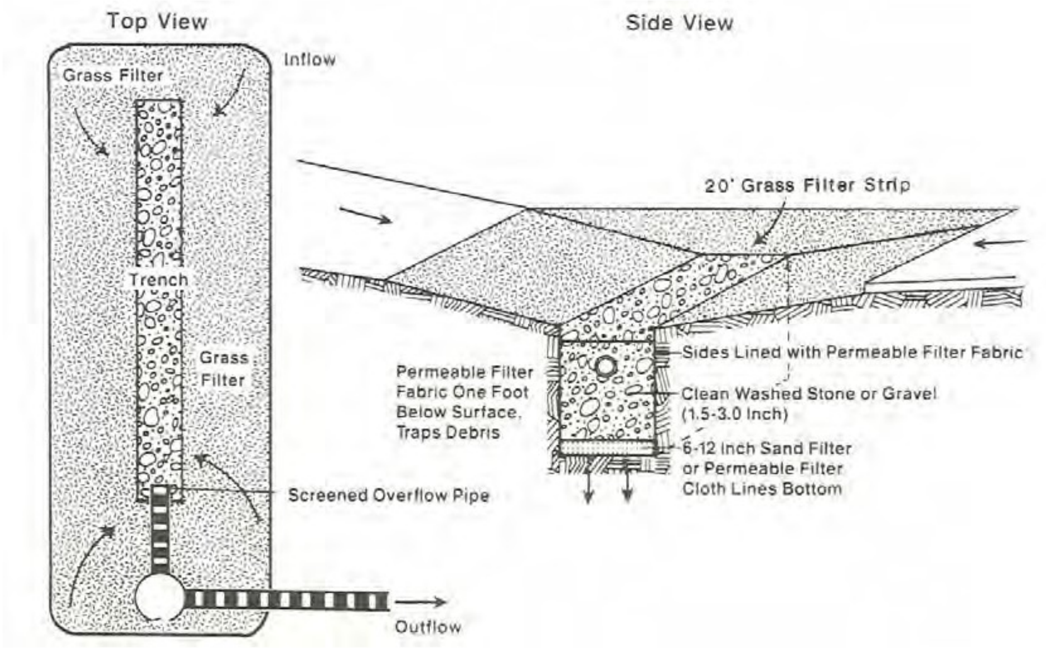


Figure C5-S2- 7: Median strip design – multiple lane street/highway
Source: MWCOG, 1987

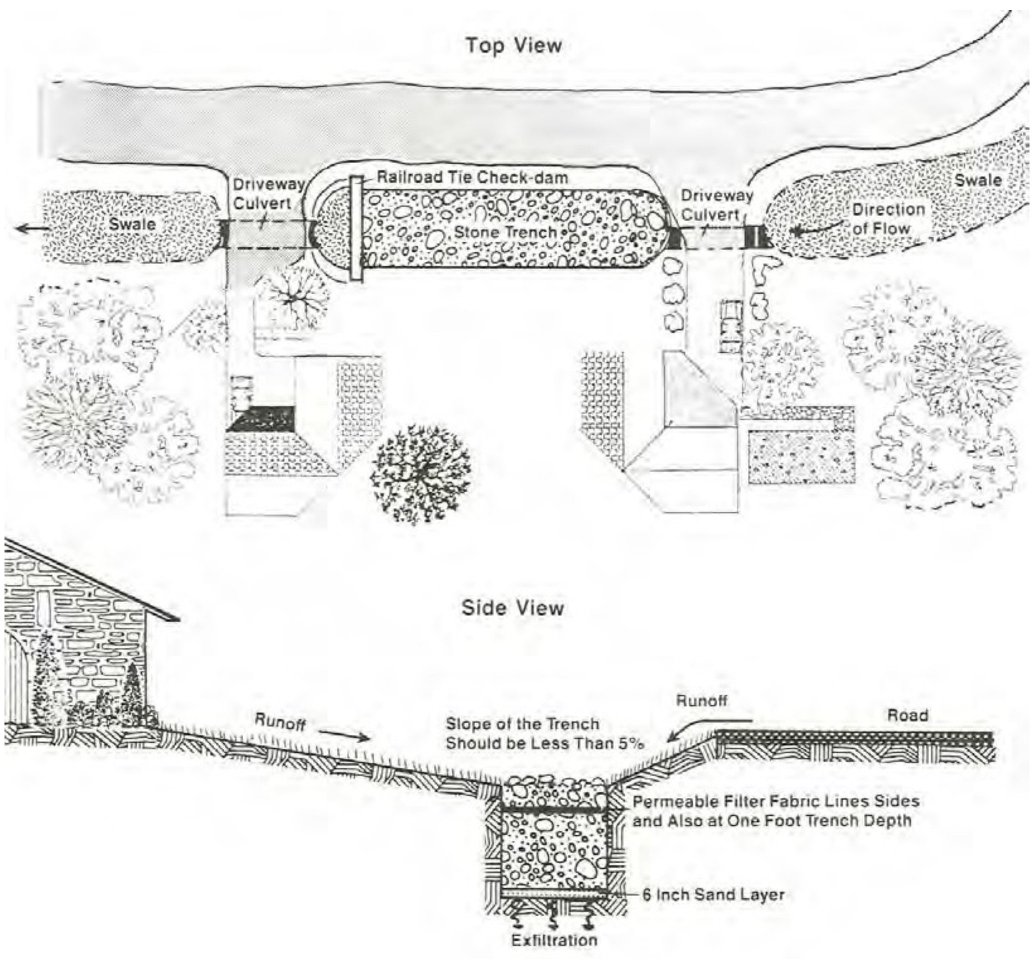


Figure C5-S2- 8: Swale/trench design (residential application)
Source: MWCOG, 1987

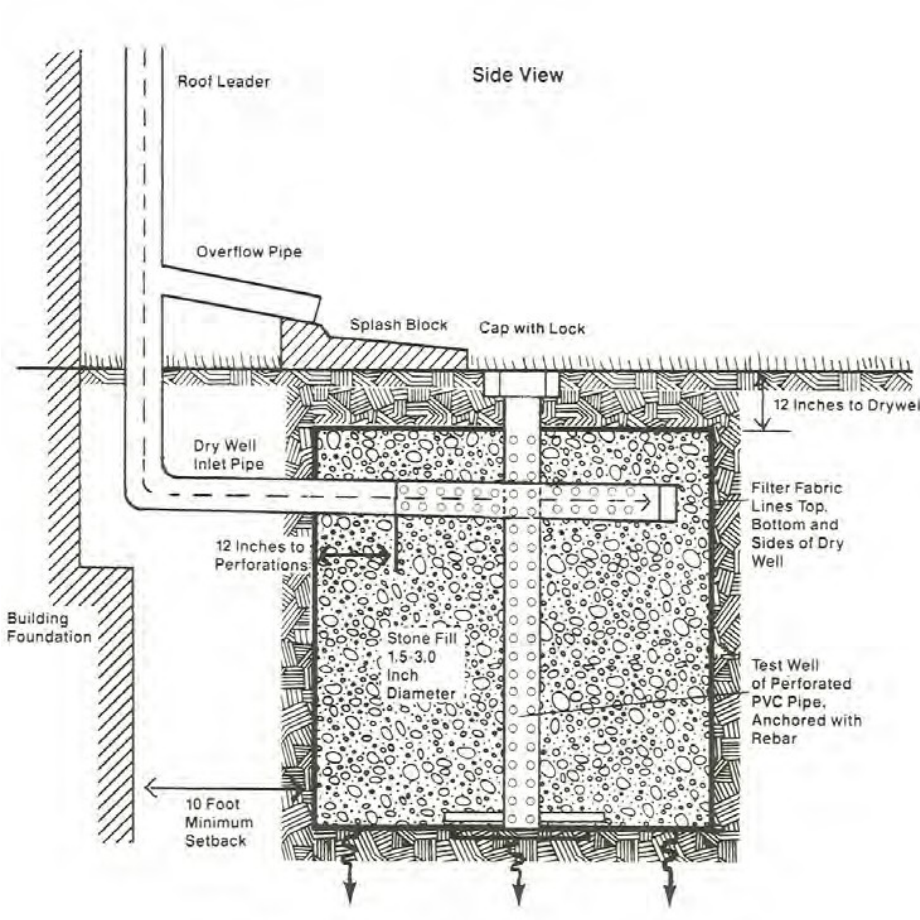


Figure C5-S2- 9: Infiltration: dry well design
Source: MWCOG, 1987

K. Design example

Infiltration Trench

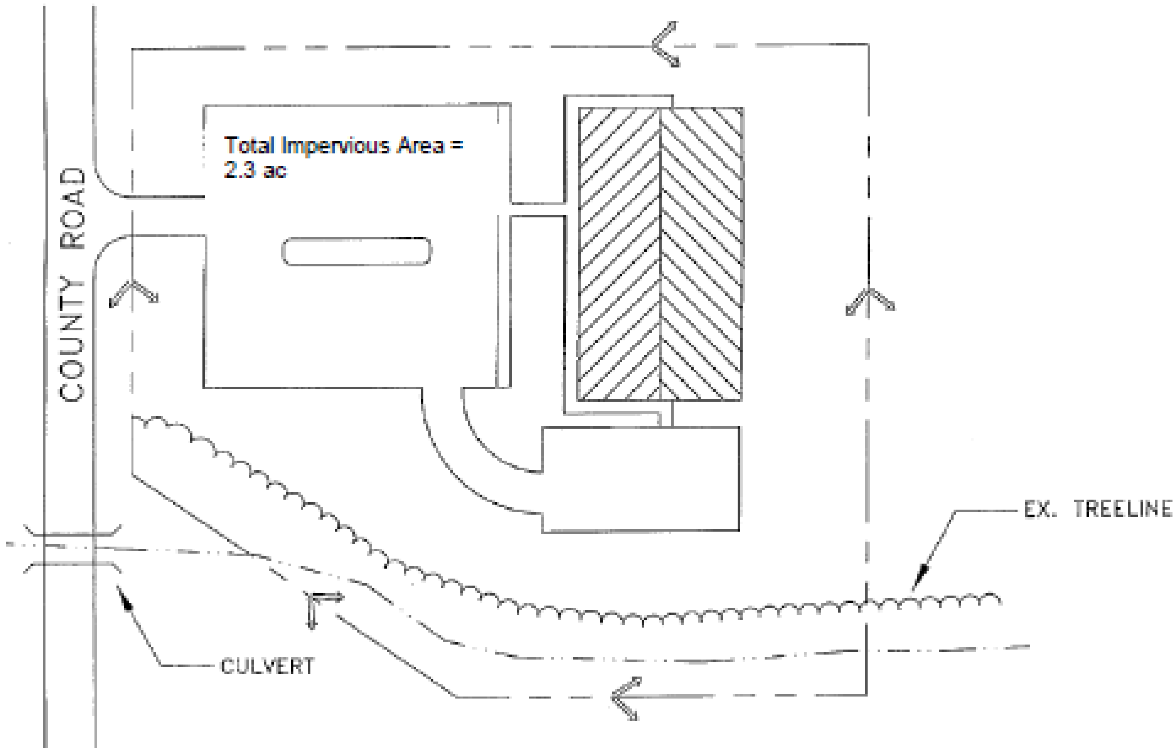


Figure C5-S2- 10: Infiltration design example site plan

Table C5-S2- 2: Bucketsville, IA Community Center; Marshould County, IA

Base Site Data	Hydrologic Data		
Total site drainage area (A) = 4 ac		Pre-	Post-
Impervious area = 2.3 ac; I = 2.2/4 = 57.5%	CN	68	84
Soils: HSG B (loam)	T_c	0.32	0.18

This example is focused on the design of an infiltration trench to meet the water quality treatment requirements for the site. Cpv and Q_p are not addressed in this example, other than determination for preliminary storage volume and peak discharge requirements. The Cpv and Q_p requirements will be handled by another set of downstream BMPs. Infiltration trenches provide water quality treatment (WQv) and recharge volume (Rev). Flows in excess of the WQv will be bypassed. The bypassed flow will be conveyed downstream and combined with other off-site flows in a conventional detention basin for Q_p control.

1. **Step 1.** Compute runoff control volumes from unified sizing criteria.

   a. Compute WQv:

   $R_v = 0.05 + (57.5)(0.009) = 0.57$

   $WQv = \dfrac{(1.25in)(R_v)(A)}{12}$

   $= (1.25)(0.57)(4.0)\left(\dfrac{1ft}{12in}\right)(43{,}560\, ft^2/ac)$

   $= 10{,}345 ft^3 = 0.237 ac\text{-}ft$

   b. Compute stream channel protection volume, (Cpv):

   1) Use WinTR-55 to compute the pre- and post-development peak runoff rates for the 1-year, 24-hour

duration storm.

Condition	**CN**	**Q_1** inches	**Q_1** cfs	**Q_{25}** cfs	**Q_{100}** cfs
Pre-developed	68	0.5	0.6	6.0	9.0
Post-developed	84	1.9	5.5	17.0	22.0

2) Use WinTR-55 to compute channel protection storage volume:
   a) $q_u = 600\, csm/in$
   b) $\frac{q_o}{q_i} = 0.04$
   c) $\frac{V_s}{V_r} = 0.683 - 1.43\left(\frac{q_o}{q_i}\right) + 1.64\left(\frac{q_o}{q_i}\right)^2 - 0.804\left(\frac{q_o}{q_i}\right)^3$
   d) $V_s = CPv\ and\ V_r = volume\ of\ runoff\ in\ inches$
   e) $\frac{V_s}{V_r} = 0.64$
   f) $V_s = CPv = 0.64(1.9in)\left(\frac{1}{12}\right)(4ac) = 0.405ac - ft = 17{,}646ft^3$
   g) $CPv\ of\ 17{,}756ft^3\ to\ be\ released\ over\ 24\ hours$:
   h) $\frac{17{,}656ft^3}{24hr \times 3600sec/hr} = 0.2cfs(average\ release\ rate\ for\ CPv)$

c. Determine overbank protection flood protection volume (Q_{p25}).
   1) Use WinTR-55 for analysis of Q_5 to Q_{100} runoff volume in inches and respective peak rates.
   2) For a Q_{in} of 17 cfs and an allowable Q_{out} of 6 cfs, the V_s necessary for 25-year control is 0.52 ac-ft or 22,677 ft^3 under a developed CN of 84.

d. Compute WQv peak discharge (Q_{wq}) from Chapter 3, section 6 and Modified NRCS WinTR-55 procedure.
   1) $WQv = 10{,}345ft^3 = 0.237ac - ft$
   2) $CN = \frac{1000}{\left[10+5P+10Q_a-10\left(Q_a{}^2+1.25Q_aP\right)^{0.5}\right]}$

      $P = rainfall\ depth\ for\ water\ quality\ storm - 1.25inches$
      $Q_a = runoff\ volume, inches\ (equal\ to\ P \times R_v) = (1.25)(0.57) = 0.712in$
   3) $CN = 1000/[10 + 5(1.25in) + 10(0.71in) - 10[(0.71in)^2 + 1.25(0.71in)(1.25in)]^{0.5}$
      CN = 93.8 Use CN = 94
   4) Use T_c = 0.18 hour

d. Compute Q_{wq} using WinTR-55 using modified CN and T_c:
   1) WinTR-55 results for *modified* CN = 94 and T_c = 0.18 hr: For 1.25-inch rainfall, q_u = 622.89 csm/in
   2) Q_{wq} = 3.89 cfsec

e. Compute 1-year, 2-year, and 10-year peak discharge using conventional WinTR-55 procedure:
   1) For 57.5% impervious, B soils, CN=98 for impervious and CN=64 for open space
   2) CN = 84
   3) Use T_c = 0.18 hr
   4) WinTR-55 results (runoff and peak discharge summary):

Design storm event	Runoff volume (inches)	Peak discharge, Q (cfs)	Unit discharge, q_u (csm/in)
1-year	1.018	5.57	891.69
2-year	2.187	11.94	1910.82
10-year	2.897	15.7	2511.58
25-year	4.095	21.9	3504.19
50-year	4.754	25.25	4040.43
100-year	5.706	30.04	4806.93

2. **Step 2.** Determine if the development site and conditions are appropriate for using an infiltration trench. Site specific data:
   - Soil: loam
   - Infiltration rate: 0.8 in/hr
   - Ground elevation at BMP: 1020
   - Seasonally high water table: 1008
   - Stream invert: 1006
   - Soil slopes: 1.2%

3. **Step 3.** Confirm design criteria and applicability.

Infiltration Feasibility	
Criteria	**Status**
Infiltration rate (*f*) greater than or equal to 0.5 in/hr.	Infiltration rate is 0.8 in/hr. OK.
Soils have a clay content of less than 20% and a silt/clay content of less than 40%.	Loam soil at this site meets both criteria.
Infiltration cannot be located on slopes greater than 6% or in fill soils.	Slope is 1.2%; not fill soils. OK.
Hotspot runoff should not be infiltrated.	Not a hotspot land use. OK.
Infiltration is prohibited in karst topography.	Not in karst. OK.
The bottom of the infiltration facility must be separated by at least 4 feet vertically from the seasonally high water table.	Elevation of seasonally high water table: 1008 feet. Elevation of BMP location: 1020 feet. The difference is 12 feet. The trench can be up to 8 feet deep. OK.
Infiltration facilities must be located 100 feet horizontally from any water supply well.	No water supply wells nearby. OK.
Maximum contributing area generally less than 5 acres. (Optional)	4 acres. OK.
Setback 25 feet down-gradient from structures.	50 feet straight-line distance between the parking lot and the tree line. OK if the trench is 25 feet wide or narrower.

4. **Step 4.** Size the infiltration trench.

   a. Use Equation C5-S2- 4.
      1) $A_t = WQv/(nd_t + fT/12)$
      2) $A_t - area\ of\ trench, ft^2$
      3) WQv = volume to be stored/infiltrated, ft^3
         n = porosity
         F = infiltration rate, in/hr
         T = fill time
         d_t = depth of trench, ft

4) Assume that: n = 0.34
f = 0.8 in/hr
T_s = maximum storage time = 72 hours
$d_{max} = \left(\frac{fT_s}{n}\right)/12 = \left[\frac{(0.8)(72.0)}{0.34}\right]/12 = 14.2ft$

However, maximum depth for this site will be 8 ft due to water table

b. $A_t = (10{,}345ft^3/[(0.34)(8ft) + (0.8)(2.0)/12] = 3{,}635ft^2$

c. For a width, W, of 25 feet, determine the length, L:
L ft = 3,635 ft/25 ft = 145 ft

Assume that ⅓ of the runoff from the site drains to Point A and ⅔ drains to Point B. Use an L-shaped trench in the corner of the site (see Figure C5-S2- 11 for a site plan view). The surface area of the trench is proportional to the amount of runoff it drains (e.g., the portion draining from Point A is half as large as the portion draining Point B).

5. **Step 5.** Size the flow diversion structures.

a. Since two entrances are used, two flow diversions are needed. For the entire site: Q_{25} = 21.9 cfs
b. Peak flow for WQv = 3.89 cfs (step 3).
c. For the first diversion (Point A):

1) Assume peak flow equals ⅓ of the value for the entire site. Thus, $Q_{25} = \frac{21.9}{3} = 7.3cfs$. Peak flow for $WQv = \frac{3.89}{3} = 1.3cfs$

2) Size the low-flow orifice to pass 1.3 cfs with 1.5 feet of head using the orifice equation.
$Q = CA(2gh)^{1/2}; 1.3cfs = 0.6A(2 \times 32.2fps^2 \times 1.5ft)^{1/2}$
$A = 0.22ft^2; d = 0.53ft$; use 8-inch pipe with 8-inch gate valve

3) Size the 25-year overflow weir crest at 1022.5 feet.
Use a concrete weir to pass the 25-year flow (7.3-1.3 = 6 cfs). Assume 1 foot of head to pass this event. Size using the weir equation.
$Q = CLH^{1.5}$
$L = Q/(CH^{1.5})$
$L = 6$ cfs/ $(3.1)(1)^{1.5}$ = 1.93 ft; use 2 feet (see Figure C5-S2- 12)

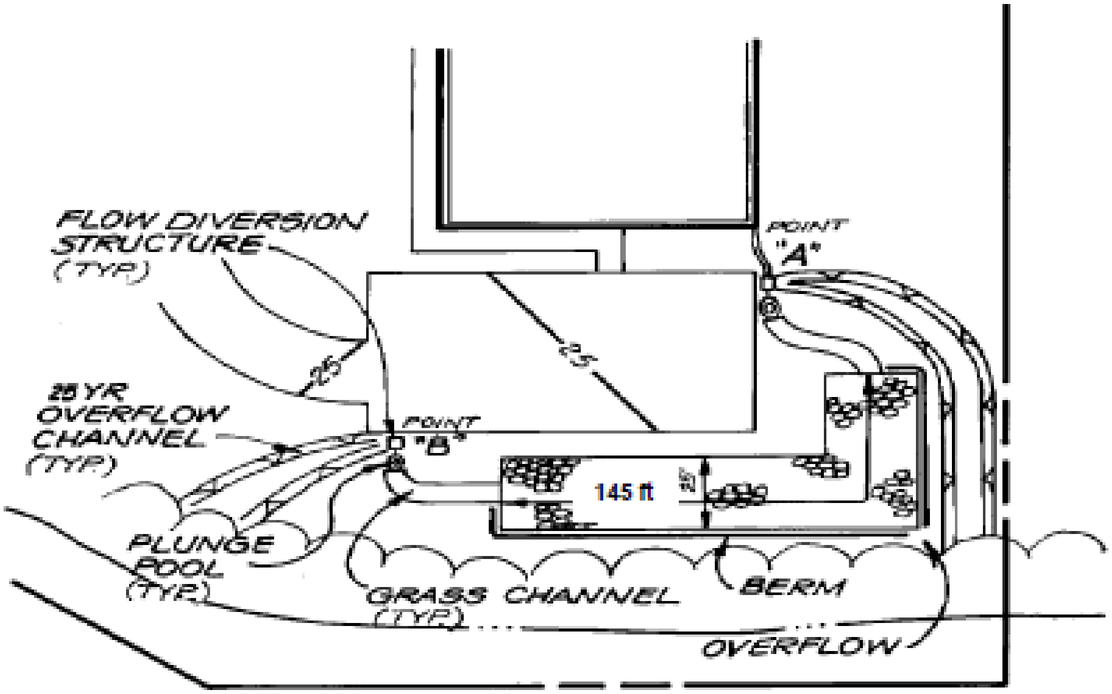
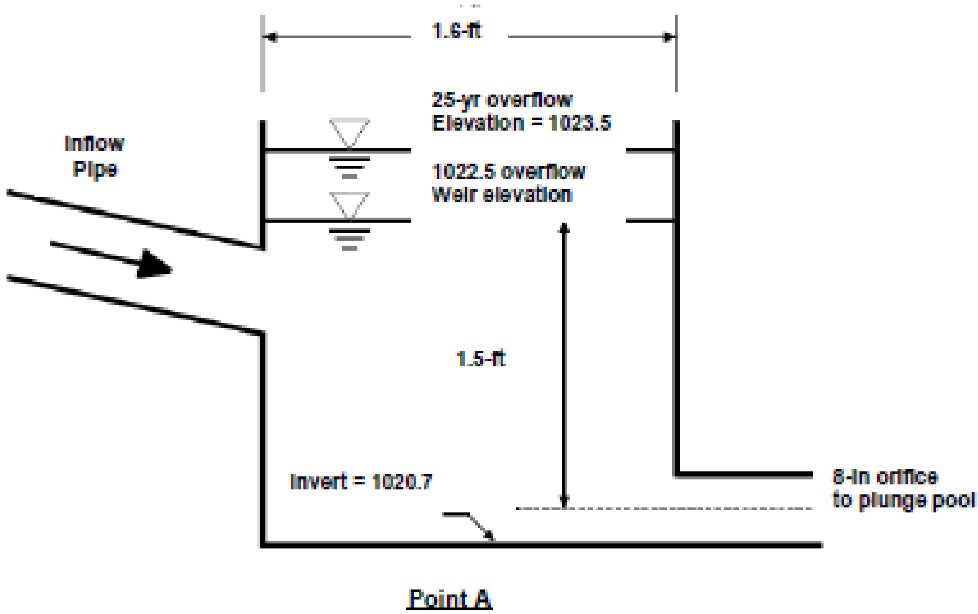


Figure C5-S2- 11: Infiltration trench site plan

d. Size the second diversion (Point B) using the same techniques.
   1) Peak flow equals ⅔ of the value for the entire site. Thus: Q_{25} = (21.9)(0.67) = 14.7 cfs Peak flow for WQv = (3.89)(0.67) = 2.6 cfs
   2) Size the low-flow orifice to pass 2.6 cfs with 1.5 ft of head using the orifice equation.
      $Q = CA(2gh)^{1/2};\ 2.6cfs = 0.53(2 \times 32.2fps^2 \times 1.5ft)^{1/2}$
      $A = 0.44ft^2; d = 0.75ft$; use 10-inch pipe with 10-inch gate valve

e. Size the 25-year overflow weir crest at 22 feet. Use a concrete weir to pass the 25-year flow (14.7 – 2.6 = 12.1 cfs). Assume 1 foot of head to pass this event. Size using the weir equation.
   $Q = CLH^{1.5}$;
   $L = Q/(CH^{1.5})$
   $L = 12.1\ cfs/(3.1)(1)^{1.5} = 3.9$ ft; use 4 ft (see Figure C5-S2- 12)



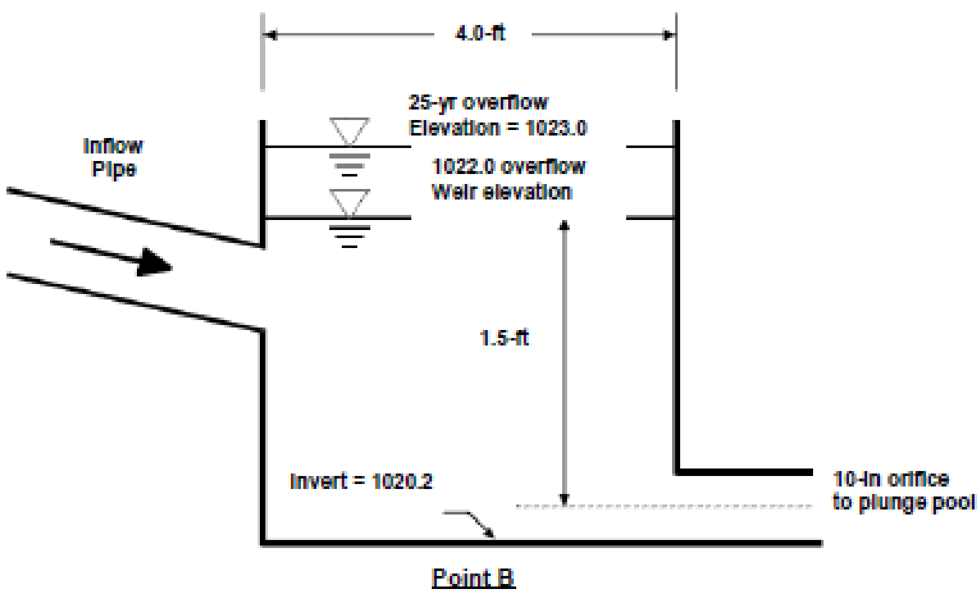


Figure C5-S2- 12: Weir elevation sizing

6. **Step 6.** Size pre-treatment volume and design pre-treatment measures. As a rule of thumb, size pre-treatment to treat 25% of the WQv. Therefore, treat 10,345 ft^3 x 0.25 = 2,586 ft^3. For pre-treatment, use a pea-gravel filter with a geotextile filter fabric, a plunge pool, and a grass channel.

   a. **Pea gravel filter.** The pea gravel filter layer covers the entire trench with 2 inches of material (see Figure C5-S2- 11). Assuming a porosity of 0.32, the water quality treatment volume in the pea gravel filter layer is: $WQv_{(filter)} = (0.32)(2 \text{ in})(1/12)(3635 \text{ ft}^2) = 194 \text{ ft}^3$.

   b. **Plunge pools.** Use a 5-ft x 10-ft plunge pool at Point A and a 10-ft x 10-ft plunge pool at Point B with average depths of 2 feet. $WQv_{(pool)} = (10 \text{ ft})(10 \text{ ft} + 5 \text{ ft})(2 \text{ ft}) = 300 \text{ ft}^3$

c. **Grass channel.** The WQv for the grass channel needs to treat the remaining volume (2,586- 194-300)ft^3 = 2092- ft^3

Use the procedure in Chapter 9 to design the grass channels:

a. The channel at Point A should treat ⅓ of 2092 ft^3 or 698 ft^3.

b. Assume a trapezoidal channel with a 4-foot channel bottom, 3:1 side slope, and a Manning's n of 0.15. Use a 1% longitudinal slope.

c. Use a peak discharge of 1.3 cfs (Peak flow for ⅓ of the WQv).

d. Compute velocity: V=0.5 fps.

e. To retain the ⅓ of WQv (3,448 ft^3) for 10-minutes, the length would be 300 feet. Since the swale only needs to treat 25% of the WQv minus the treatment provided by the plunge pool and the gravel layer, or 698 ft^3, the length is pro-rated to reflect this reduction.

   Therefore, adjust the length: L = (300 ft) (698 ft^3/3448 ft^3) = 60.7 feet. **Use 60 feet.**
   Size the channel at Point B in a similar manner for the ⅔ of WQv.

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

Description: Infiltration basins are dry ponds constructed to allow infiltration to occur simultaneously with other treatment processes. Infiltration basins are often designed as offline or end-of-pipe BMPs to capture a defined volume of stormwater runoff volume and transform the water into groundwater flow through infiltration. Pollutants are also removed through filtration and adsorption as the water percolates through the underlying soils. A key feature of an infiltration basin is vegetation, which increases the infiltration capacity of the basin. Dense vegetation also impedes soil erosion and scouring of the basin floor. It is generally characterized as an open impoundment dedicated to infiltration, greater than 15 feet wide, with a flat earthen floor.

Typical uses: Residential watersheds

Advantages:
- Can be effective for removing fine sediment, trace metals, nutrients, and bacteria
- Principle benefit is groundwater recharge and preservation of the natural water balance of the development site
- Can be useful for controlling the WQv and often can provide for the channel protection volume (Cpv)
- Reduce flooding
- Reduce thermal impacts to streams

Disadvantages/limitations:
- Not appropriate for treating significant loads of sediment and other pollutants due to potential for clogging of the basin infiltration surface
- Potentially high failure rate due to improper siting, design, and lack of maintenance – especially if pre-treatment is not included in the overall design
- Maintenance of effective upstream pre-treatment, a sediment forebay, and vegetation in the basin infiltration area will prolong infiltration performance and increase the interval between cleaning
- Not recommended in karst areas, industrial parks, high-density or heavy industrial areas, chemical or pesticide storage areas, or fueling stations

Maintenance requirements:
- Remove sediment accumulation from basin and pre-treatment areas
- Mow and remove litter and debris

A. Description

Infiltration basins are dry ponds constructed to allow infiltration to occur simultaneously with other treatment processes. An infiltration basin can be used for both stormwater quality and quantity controls. The storage basin is designed with a large surface area and the design water depth is kept shallow (≤1 foot). The influent point(s) to the basin are configured with energy dissipation and/or a level spreader to efficiently distribute the flow into the basin. Infiltration basins are detention ponds constructed to allow infiltration to occur simultaneously with other treatment processes. Figure C5-S3- 1 provides a typical detail for a conventional infiltration basin. Figure C5-S3- 2 illustrates a combined infiltration/detention basin. The operating characteristics of infiltration basins are essentially the same as for dry detention, with a few significant exceptions:

1. Infiltration basins also remove dissolved solids in the volume of infiltrated water, whereas dry detention basins do not.
2. The settling velocities of the particles are increased by a value equal to the infiltration rate in the basin. The impact would be more important for the clay-sized particles than for silt, sand, and small or large aggregates.
3. Infiltration practices differ from typical dry basins because they have the ability to meet the groundwater recharge requirements (see Chapter 2), and therefore provide an additional element of control or performance.

B. Stormwater management suitability

Infiltration basins are designed primarily for reduction in stormwater runoff volume, but also have high removal capability for fine particulates, metals, and bacteria. Runoff volume control can be achieved for the WQv for smaller storm events, up to the limits of the local infiltration capacity of the local soils. The runoff volume gradually infiltrates through the bottom and sides of the trench and into the subsoil, eventually reaching the water table. By diverting runoff into the soil, an infiltration trench not only treats the water quality volume, but also helps to preserve the natural water balance on a site, recharge groundwater, and preserve base flow.

An infiltration basin can also be designed to capture and infiltrate the entire channel protection volume in either an offline or online configuration. For larger sites, or if only the WQv is diverted to the basin, another structural control must be used to provide Cpv extended detention. Since infiltration basins are similar in form to traditional dry detention basins, additional control for peak discharge reduction (overbank flooding-Q_p) can be provided by adding additional depth for detention storage and including a suitably-sized outlet structure.

C. Pollutant removal capabilities

Infiltration basins are effective in removing both soluble and fine particulate pollutants in urban runoff. Coarse-grained particulates should be removed with preliminary upstream BMPs. While the pollutant removal capability of infiltration basins can be highly variable, the removal is achieved by diverting the run off through the floor of the basin and into the soil. Table C5-S3- 1 provides estimates of removal rates for infiltration basins sized to capture the WQv.

Table C5-S3- 1: Pollutant removal rates for infiltration basins

Pollutant	**Removal rate %**
Sediment	90%
Total P	60-70%
Total N	55-60%
Metals	85-90%
Bacteria	90%

Source: US EPA, 1983; Stahre and Urbonas, 1990; ASCE, 2001

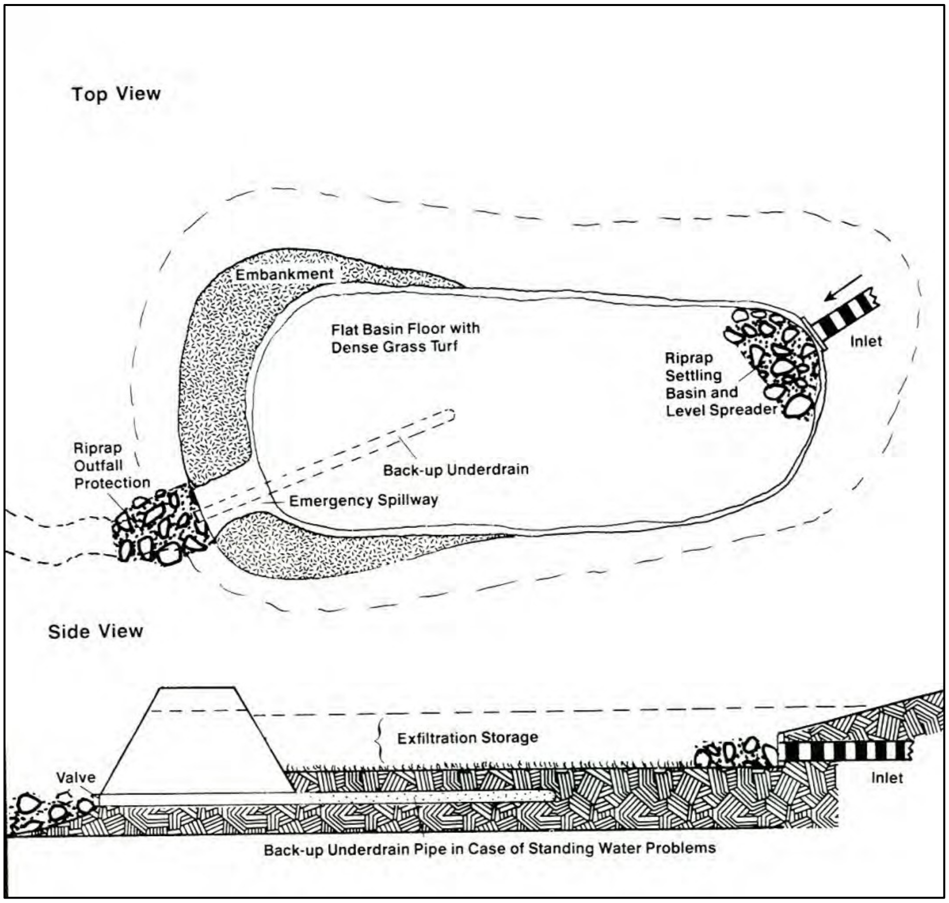


Figure C5-S3- 1: Infiltration basin schematic
Source: Schueler, 1987

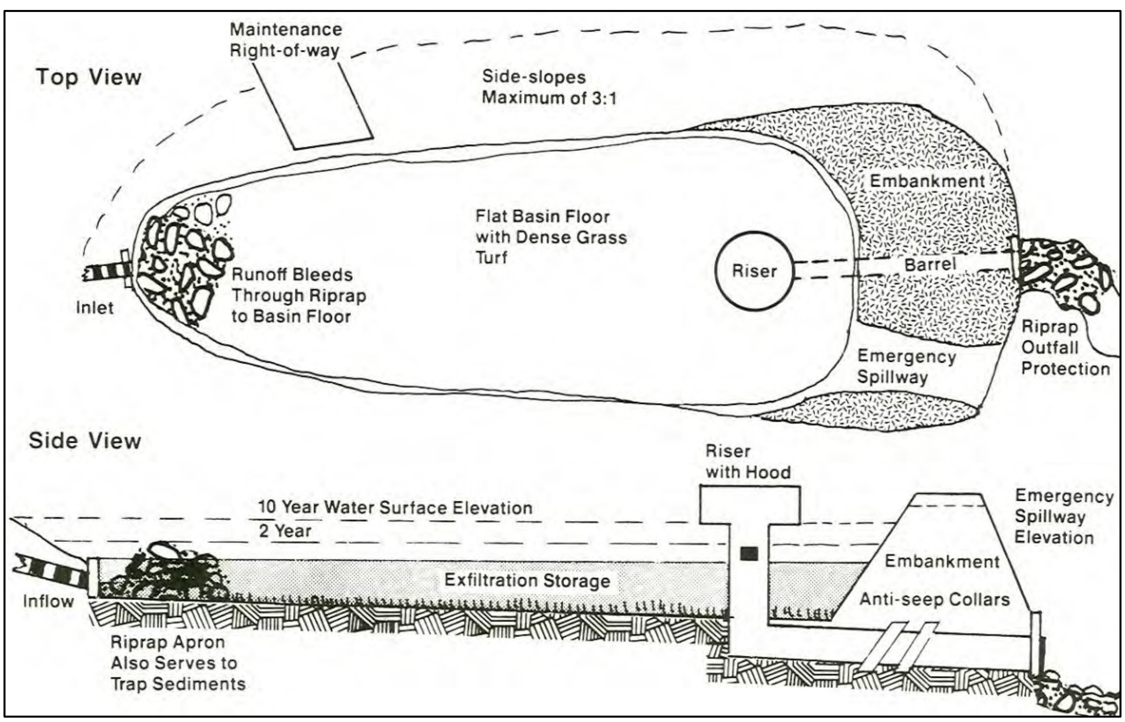


Figure C5-S3- 2: Combined infiltration and detention basin configuration
Source: Schueler, 1987

D. Application and feasibility

The infiltration basin uses an open area or shallow depression for storage. These basins may or may not have a permanent pool. The success of infiltration basins depends on locating the basins above highly-pervious soils and properly constructing the basins to maintain the permeability of the basin floor infiltration area.

1. **Soils.** Soils are the key evaluation factor, and are initially based on an investigation of the NRCS hydrologic soils groups (HSG) at the site (see Chapter 5, section 1). Note that more detailed geotechnical tests are usually required for infiltration feasibility, and during design to confirm permeability and other factors. Infiltration basins must be built in soils with high infiltration rates.

    a. Infiltration basins are not a feasible option on sites with HSG-D soils, or any soil with clay content greater than 30%. Silt loams and sandy clay loams (HSG-C soils) provide marginal infiltration rates, and would not be suitable for infiltration basin application in most circumstances. Soils with a combined silt/clay percentage of over 40% by weight will likely experience frost-heave and should be avoided for infiltration basin application. A site located over fill soils that form an unstable subgrade should also be avoided.

    b. If the soils at the site pass the initial screening discussed above, an additional series of soil cores are collected to a depth of at least 5 feet below the proposed elevation of the basin bottom. Since soil conditions can vary substantially over a short distance, a minimum of 6-8 soil borings may be required across the site to predict future infiltration performance. The soil cores are examined for evidence of impermeable soil layers that can impede infiltration. The presence of impermeable layers may not preclude the use of a basin as long it penetrates the layers completely. Alternately, if impervious layers are present, soils can be removed and replaced with more permeable materials that penetrate to a pervious layer.

    c. At least three in-hole conductivity tests should be performed using USBR 7300-89 or Bouwer-Rice procedures (the latter if groundwater is encountered within the boring); two tests at different locations within the proposed basin, and the third down-gradient by no more than approximately 40-50 feet. The tests should measure permeability in the side slopes and the basin subgrade within a depth of 12-15 feet of the basin floor invert. The minimum acceptable hydraulic conductivity as measured in any of the three required test holes is 0.5 in/hr. If any test hole shows less than the minimum value, the site should be disqualified from further consideration.

    d. The results of a study of disturbed and compacted urban soils (i.e. heavy equipment) compared to undisturbed sites by the NRCS National Soil Mechanics Center show that as soil bulk density increases to 1.65 g/cm^3, infiltration rates of the soil decrease rapidly. When the bulk density increases above 1.65 g/cm^3 infiltration rates decline slowly, approaching zero. The measured infiltration rates for disturbed soils with high bulk densities were significantly lower than expected (OCSCD et al., 2001). For infiltration basin design, soil borings taken throughout the proposed site should indicate soil bulk densities in the basin bottom of $\leq$1.45 g/cm^3, and measured permeability rates of $\geq$0.5 inches/hr.

2. **Slope.** Infiltration basins are not feasible if the slope of the contributing watershed is greater than 20%. Within the basin itself, a slope of less than 5% is preferable.

3. **Water table.** The bottom of the infiltration facility should be separated by at least 4 feet vertically from the seasonally high water table or bedrock layer, as documented by on-site soil testing.

4. **Drainage area.** The contributing drainage area to an individual infiltration basin practice designed solely for water quality control can range from 5-25 acres. A maximum of 10 acres is recommended for full conventional infiltration basins when all of the site criteria have been met satisfactorily and good pre-treatment is provided. For combination infiltration/detention basins, a drainage area up to 50 acres is typical. The volume to be infiltrated is determined from the WQv and/or Cpv, and the remaining volume for peak discharge control is established above the maximum depth established for infiltration. The storage volume for peak discharge control is discharged through a separate outlet structure. If the drainage area is more than 50% impervious, the space required for

infiltration may be become too large to accommodate on the site.

5. **Head.** Head is the elevation difference needed at a site (from the inflow to the outflow) to allow for gravity operation within the practice. A minimum head of 1 foot, and a maximum of 3 feet is recommended. Additional head may be required if additional storage volume for peak discharge control for Q_p is provided.

6. **Separation distances.** Infiltration basins should be located a minimum of 100 feet horizontally from any water supply well. Infiltration practices should not be placed in locations that cause water problems to downgrade properties. Infiltration facilities should be setback 25 feet down-gradient from structures.

E. Planning and design criteria

1. **Pre-treatment.** A minimum of 25% of the WQv is recommended to be pre-treated prior to entry into the infiltration basin. If the infiltration rate for the underlying soils is greater than 2 in/hr, 50% of the WQv should be pre-treated prior to entry into the infiltration facility. Exit velocities from pre-treatment should be non-erosive (<10 ft/sec) during the 2-year design storm. Infiltration systems can be designed using redundant methods (treatment train approach) to protect the long term integrity of the infiltration rate. The following pre-treatment techniques can be used to provide protection against premature clogging and failure:
   - Grass swale or grass filter strip
   - Sedimentation basin, sediment forebay, stilling basin, sump pit, or other acceptable measures
   - Bottom sand layer

2. **Surface area of basin floor.** The rate and quantity of ex-filtration is enhanced by increasing the surface area of the basin floor, especially as the soil infiltration rate approaches the minimum rate of 0.5 in/hr. Therefore, large and relatively shallow (<3 ft) basins are preferable to basins that are small and deep. Additional surface area for the basin floor can also help compensate for diminished infiltration capacity from long-term surface clogging.

3. **Reducing influent water velocity.** Inlet conveyance channels to the basin are stabilized to prevent incoming runoff velocities from reaching erosive conditions and scouring the basin floor. Providing riprap at the inlet channels and pipe outfalls will provide effective control. The riprap will also serve to spread the incoming flow more uniformly over the surface of the basin floor to provide improved infiltration. The best approach is to avoid a riprap pilot channel, and instead terminate the riprap in the form of a wider structure to serve as a level spreader (Figure C5-S3- 1). A 20-foot filter strip combined with a riprap level spreader will provide effective sheet flow onto the basin floor.

4. **Basin slopes.** The floor of the basin is graded to have a slope close to zero. The goal in infiltration basin design is to achieve a uniform ponding depth across the entire surface of the basin. If the basin is sloped towards the outlet structure riser, or if low spots are created, the runoff volume will concentrate in small portions of the basin and reduce the infiltration effectiveness. The low spots will tend to remain under water a longer period of time, due to the limited soil infiltration capacity. Over a longer period of time, these low spots may eventually become clogged with excess sediment. The basin side slopes should be ≤3:1 to enhance vegetative stabilization. The shallower side slopes facilitate mowing, access, and improved public safety.

5. **Establishing vegetation.** A dense turf of water-tolerant grass is established on the floor and side slopes of the infiltration basin immediately after construction. The turf promotes better pollutant removal because:
   - Root penetration and thatch formation in the turf maintains and may improve the original infiltration capacity of the basin floor
   - The turf grows through the accumulated sediment and pollutant deposited in the basin, preventing re-suspension during larger storm events
   - The turf assimilates soluble nutrients for growth. Plant nutrients can be effectively removed from the system if the clippings are removed during/after mowing operations
   - A dense growth of turf will prevent soil erosion and scouring of the basin floor that could reduce the overall efficiency of the basin. Ground covers such a tall fescues and Bermuda grass are generally used for this purpose.

   A dense and vigorous vegetative cover is established over the contributing pervious drainage areas before runoff is accepted into the facility. Infiltration basin sites should not serve as a sediment control device during the site construction phase. In addition, the erosion and sediment control plan for the site must clearly indicate how sediment will be prevented from entering the infiltration site. Do not construct infiltration practices until all of the contributing drainage area has been completely stabilized.

6. **Basin buffer.** A vegetative screen around the basin to restrict direct view from adjacent properties may improve the aesthetics of the site and public acceptance of the facility. Regular mowing will prevent establishment of woody vegetation growth from the buffer area onto the basin bottom.

7. **Maximum drain time.** The depth of ex-filtration storage within the basin is adjusted to ensure it completely drains within 72 hours after the maximum design ex-filtration event. A drain time of 48 hours can be used for a more conservative design. Complete drainage is needed to maintain aerobic conditions in the unsaturated zone under the basin, to support bacteria that aid in organic pollutant removal. It is also important to completely empty the basin before the next storm. For example, the average time between storms events in Iowa in the warmer season (June to September) is 96 hours.

8. **Sediment forebays.** The long-term performance of an infiltration basin can be enhanced if sediment forebays are constructed near the inlet(s) to trap incoming sediment loads. The forebays also serve to reduce the influent velocity and provide uniform dispersal of flow into the basin area.

9. **Winter operation.** When the soil freezes, infiltration will likely cease. While some nominal infiltration may occur under partially frozen conditions, the basin will not likely treat rain or snowmelt during the winter. In this case, a bypass at the inlet can be provided for the winter season; or an accessory low-level outlet can be provided and opened to allow direct drainage of snowmelt or winter runoff.

10. **Safety.** Fencing around the basin area can be included in the final site plan if public access to the area is not desired. If the area around the basin has a recreational use, a safety shelf around the perimeter of the basin can be included for times when the basin is flooded and the design depth will exceed 3 feet. Steep slopes should be avoided (≤3:1), and signs should warn against deep water or any health risks. An auxiliary spillway is provided to safely bypass or move high flows through the basin and protect against structural failure.

F. General design

1. Water quality volume (WQv) is determined as described in Chapter 2, section 1 and Chapter 3, section 6.
2. Basin should be sized so the entire water quality volume is infiltrated within 48-72 hours.
3. Vegetation establishment on the basin floor may help reduce the clogging rate.
4. The truncated hydrograph method described later in this section can be used as an alternative analytical procedure if the infiltration basin is used to control peak discharge.
5. If runoff is delivered by a storm drain pipe or along the main conveyance system, the infiltration practice should be designed as an offline practice (see Figure C5-S3- 4).
6. Adequate stormwater outfalls should be provided for the overflow associated with the 10-year design storm event (non-erosive velocities on the downslope).
7. A minimum of 25% of the WQv must be pre-treated prior to entry to an infiltration facility. If the “f” for the underlying soils is greater than 2 in/hr, 50% of the WQv should be pre-treated prior to entry into an infiltration facility. This can be provided by a sedimentation basin, sediment forebay, stilling basin, sump pit, or other acceptable measures. Exit velocities from pre-treatment should be non-erosive during the 2-year design storm.
8. The construction sequence and specifications for each infiltration practice should be followed, as outlined in the SUDAS specifications. The longevity of infiltration practices is strongly influenced by the care taken during construction.
9. Groundwater separation should be at least 4 feet from the basin invert to the measured ground water elevation.
10. Location away from buildings, slopes, and highway pavement (greater than 25 feet) and wells and bridge structures (greater than 100 feet).
11. Sites constructed of fill, having a base flow or a slope greater than 15%, should not be considered.
12. Ensure that adequate head is available to operate flow splitter structures (to allow the basin to be offline) without ponding in the splitter structure or creating backwater upstream of the splitter.
13. A conveyance system should be included in the design of all infiltration basins in order to ensure that excess flow

is discharged at non-erosive velocities. The overland flow path of surface runoff exceeding the capacity of the infiltration system is evaluated to preclude erosive concentrated flow. If computed flow velocities do not exceed the non-erosive threshold, overflow may be accommodated by natural topography or grass swales (see Chapter 9). Adequate stormwater outfalls should be provided for the overflow associated with the 10-year design storm event (non-erosive velocities on the downslope).

14. If runoff is delivered by a storm drain pipe or along the main conveyance system, the infiltration practice should be designed as an offline practice (see Figure C5-S3- 4 for an example of an offline infiltration basin).

G. Physical specifications, geometry, and volume

1. Configuration of a conventional infiltration basin for water quality (WQv) treatment and a combination infiltration/detention basin for quality and quantity control are shown in Figure C5-S3- 1 and Figure C5-S3- 2, respectively.
2. For the design of larger infiltration basins, the routing of small baseflows and larger storm runoff volume can be problematic while still providing effective ex-filtration capacity for the small and moderate size storms. A design variant called a side-by-side infiltration basin (Figure C5-S3- 3) contains a riprap pilot channel along one margin of the basin, and extends all the way to the outlet structure riser. The pilot channel is elevated several feet above the basin floor. Baseflow is confined to the pilot channel (use an impermeable geotextile liner), and travels directly to an under-sized low-flow orifice at the base of the riser, and then discharges from the basin. Storm flow pulses are also directed though the pilot channel. Once the incoming storm flow reaches a given depth, it overflows the liner in the pilot channel and is conveyed down across the basin floor. The invert of the low-flow orifice is set from a dead storage zone down to the basin floor, thus storing the equivalent of the first flush runoff volume, and/or the WQv.
3. The offline design variant (Figure C5-S3- 4) is used to divert and ex-filtrate the first flush runoff volume of larger storms (>1.25 inch) and the WQv design storm (≤1.25 inch) from a storm sewer or open surface channel. These may be useful for development situations where ex-filtration cannot be achieved at a downstream stormwater detention basin due to soil limitations. The design utilizes a combination of an offline sand filter and infiltration basin to treat the WQv or first flush volume. A weir is placed across a natural or man-made channel diverting runoff into an offline sand filter. After passing through the filter, runoff is collected by subdrains leading to a level vegetated infiltration basin. This design is recommended for sites which produce high sediment loads.

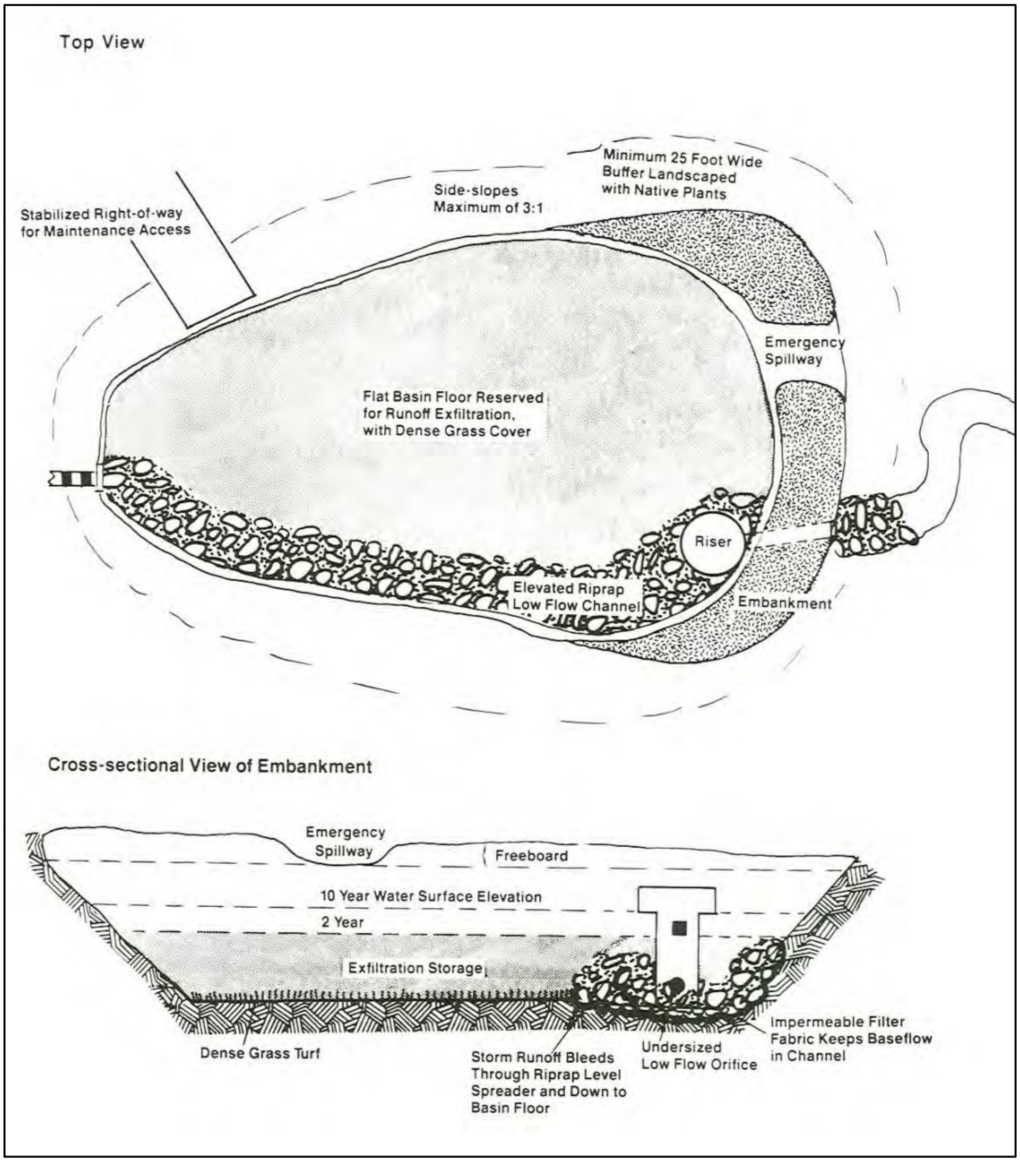


Figure C5-S3- 3: Side-by-side infiltration basin configuration
Source: Schueler, 1987

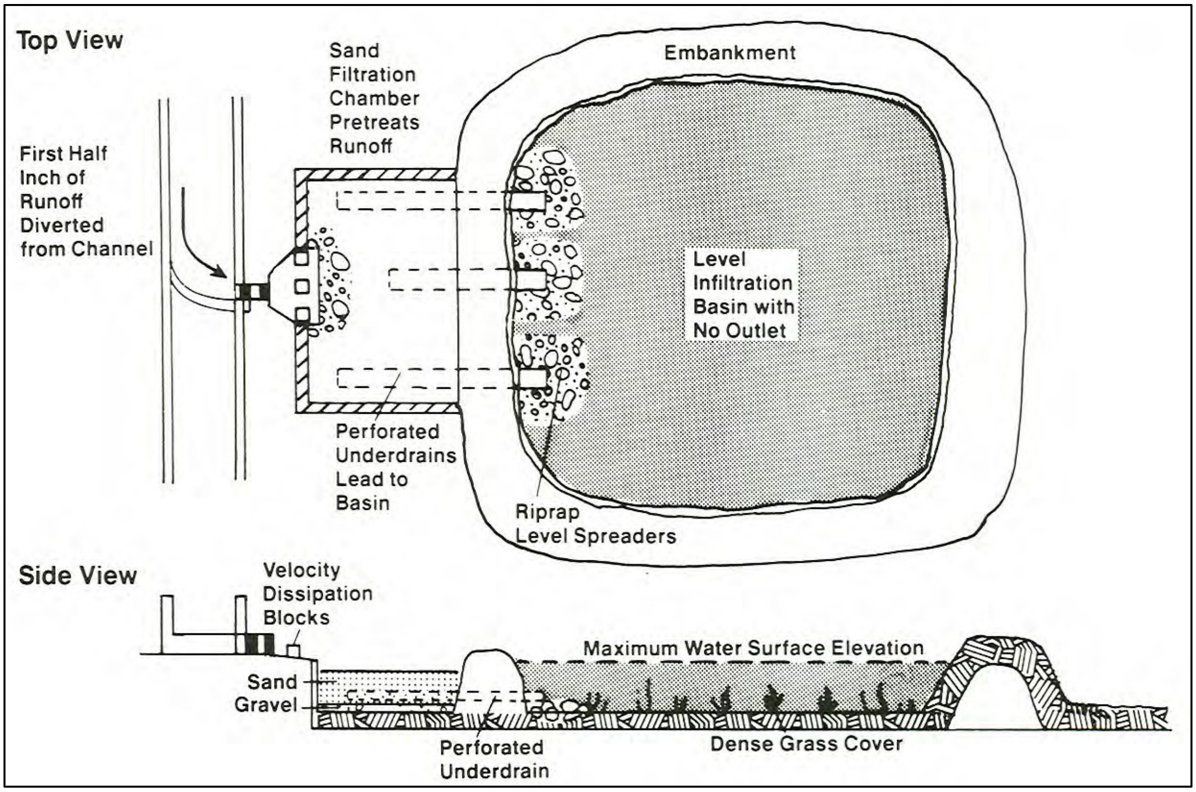


Figure C5-S3- 4: Side-by-side infiltration basin configuration
Source: Schueler, 1987

H. Design of infiltration basins

There are two general types of situations where infiltration basins may be used: First is the determination of the dimensions of an infiltration basin required to provide storage and treatment of the WQv or design peak discharge (Q_p). Second, site conditions may dictate the layout and capacity of the infiltration basin, and in this case, the level of control provided by such a layout might only provide partial treatment of the WQv. In the latter case, control may not be sufficient, and additional control, possibly using other acceptable BMPs, may be required. However, both cases are suitable for use when considering incorporating groundwater recharge into future development. The following procedure can be used for designing infiltration basins to meet the WQv, and the overbank flood protection (Q_p) volume requirements. These methods are based on the methodology described in the Maryland Stormwater Design Manual (2000). The design procedures are based on either intercepting the WQv from the area contributing runoff, or using the truncated hydrograph method for control of the runoff from an area for either the Cpv or Q_p. The design equations may be defined for either case of stormwater quality or quantity control because the volume of water (V_B) stored in the individual infiltration practice may be determined from the methods described earlier.

An alternative sizing criteria is the use of the maximized capture volume method (ASCE/WEF, 1998) described in Chapter 3, section 2.

The design of an infiltration basin is based on the same soil textural properties and maximum allowable depth as the infiltration trench such that a feasible design is possible. However, because the infiltration basin uses an open area or shallow depression for storage, the maximum allowable depth (d_{max}) should meet the following criteria:

Equation C5-S3- 1

$$d_{max} = fT_p$$

Where:
f is the final infiltration rate of the trench area (in/hr)
T_p is the maximum allowable ponding time (hr)

Values of d_{max} for selected types and minimum infiltration rates and drain times of 48 hours and 72 hours are given in Table C5-S3- 2.

Table C5-S3- 2: Soil types, infiltration rates and maximum storage depth for infiltration basins

Soil texture type	Minimum infiltration rate	NRCS HSG	Maximum depth of storage dmax (in)	
	f (in/hr)		48 hours	72 hours
Sand	8.27	A	397	595
Loamy sand	2.41	A	116	174
Sandy loam	1.02	B	49	73
Loam	0.52	B	25	37

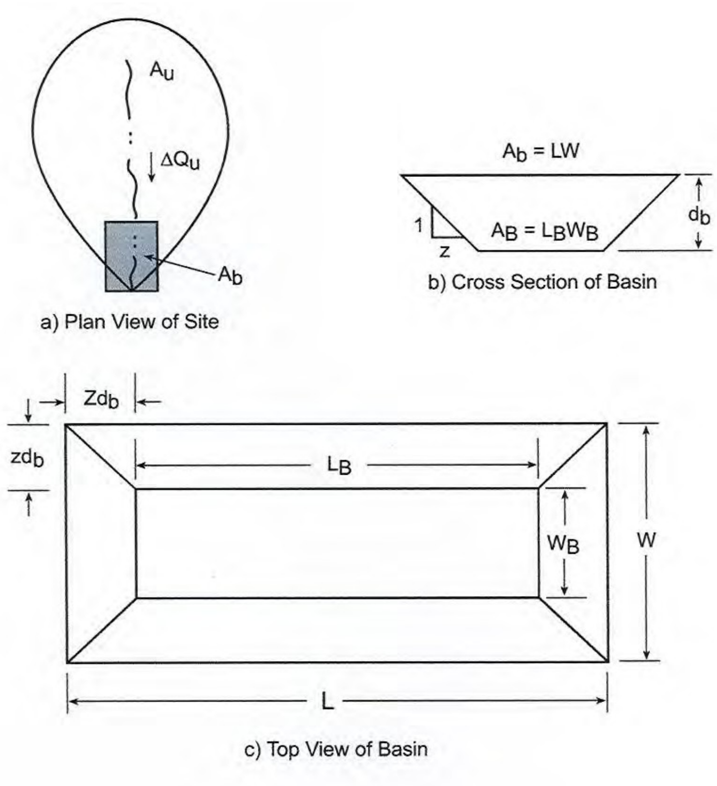


Figure C5-S3- 5: Schematic of basin nomenclature

The following design calculations assume infiltration only through the basin bottom area. Neglecting the likely infiltration through the sides of the basin will provide some additional design factor of safety.

Three design elements must be considered when sizing an infiltration basin:

- Required storage volume
- Maximum basin depth
- Basin volume

An infiltration basin is sized to accept the design volume that enters the basin (V_r), plus the volume of rain that falls on the surface of the basin (PA_B), minus the ex-filtration volume (fTA_B) out of the bottom of the basin. The design volume in most cases will be the WQv determined for the drainage area (A_u). Based on NRCS hydrograph analysis, the effective filling time for most infiltration basins will generally be less than 2 hours. Therefore use T = 2 hours. The volume of water that must be stored in the basin (V_B) is defined as:

Equation C5-S3- 2

$$V_B = V_r + \frac{PA_B}{12} - fTA_B$$

Or

$$V_B = WQv + \frac{PA_B}{12} - fTA_B$$

Where:
P is the design rainfall event (in)
A_B is the basin surface area (ft^2)
WQv (ft^3)

For most design storm events, the volume of water due to rainfall on the surface area of the basin (PA_B) is small when compared to the design volume (V_r or WQv) of the basin, and may be ignored with little loss in accuracy to the final design. Likewise, the term fTA_B represents the volume of water infiltrated through the basin bottom during the time inflow exceeds the outflow (fill time). For a fill time of 2 hours, this volume may be so small in relation to the runoff volume that it can be ignored without introducing significant error.

The volume of rainfall and runoff entering the basin can be defined in terms of basin geometry. The geometry of a basin will generally be in the shape of an excavated trapezoid with a specified side slope (See Figure C5-S3- 5). The average end-area equation (Equation C5-S3- 3) is used to estimate the storage volume of the infiltration basin. The volume of a trapezoidal shaped basin may be approximated by:

Equation C5-S3- 3

$$V = [(A_B + A_b)d_b]/2 = [LW + L_b W_b)d_b]/2$$

Where:
A_b = water surface area at the design depth (ft^2)
A_B = the bottom surface area (ft^2)
d_b = design depth (ft)
L = basin top length
W = basin top width
L_B = L-2Zd_b = bottom length
W_B = W-2Zd_b = bottom width
Z = side slope ratio (W:H)

Calculating Equation C5-S3- 2 and Equation C5-S3- 3 provides the following expression for the required bottom area of the basin (A_B):

Equation C5-S3- 4

$$A_B = \frac{2V_B - A_t d_b}{d_b - 2P + 2fT}$$

If a rectangular shape is used, the bottom length and width of the basin may be defined in terms of the top length and width as:

$$L_B = L - 2Zd_b \qquad W_B = W - 2Zd_b$$

Where:
Z is the specified side slope ratio (i.e., Z: 1)

Substituting the above relationships for L_B and W_B into Equation C5-S3- 4 provides an equation for the basin top length:

Equation C5-S3- 5

$$L = \frac{[V_B + Zd_b(W - 2Zd_b)]}{W(d_b - P)} - Zd_b{}^2$$

The solution of Equation C5-S3- 5 will be based on assuming an initial basin top width or a width set by the constraints of the site. The solution will iterative until the desired L/W ratio is achieved.

I. Procedures for infiltration basin design

1. **Step 1.** Determine the volume of water for storage using the methods for WQv, Cpv, or Q_p, summarized in Chapter 2, section 1 and Chapter 3, section 6.

2. **Step 2.** Compute the maximum allowable basin depth (d_{max}) from the feasibility equation, $d_{max} = fT_p$. Select the basin design depth (d_b) based on the depth that is the required depth above the seasonal groundwater table (4-foot minimum), or a depth less than or equal to d_{max}, whichever results in the smaller depth.

3. **Step 3.** Compute the basin surface area dimensions for the site soil type using C5-S3- 5. A long, narrow basin generally improves infiltration, and may influence the selection of a length-to-width ratio. A side slope steepness must be selected. An initial length or width of the basin is set, and the equation solved for the remaining dimension. If a rectangular shape is used, the basin top length (L_t) and width (W_t) must be greater than $2Zd_b$ for a feasible solution. If L_t and W_t are not greater than $2Zd_b$, the bottom dimensions would be less than or equal to zero. In this case, the basin depth (d_b) is increased for a feasible solution.

4. **The truncated hydrograph method for stormwater quantity management.** For local overbank flooding control (Q_p), the peak discharge for the post-developed hydrograph for a selected return period(s) should not exceed the peak discharge from the pre-developed hydrograph after development for stream channel erosion control and/or flood control purposes. In previous stormwater quantity management infiltration design methods, the difference between the pre-development and post-development runoff volumes was stored in the proposed infiltration structure. In most cases, this volume of runoff occurs prior to the actual hydrograph peak (see Figure C5-S3- 5), and therefore actual peak discharge control is not provided. Therefore, when considering an infiltration basin for peak discharge or stormwater quantity control, the truncated hydrograph method is used to determine the necessary infiltration storage volumes.

   The pre-development and post-development peak discharges are computed using NRCS WinTR-55 or WinTR-20 methodology. The time (T_2) at which the allowable discharge occurs on the receding limb of the post-development hydrograph, as shown in Figure C5-S3- 5, is determined from the NRCS methods. The volume of runoff under the post-development hydrograph and to the left of the allowable discharge at T_2 is the design storage volume (V).

   The computed infiltration storage volume, V, may be adjusted to account for the volume of water which ex-filtrates from the infiltration structure during the period of time required to fill the structure. The ex-filtration volume (V_e) is the product of the minimum soil infiltration rate (ft/hr), the filling time (hr), and the surface area of the infiltration practice. The filling time (T_f) of the infiltration practice may be determined directly from the post-development hydrograph, as shown in Figure C5-S3- 5. T_f is the difference between T_2, where the allowable discharge occurs on the recession limb, and the time, T_1, where the discharge value on the rising of the hydrograph is equal to the minimum infiltration discharge. The times T_1 and T_2 can be determined from the TR-20 output file after the WinTR-55 program scenario is run. The minimum discharge is equal to the minimum soil infiltration rate (expressed as ft/sec) times the surface area (ft^2) of the infiltration practice.

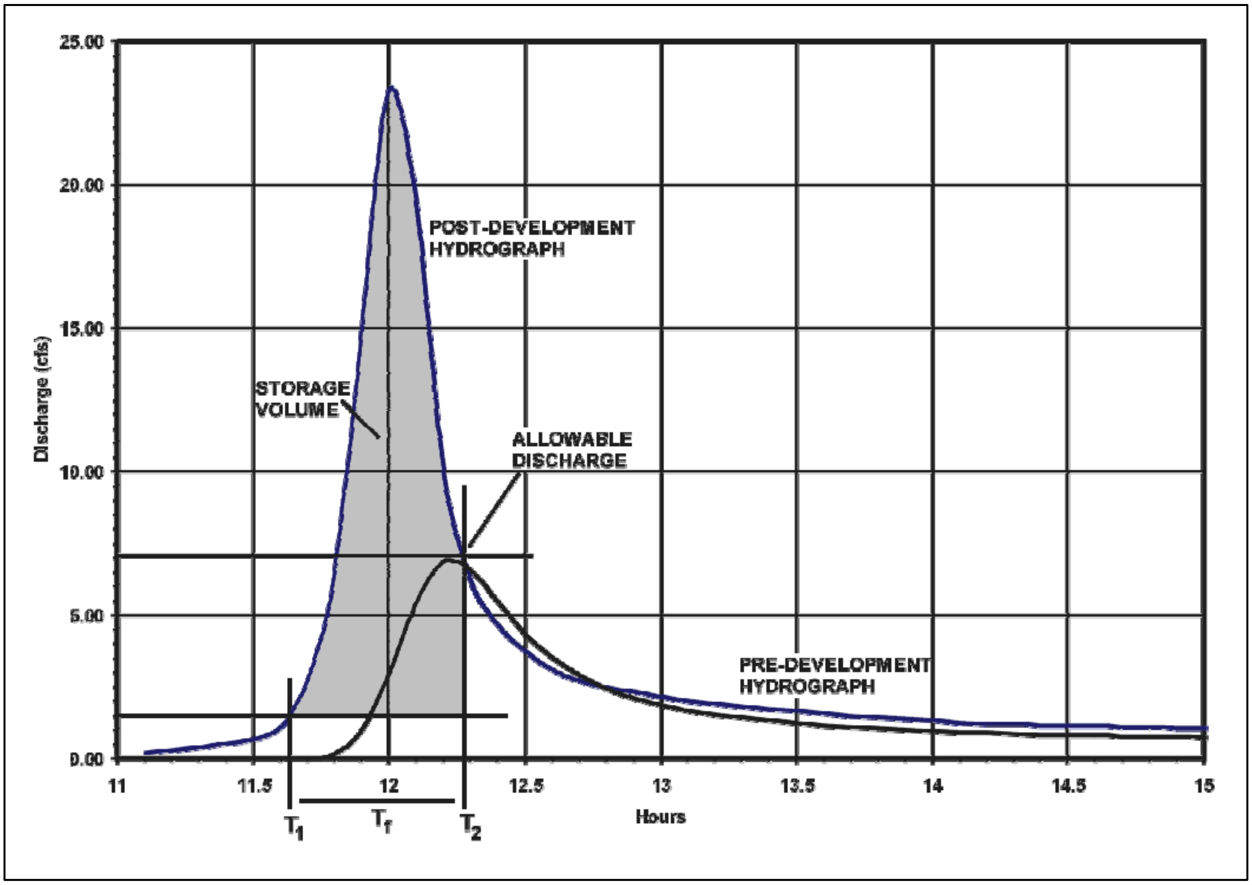


Figure C5-S3- 6: Truncated hydrograph method

J. Maintenance

Infiltration basins are relatively high-maintenance BMPs. Infiltration basins fail for one or more of the following reasons:

- Premature clogging with sediment
- A design infiltration rate greater than the actual infiltration rate
- The basin site was used for construction site erosion control (sediment trap or basin)
- Soil was compacted during construction
- The upland soils or the basin sides were not stabilized with vegetation, and excessive sediment was delivered to the basin

Consideration should be given to placing the basin into operation only after 90% of the upland development site has been built out and stabilized with vegetation. The other option is to strictly enforce construction site erosion controls during the development build-out process. If the basin was designed as an offline structure, bypass the structure as excessive sediment loads are being transported from the drainage areas during development.

The stormwater management plan includes maintenance, inspection, access, and enforcement of the operating requirements for the structure. The key elements of the plan are as follows:

1. An operation and maintenance plan should be prepared prior to placing the basin into operation.

2. Following construction, inspect the basin monthly, as well as after every major storm to ensure the basin is draining within the maximum drain time limit.

3. Inspect annually or semi-annually for settling, cracking, erosion, leakage, tree growth on the embankments, condition of the inlet and outlet channels, sediment accumulation in the basin bottom, and the condition of the grass turf.

4. If the basin has a sediment forebay, determine the degree of sediment accumulation and schedule a clean-out if necessary.

5. The basin should be mowed at least twice a year to prevent woody growth, stimulate grass growth, and enhance nutrient removal.

6. Do not mow when the ground is wet to avoid compaction of the bottom soils.

7. Remove trash and debris at least twice a year, or more often as necessary.

8. If the soils were marginal for infiltration and the basin is prone to extended ponding, periodic tilling of the basin bottom and re-seeding might be necessary. Till and re-vegetate in the early fall.

9. Over time, an infiltration basin will accumulate sediment, and the overall infiltration rate will diminish. Deep tilling, regrading, and replanting will help to restore the original infiltration performance. When the basin is thoroughly dry, remove the top cracked layer of sediment, and till and re-seed the remaining soil. Basins can be designed with a 6-12 inch layer of sand on the bottom or a filter fabric to facilitate removal.

10. If the sediment is accumulating faster than the growth of the turfgrass, the pre-treatment system needs to be re-evaluated. Maintenance of the pre-treatment system (sediment forebay, filter strip, grass) must occur on a regular basis to prevent heavy sediment build-up in the basin. The operating life of the pre-treatment system or inlet/bypass structure will likely be shorter than the infiltration basin, and will require occasional structural repair or equipment replacement.

K. Design example

Infiltration Basin

An infiltration basin is proposed for a development site northwest of Sioux City, IA. The site soil conditions have been investigated and found to be appropriate for an infiltration basin. The preliminary NRCS soil survey indicated HSG-B soils, which was confirmed with a series of soil borings to verify soil texture gradation, depth to groundwater, and soil bulk densities. The pre- development condition is undeveloped, and current land use is pasture and some wooded area. The total drainage area is 6 acres. The development is a residential subdivision with ½-acre parcels. The estimated impervious area after development will be 28%.

- Soils – 80% loam and 20% sandy loam (HSG-B)
- Nominal infiltration rate from soil survey analysis – 0.8 in/hr
- Depth to seasonal high groundwater – 12 feet
- Soil cores indicate a uniform loam/sandy loam to a depth of 15 feet
- Average of four in-situ soil permeability tests with a double ring infiltrometer indicating an average final infiltration rate of 0.96 in/hr. Initial infiltration rates averaged 2.3 in/hr.

Determine the bottom dimensions of an infiltration basin given the following design criteria:

- Design for drain time 72 hours
- Maximum ponding depth will be 24 inches for the WQv
- Design storm for WQv is 1.25 inches
- Additional storage volume will be provided in the infiltration/detention basin for Cpv (Calculations not included in this design example)
- The pre-development T_c was determined to be 0.42 hour
- The post-development T_c is estimated to be 0.2 hour at full build-out
- The goal for the L/W ratio for the basin is a minimum of 3:1

1. **Determine WQv:**

$$R_v = 0.05 + 0.009(28\%) = 0.302$$
$$WQv = R_v \times P = (0.302)(1.25 inches) = 0.38 inches$$
$$WQv = \left(\frac{0.38 inches}{12}\right) \times (43{,}560 ft^2/acre)(6 acre) = 8{,}276 ft^3$$

2. **Determine Rev - percent volume method:**

$$Rev = \frac{SR_vA}{12}$$

Where:
R_v = 0.302
A = site area in acres
S = soil-specific recharge factor (from HSG) = 0.34 (Table C2-S1- 4, for HSG-B soils)

$$Rev = \frac{(0.34)(0.302)(6ac)}{12} = 0.051ac - ft = 2{,}236ft^3 \; (considered\ part\ of\ the\ WQv)$$

For this site, the goal is to infiltrate the entire WQv so the Rev requirement will be met.

3. **Compute the maximum allowable depth, d_{max} (Equation C5-S3- 1):**

$$d_{max} = fT_p$$

d_{max} = 0.96 inches/hr x 72-hr = 69.12 inches = 5.76-ft

4. **Determine the required storage volume for the basin, V_B (Equation C5-S3- 4). Assume a trapezoidal basin with 3:1 side slopes. For this design, the design depth, d_b = 2 ft (24 inches), the desired basin L/W ratio is 3:1, and the desired basin top width (W) is 40 feet.**

5. **Desired design depth of 2 feet is less than d_{max}, or 5.7 feet.**

6. **Calculate storage volume required – Equation C5-S3- 2:**

$$V_B = WQv + \frac{PA_B}{12} - fTA_B$$
$$= 8{,}276ft^3 + \frac{(1.25in)WL}{12} - \frac{(0.96in/hr(2hr)}{12WL}$$
$$= 8{,}276ft^3 + \frac{(1.25in)(40ft)(L\ ft)}{12} - \frac{(0.96in/hr(2hr)}{12(40ft)(L\ ft)}$$
$$= 8{,}276ft^3 + 4.17(L)ft^3 - 6.4(L)ft^3 = 8{,}276ft^3 - (2.23ft^2)(L\ ft)$$

7. **Determine the length dimension for the basin using a W = 40 ft, Z = 3, and D_b = 2 feet:**

$$V_B = \left[\frac{L_W + L_BW_B}{2}\right]d_b$$

From Step 6 above:

$$V_B = 8{,}276ft^3 - (2.23ft^2)(L\ ft) = [(L\ ft)(40ft) + (40ft - 12ft)(L\ ft - 12ft)]/2 \times 2ft$$

$$8{,}276ft^3 - (2.23ft^2)(L\ ft) = [(40ft)(Lft) + (28ft)(L\ ft) - 336ft^2]/2\ \times 2ft$$

$$8{,}276ft^3 - (2.23ft^2)(L\ ft) = (68ft^2)(L\ ft) - 336ft^3$$

$$(70.23ft^2)(L\ ft) = 8{,}612ft^3$$

L = 122.6 ft ≈ **123 ft** (L/W = 3.07)

8. **Total surface area of basin, A_b = 123 ft x 40 ft = 4,920 ft^2**

9. **$A_B = L_B \times W_B = 111$ ft x 28 ft = 3,108 ft^2**

10. **Check drain time for f = 0.96 in/hr and ponding depth of 24 in:**

$T_p = d_b/f$ = 24 inches/0.96 in/hr = 25 hrs

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

Description: Bioretention systems incorporate shallow landscaped level depressions that temporarily store and readily infiltrate runoff. They include both rain gardens and bioretention cells. A rain garden relies solely on soils with good percolation rates. Bioretention cells typically include a rock chamber, subdrain, and modified soil mix. In bioretention cells, stormwater runoff collected in the upper layer of the system is filtered through the surface vegetation, mulch layer, pervious soil layer, and then stored temporarily in a stone aggregate base layer. The Water Quality Volume (WQv) is drained from the aggregate base by infiltration into the underlying soils and/or to an outlet through a perforated pipe subdrain. Systems can operate either off-line or online. They are designed with a combination of plants that may include grasses, flowering perennials, shrubs, or trees. Integrated upstream treatment is provided by a perimeter grass filter strip or grass swale for initial capture of sediment.

Typical uses:
- Manages water quality runoff volume from residential, commercial, and institutional sites.
- Drainage area for each cell is typically 0.5-2.0 acres. Larger drainage areas should be divided into smaller sub-areas with individual bioretention cells distributed throughout the site.
- Suitable for landscaped depressional areas such as parking lot islands, road medians, and street right- of-ways.

Advantages/benefits:
- Reduce runoff rate and volume from impervious areas; provide opportunity for infiltration and filtration processes. Good for highly-impervious areas, such as parking lots.
- Removes fine sediments, heavy metals, nutrients, bacteria, and organics. Reduces thermal pollution from runoff across pavement surfaces.
- Flexible design options for varying site conditions; subdrain system allows use on sites with limiting soils. Good retrofit opportunities.
- Flexible landscaping options can provide an aesthetic feature.

Disadvantages/limitations:
- High entrance velocities and concentrated flows may need special design considerations.
- High sediment loads can cause premature failure; upstream practice is needed.
- High water table may require special design considerations.

Maintenance requirements:
- Routine landscape maintenance - removal of undesirable and dead vegetation.
- Replenish mulch layer.
- Removal of accumulated sediment in pretreatment areas.

A. Overview

1. **Description.** Bioretention cells are structural stormwater controls. They capture and temporarily store the water quality volume using soils and vegetation in shallow basins or landscaped areas to remove pollutants from

stormwater runoff.

Bioretention cells use vegetation and engineered soils in a treatment area to accept runoff from impervious surfaces. Stormwater flows into the bioretention cell, temporarily ponds on the surface, and gradually infiltrates into the modified soil layer. Examples of bioretention cells are shown in Figure C5-S4- 1. Components of a bioretention cell are illustrated in Figure C5-S4- 2 and Figure C5-S4- 3. Bioretention cells are intended to replicate the stable hydrologic functions of a native ecosystem. Bioretention functions as a soil and plant-based filtration system for stormwater runoff, and removes pollutants through a variety of physical, chemical, and biological processes in the upper engineered soil layer and the underlying native soils. The design can impact the processes and their function. Some of the major processes that occur through bioretention include interception, infiltration, settling, evapo-transportation, filtration, absorption, thermal attenuation, and biological degradation/decomposition.

The filtered runoff can be allowed to either percolate into the underlying soils or be temporarily stored in the aggregate subdrain system and discharged at a controlled rate to the storm sewer system or a downstream open channel. Runoff can be controlled closer to where it is generated by the uniform distribution of bioretention cells to break up the area in manageable sub- watersheds. Higher flow events (> Q_2), and runoff volume that exceeds the infiltration capacity of these systems can be returned to the conveyance system or safely bypassed.

Plants in bioretention cells enhance infiltration and provide an evapotranspiration component. Native species provide resistance to moisture changes, insects, and disease and provide uptake of runoff water and pollutants. Deep-rooted native plants (grasses and forbs) are recommended to maintain high organic matter content in the soil matrix, provide high infiltration rates, and provide uptake of runoff water. The mulch layer and organic matter component of the soil matrix provide filtration and a place for beneficial microbial activity. Aerobic conditions are necessary to maintain microbial activity for processing pollutants.

There are many ways to incorporate bioretention cells into new construction projects or to retrofit existing urban areas. Bioretention can be used in residential yards, as interior or perimeter structures in parking lots, for rooftop drainage at residential and commercial building sites, along highways and roads, within larger landscaped pervious areas, and as landscaped islands in impervious or high-density environments.

A complementary upstream practice is provided to reduce the sediment loading to the bioretention cell. Bioretention cells are often built with grass filter strips around the bioretention area. These filter strips remove particulates and reduce runoff velocity. Filter strips also prevent crusting of pore spaces with fines and reduce maintenance. A freeboard storage area (temporary ponding) creates temporary storage for runoff prior to infiltration, evaporation, and uptake.

Each component of the bioretention cell is important. The engineered soil layer provides filtration and holds water and nutrients for the plants, enhances biological activity, encourages root growth, and provides storage of stormwater through the voids within the soil particles. The plant material evapo-transpires stormwater, creates pathways for percolation through the soil, improves soil structure, improves aesthetics, and reinforces long-term performance of subsurface percolation. Native plant material is recommended because of its deep root structure and ability to improve soil quality. The mulch layer acts as a filter for pollutants in runoff, protects underlying soil from drying and eroding, and provides an environment for microorganisms to degrade organic pollutants. It also provides a medium for biological growth, decomposition of organic material, and adsorption and bonding of heavy metals.

When bioretention cells are installed at locations such as gas stations or other sites where spills of hazardous materials could occur, the practices should be lined with an impermeable membrane. A shutoff valve should be installed at the lower end of the subdrain so that the materials can be contained within the practice and managed appropriately.

Mosquitoes are not a problem because bioretention cells do not retain standing water long enough for mosquito reproduction (4-10 days). Properly designed bioretention cells will infiltrate standing water within 4-12 hours.

(a) Cascading feature in two-stage bioretention cell.

(b) The grassed pretreatment area provides a long flow path to this bioretention cell. Curb cuts were installed after the young plants became established.

(c) Newly planted bioretention area with native grasses

(d) Terraced bioretention cells under construction.

Figure C5-S4- 1: Example bioretention applications

2. **Applications for stormwater management (stormwater management suitability).**
   Bioretention cells are designed primarily for stormwater quality in the removal of pollutants. Bioretention can provide limited runoff quantity control, particularly for smaller storm events. These facilities may sometimes be used to meet channel protection requirements on smaller sites. However, bioretention cells will typically need to be used in conjunction with another structural control to provide channel protection as well as overbank flood protection. It is important to ensure that a bioretention cell safely bypasses higher flows.

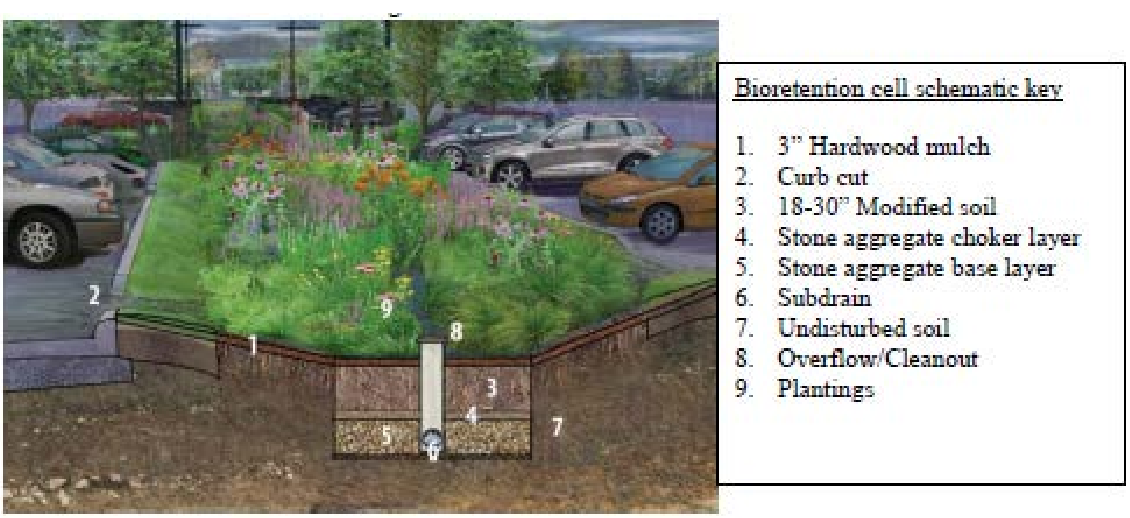


Figure C5-S4- 2: Bioretention cell schematic

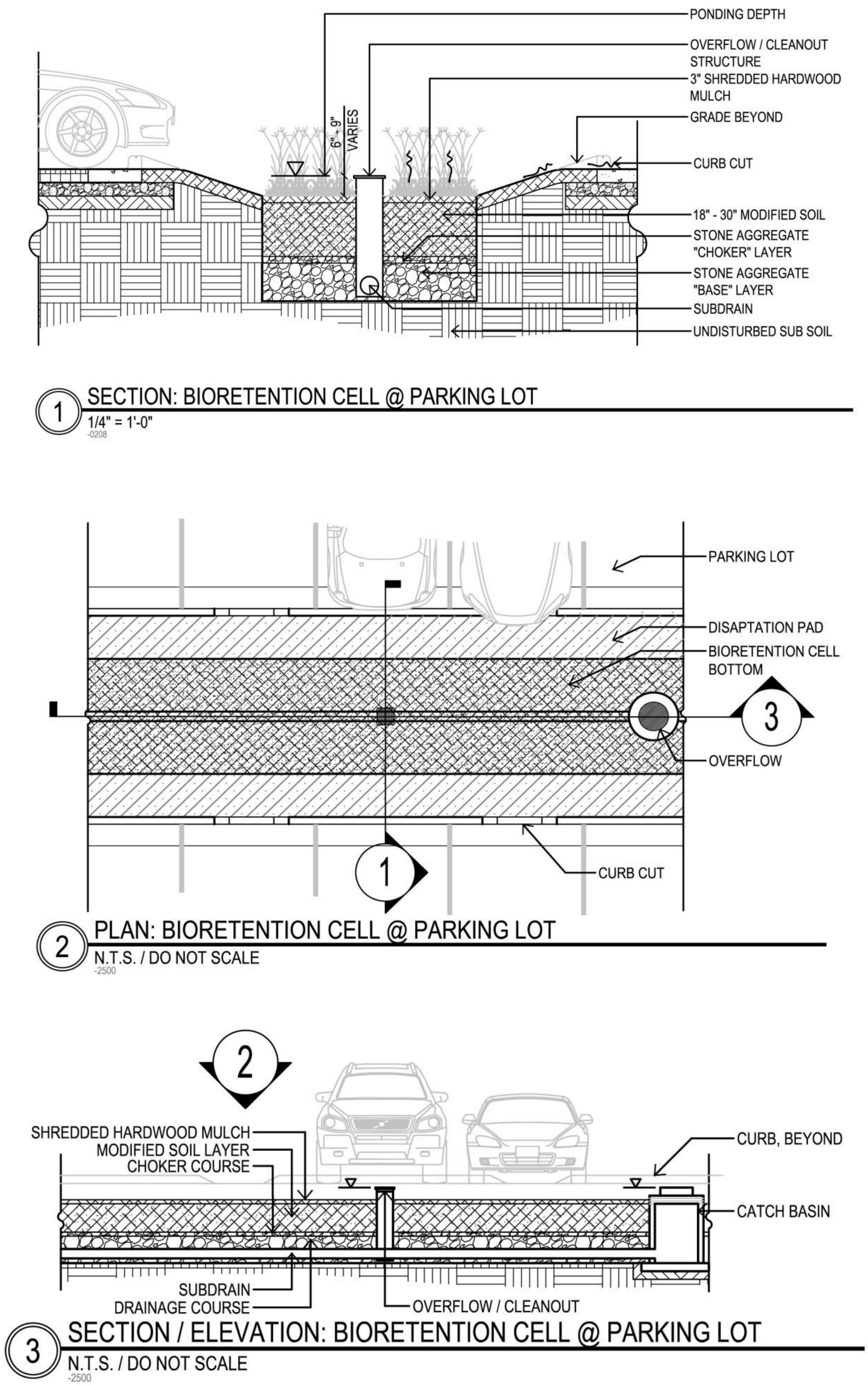


Figure C5-S4- 3: Cross-section and plan views of a bioretention cell

a. **Water quality.** Bioretention is an excellent stormwater treatment practice due to the variety of pollutant removal mechanisms. Each of the components of the bioretention cell is designed to perform a specific function (see Figure C5-S4- 3).
   1) Pretreatment practices reduce incoming runoff velocity and filter particulates from the runoff.
   2) The ponding area provides for temporary storage of stormwater runoff prior to its evaporation, infiltration, or uptake and provides additional pollutant settling capacity.
   3) The organic or mulch layer provides filtration, as well as an environment conducive to the growth of microorganisms that degrade hydrocarbons and organic material.
   4) The modified soil in the bioretention cell acts as a filtration system, and clay organic matter in the soil provides adsorption sites for hydrocarbons, heavy metals, nutrients, and other pollutants.
   5) Herbaceous and woody plants in the ponding area provide vegetative uptake of runoff and pollutants, and also serve to stabilize the surrounding soils, but will require maintenance such as trimming, pruning, and selective removal of volunteer species.
   6) Finally, an aggregate layer provides for positive drainage and aerobic conditions in the modified soil, and provides a final polishing treatment media.

b. Channel protection. For smaller sites, a bioretention cell may be designed to capture the entire channel protection volume in either an off-line or on-line configuration. The requirement of extended detention of the 1-year, 24-hour storm runoff volume can be achieved by increasing the footprint of the practice, or combining additional storage above the WQv ponding depth, with a slow release stage of an intake or other surface outlet structure. For off-line systems on larger sites, where only the WQv is diverted to the bioretention cell, another structural control must be used to provide Cpv extended detention.

c. Overbank flood protection*. Typically, another structural control must be used in conjunction with a bioretention cell to reduce the post-development peak flow of storms greater than the 5-year storm (Q_p) to pre-development levels (detention).

d. Extreme flood protection*. Bioretention cells must provide flow diversion and/or be designed to safely pass extreme storm flows and protect the ponding area, mulch layer, and vegetation.

*Refer to design procedures included in this section for more discussion of on- and offline systems as well as detention or attenuation of larger storm events.

(See Chapter 2, section 1 and Chapter 3, section 6 for more details on the Unified Sizing Criteria and Small Storm Hydrology)

3. Pollutant removal capabilities. In landscaped and residential areas, the major pollutants of concern are fertilizers such as nitrogen and phosphorus. The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes, derived from sampling data, modeling, and professional judgment (Table C5-S4- 1). 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. For additional information on monitoring BMP performance, see ASCE/EPA "Urban Stormwater BMP Performance Monitoring: A Guidance Manual for Meeting the National Stormwater BMP Database Requirements."

Table C5-S4- 1: Pollutant removal efficiency of bioretention cells

Parameter	Removal Efficiency (%) Median values (N=10)
Total suspended solids	59
Total phosphorous	5
Total nitrogen	46
Nitrogen NO_2-NO_3	43
Copper	81
Zinc	79

Source: CWP National Pollutant Performance Database, v3, ept., 2007

More information on pollutant removal capabilities for bioretention BMPs can be found in the National Stormwater Best Management Practices Database (http://www.bmpdatabase.org/) and the ASCE/EPA database.

The University of Maryland Engineering Department, completed an evaluation "Optimization of Bioretention," of the effectiveness of pollutant removal. The experiment yielded valuable data on pollutant removal efficiency rates and processes for bioretention. This manual incorporates those findings into the design criteria. Table C5-S4- 2 summarizes the efficiency removal rates for various pollutants.

Table C5-S4- 2: Cumulative percent removal of pollutants by depth in bioretention cells

Laboratory/Field Summary (%)								
Depth	Cu	Pb	Zn	P	TKN	NH_4	NO_3	TN
1 ft	90	93	87	0	37	54	-97	-29
2 ft	93	99	98	73	60	86	-194	0
3 ft	93	99	99	81	68	79	23	43

Source: Davis, A.P. et al, University of Maryland, 1998

4. **Application and feasibility.** Bioretention is suitable for a wide variety of development options, including commercial, high-density urban, and single-family residential areas. They can be used for new construction and also to retrofit urban landscapes. Their capacity to be used as a landscaped feature allows them to fit into many types of urban design. Bioretention cells are ideally suited to many ultra-urban areas, such as landscaped parking lot islands and along streets and boulevards. Ultra-urban areas are densely-developed urban areas in which little pervious surface exists. While they consume a fairly large amount of space (approximately 5%-10% of the impervious area that drains to them), they can fit into existing parking lot islands or other landscaped areas, when used as a standalone practice. They can also treat runoff from intensively managed areas that have the potential for pollutants, such as golf courses. Figure C5-S4- 3 includes an example site configuration.

   The following criteria should be evaluated to ensure the suitability of a bioretention cell for meeting stormwater management objectives on a site or development. Table C5-S4- 3 provides a list of considerations when planning for a bioretention cell.

   a. General feasibility:
      - Suitable for use in developed or developing areas, provided that heavy sediment loads are not expected in post-construction conditions (i.e. may not be suitable in watersheds with on-going site construction, routinely disturbed areas, agricultural lands without conservation practices, etc.). Suitable for use in brownfield projects and areas with pollutant hotspots. Special considerations are needed in areas with karst topography, loess soils, or high water tables.
      - Bioretention practices should be located where they are accessible to be maintained and where maintenance is assured by a designated responsible party.
      - Bioretention practices are not recommended to be used as a single large BMP (regional stormwater

control). Flow velocities may be too high near the entrance to the practice, and/or the required area for treatment would likely be too large to be expected to be constructed with a level bottom. Divide larger watersheds into multiple, smaller sub- areas for treatment or review other water quality BMPs that are better at managing larger drainage areas.

b. Physical feasibility – physical constraints at project site:
   - Drainage area: 0.5-2.0 acres of impervious area are preferred. Larger areas of imperviousness can be broken into smaller catchments.
   - Space required: Approximately 3% to 7% of the tributary impervious area is required.
   - Site slope: Special design considerations for sites with steep slopes.
   - Minimum head: Need sufficient elevation to allow subdrain system to daylight downstream, or connect to available storm sewer system.
   - Minimum depth to water table: A separation distance of 2 feet is recommended between the bottom of the bioretention cell and the elevation of the seasonally high water table.
   - Soils: No restrictions; engineered media required. Since modified soils and a subdrain system are included in bioretention cells, cells do not need to rely on the percolation rates of subsoil layers to function. However, locating cells in areas with higher percolation rates allows opportunities to reduce the volume of surface runoff from a site and provide groundwater recharge.

Table C5-S4- 3: Planning criteria for bioretention cells

Site Conditions	The bottom of the aggregate layer should have 2 feet of vertical separation from expected high groundwater elevations or bedrock layers.
Source of Runoff	Bioretention cells can be placed close to the source of runoff generation.
Distributed Placement and Location	It is preferred to consider stormwater management during initial site design. Several, smaller bioretention cells can treat more manageable amounts of runoff closer to its source. Use site grading to divert runoff to smaller depressions in open spaces such as parking islands, landscaped areas, etc.
Site Integration	Stormwater management site integration is a preferred alternative to end-of- pipe BMP design, where feasible.
Location	Bioretention cell locations should be integrated into the site planning process, and aesthetic considerations should be taken into account in their siting and design. Elevations must be carefully worked out to ensure that the desired runoff flow enters the facility with no more than the maximum design depth.
Drainage Area	Potential bioretention cells should be applied where impervious surfaces within drainage subareas to each cell are limited to less than 2 acres.
Online or Offline	Offline systems employ some type of diversion structure, which typically diverts the first flush of flow to the treatment practice, but allows flows from larger events to bypass the practice. This can prevent erosion within the practice and re-suspension of captured sediments. A cell is considered online if all runoff from the upstream area enters the practice.
Flow Diversion for Offline cells	When used in an offline configuration, the WQv (and perhaps Cpv) is diverted to the bioretention area using a flow splitter, diversion structure, and/or overflow outlet. Larger stormwater flows are diverted to other controls downstream (see Chapter 6, section 1, F for more discussion of offline systems and design guidance for diversion structures and flow splitters).
Intermittent Flow	Bioretention cells are designed for intermittent flow and must be allowed to drain and re-aerate between rainfall events. They should not be used on sites with a continuous flow from groundwater, heavy irrigation, sump pumps, or other sources.
Storm Events	Typically, bioretention cells are used to manage small storm events (this may include events smaller than the Water Quality event (WQv) or the Channel Protection event (Cpv-1-year event). Refer to Chapter 3, section 6 for additional information about small storms.

	Online systems may offer the possibility to attenuate or detain flows from larger storm events, with caution needs to prevent: • Erosive flow velocities near inlets/outlets • Deep ponding could compact soil layers • Extended drawdown periods that could affect desired plants

B. Design Methods

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

   Before choosing to employ a bioretention practice, review the feasibility information included earlier in this section. If feasible, proceed with designing a bioretention practice, starting with a review of the initial design considerations listed in Table C5-S4- 4, as well as the preliminary investigation information in Table C5-S4- 5.

Table C5-S4- 4: Initial design considerations

Limiting Factors	Determine the depth to bedrock or typical groundwater elevation. Verify with geotechnical explorations or other methods.
Watershed Concerns	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.
Separation Distances	Ensure that room is available for installation, including any setback and/or separation requirements. Recommended setbacks are 25 feet from the foundation of a building; 5 feet from a property line; 50 feet from a private well; 20 feet from a geothermal well field; 100 feet from a municipal well.
Intermittent Flows	Bioretention cells are designed for intermittent flow and must be allowed to drain and re-aerate between rainfall events. They should not be used on sites with a continuous flow from groundwater, sump pumps, heavy irrigation, or other sources.
Local Requirements	What are the local requirements for water quality and quantity control?
Character of Runoff Generator	What is the land use that will generate runoff directed to the cell? What is the expected watershed area, and how much of that is expected to be impervious?
Pollutants	What pollutants are expected to be present in runoff?
Multiple Practices	Will the bioretention cell be used independently of other BMPs, or will it be installed along with other practices? If part of a series of practices, what portion of the treatment required will be dedicated to each practice?
Online or Offline	Will the practice be an online or offline configuration? (See Chapter 4 for more discussion of offline systems and design guidance for diversion structures and flow splitters).
Quality Control	What design storm is required to meet stormwater water quality management criteria?
Quantity Control	For an online system is the cell being used for quantity control (or attenuation / detention) of a portion of larger storm events?
Aesthetics and Site Plans	Bioretention cell locations should be integrated into the site planning process, and aesthetic considerations should be taken into account in their siting and design.
Plant Materials	Native species are recommended that are tolerant of expected moisture conditions, as their deep root structures can help preserve percolation rates. Consider salt tolerance where its use in ice removal is expected.
Maintenance	Review a typical maintenance plan, and determine the parties responsible for carrying it out. Consider access paths for equipment required for maintenance. See Table C5-S4- 8.

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

Table C5-S4- 5: Preliminary investigations

Properties of the Drainage Area Tributary to the Bioretention Cell	Determine the expected drainage area to be routed to the bioretention cell and the projected amount of impervious surfaces. It is recommended that the **impervious** area to each cell not exceed 2 acres. Multiple cells can be designed to treat runoff from larger areas. Surface properties required to determine time of concentration will be needed for final design (refer to Chapter 1, section 4).
Space Required	The required temporary ponding area will be approximately 3-7% of the tributary **impervious** area. Most of the ponding area must be level, so remember that additional space will be needed for slope grading to establish the overflow elevation and match surrounding grades.
Slope	Cells are easier to construct away from steep slopes, but special elements such as retaining walls can be included for sites with steep slopes. Care must be taken not to compact the soils within the bioretention area during installation of any structural features around the cell.
Minimum Head	Make sure that there is sufficient elevation difference to pond water as needed and drain the soil and aggregate layers through a subdrain and/or outlet works to a finished surface, swale, or storm sewer system.
Water Table	A separation distance of 2 feet is recommended between the bottom of the bioretention cell and expected high groundwater levels.
Existing Site Soils	No restrictions when modified soils are used. However, soils with higher infiltration rates can be used to promote infiltration and groundwater recharge, reducing post-development surface runoff volumes.

2. **Typical Components of a Bioretention Cell.** Before proceeding with final design, it is important to understand the function and purpose of the elements that make up this type of practice. Table C5-S4- 6 provides a summary of bioretention cell components and their function.

Table C5-S4- 6: Bioretention cell design components

Inlet Structures	Stormwater may be routed to bioretention cells in many ways, such as sheet flow off hard surfaces, or as concentrated flow from curb openings, downspouts, and pipe outlets. Inlet structures may also include features that divert only a portion of stormwater runoff to the cell (known as an offline configuration). Level spreaders can be used to disperse concentrated flows to sheet flows reducing flow depths and velocities, enhancing pretreatment possibilities.
Pretreatment Area	These areas are used to reduce velocities and capture heavier sediment and debris. They are needed to reduce the potential of clogging the porous soils desired in the cell. Features such as grass filter strips and swales can remove sediment and debris through filtration. Mechanical treatment systems, sediment traps, and ditch checks can be used to pond water in areas where heavier sediment particles can settle out. If designed and functioning properly, all of these pretreatment practices will capture sediment and debris, so provisions for regular maintenance and removal of collected materials is required.
Temporary Ponding Area	Provides for temporary surface storage of the runoff before it infiltrates into the soil bed. Typically limited to a depth of 6-9 inches. Additional freeboard depth can be provided for larger storm events in online systems. The ponding area is intended to drain dry within 4-12 hours after typical storm events, and should never have standing water longer than 24 hours after very rare events.
Organic Mulch Layer	The mulch layer should consist of 3 inches of fine, shredded hardwood mulch. This layer protects the soil bed from erosion, retains moisture in the plant root zone, provides a medium for biological growth and decomposition of organic matter, provides some filtration of larger sediment particles, and controls weeds.
Modified Soil Layer	The modified soil layer filters stormwater. Typically this layer is 18-30 inches deep and consists of a uniform mixture of 75-90% washed concrete sand, 0-10% approved organic material, 0-25% soil with a soil texture that includes A-horizon characteristics and meets specifications.
Choker Aggregate Layer	The choker layer separates the modified soil layer and aggregate subbase and prevents the modified soil from entering into the aggregate subbase. The 2-3 inch layer consists of clean, durable ⅜ inch diameter chip.
Stone Aggregate Subbase Layer	The aggregate layer at the bottom of the structure provides additional temporary storage capacity for the captured runoff after filtration. The layer consists of an open-graded, clean, durable aggregate of 1-2 inches diameter with a porosity of 35-40%.
Subdrain	Perforated pipe underdrains are recommended. They provide the outlet for filtered water in areas with soils with poor percolation rates and act as a secondary outlet where soil percolation rates are better.
Outlet Structures	To avoid excessive ponding depths and drawdown times, outlet controls are needed to manage runoff from larger storm events. An overflow spillway set above the ponding depth can release flows in a non-erosive manner (velocities less than 3 feet per second). For online configurations, riser pipes, intakes, or weirs may be used to release runoff from larger storms more rapidly than it could infiltrate through the soil layers.
Hydrologic Design	The primary goal of the practice is to capture and treat runoff from the WQv event. For offline configurations, the majority of flows larger than those generated by the WQv will bypass the bioretention cell. For online configurations, where all runoff passes through the cell, some level of “detention” or temporary storage of larger flows may be possible, with caution to avoid excessive storage depths or drawdown periods that could compact the soil layers within the bioretention cell.

3. **Bioretention Cell Sizing and Design Calculations.** The following design procedure, assumes that the designer has completed preliminary investigations, and understands the design components of a bioretention cell, as outlined in Table C5-S4- 5 through Table C5-S4- 7. It is recommended that these calculations be completed as early as possible in the design process, so that adequate room is reserved for stormwater management as site

design development continues. Calculations can be adjusted as final site design is completed. (Note: “!” = pay special attention)

Step 1: Compute the required WQv treatment volume

Refer to Chapter 3, section 6 for additional details on Small Storm Hydrology.

Use the following information:

(DA) = Drainage area to be treated, in acres
(I) = Impervious cover of drainage area, in %
(P) = WQ event rainfall depth, in inches (recommend using 1.25” for Iowa)

Step 1a: Compute (R_v) = 0.05 + 0.009(I)
i.e. 75% impervious => (I) = 75

Step 1b: Compute WQv = (R_v) x (P) x (DA) x 43,560 SF/ac x (1 ft/12in)
WQv is calculated in ***cubic feet***

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

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

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

Use method outlined in Chapter 3, section 6, C, to compute the peak rate of flow (in cubic feet per second) and volume of stormwater runoff (in cubic feet) for the Channel Protection Volume (Cpv).

Use methods such as the NRCS TR-20, TR-55 (Chapter 3, section 7) or other acceptable methods to generate hydrographs to determine peak rates of flow (in cubic feet per second) and runoff volumes (in cubic feet) for the following events:

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

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

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

Step 3: Identify if the bioretention system is intended to be an online or offline system.

- *If planning for an online system, there is no need to design a flow diversion structure; proceed to Step 4.*
- *If planning for an offline system, a diversion weir, flow splitter, or other practice needs to be designed to route flows from the WQ event to the bioretention cell, while allowing most of the flows from larger events to bypass the system (via parallel storm sewer system or other conveyance). Refer to Chapter 6, section 1, F for additional design information. Include calculation details for the diversion structure with this design procedure.*

Step 4: Select, Locate, and Size Pretreatment Practice(s).

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

- *For grass swales, refer to Chapter 9, section 2, E for general sizing requirements. The target flow velocity for water quality treatment is 1 fps during the WQv event. Chapter 9, section 2 includes methods on how to modify the value of "n" for Manning's equation to evaluate shallow flow in grass swales.*
- *For filter strips, refer to Chapter 9, section 4, C, 4 for sizing requirements.*
- *Forebays should have a storage volume of 0.1 inches per impervious acre drained (Chapter 3, section 11). Sediment will need to be mechanically removed from the forebay over time, so a depth marker and durable, solid materials are recommended for the bottom (to be certain when excavation is complete). The volume of WQv to be used to size the ponding area of the bioretention cell can be reduced by the amount addressed in the pretreatment area(s) (typically no more than 10% of WQv).*

Step 5: Review Entrance Designs

To reduce the potential for surface erosion or displacement of mulch and planting materials, it is recommended that flow velocities entering the ponding area should not exceed 3 feet per second (for all storm events reviewed). For online systems, the peak velocity of flow entering the cell during the largest Q_f event (1% AR) should be checked. Redesign the cross-section of the entrance as needed. Provide stabilization at pipe outlets and areas of rapid expansion as necessary (USDOT FHA HEC-14 is a recommended resource for energy dissipater design).

Step 6: Select Desired WQ Event Ponding Depth

A WQv ponding depth of 6-9 inches should be planned over the level bottom of the bioretention cell. The bioretention cell will need an overflow spillway, or staged outlet structure (set above the WQv ponding depth) to avoid excessive ponding during larger storms. More detail is included in *Step 11*.

Step 7: Design Cross-Sectional Elements

A 3 inch depth layer of <u>fine</u>, shredded hardwood mulch is recommended to prevent erosion, retain moisture for plants, and control weeds.

The modified soil layer should be 18-30 inches deep and consist of a uniform mixture of 75-90% washed concrete sand, 0-10% approved organic material, 0-25% soil with a soil texture that includes A-horizon characteristics and meets specifications.

- *The greater depths of modified soil (24-30 inches) are usually considered when trees or shrubs are planned within the bioretention cell or extended filtration time is required to remove certain types of pollutants are determined to be necessary. This would be determined by a known pollutant source or watershed based removal goal.*

The aggregate layer is recommended to be at least 12 inches deep. Material should be 1-2 inch clean aggregate. The aggregate layer should have a porosity of 35-40%.

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

Step 8: Calculate the Recommended Footprint of WQ Ponding Area

The footprint area for temporary ponding of the WQv can be determined by the following equation:

$$A_f = \frac{WQv \times d_f}{\left[k\left(h_f + d_f\right)t_f\right]}$$

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

Where:
WQv = Water Quality Volume, in cubic feet (from *Step 1*)
d_f = filter bed layer depth, in feet (from *Step 7*, includes mulch, soil and aggregate)
h_f = average WQv ponding depth, in feet (value from *Step 6*, divided by 2)
t_f = desired time to drain modified soil layer, in days (recommend to use 1 day)
k = coefficient of permeability, in feet/day

- If the modified soil mix described in *Step 7* is used, use a value of 2 feet/day.
- After solving for the required ponded area, check to see if it falls in the range of 3-7% of the impervious area that drains to it.
- If existing soils have permeability rates of greater than 1 inch/hour, and the cell can be constructed in a manner to prevent compaction of such soils, the modified soil layer may not be needed. In this case, the permeability rate of site soils can be used for the value of (k). However, this is usually the case only for Hydrologic Group A soils and designers are cautioned to not over-estimate the permeability of existing soils.

Step 9: Design Surface Geometry of WQv Ponding Area

The bottom of the ponding area should be level, typically ranging from 10-30 feet in width. The cell should typically be at least two times longer than it is wide, as measured along the direction of flow (longer flow paths through the system increase filtration and percolation). Length to width ratios may not be applicable when runoff enters the cell along the side via sheet flow through a pretreatment vegetative strip, or if multiple concentrated entry points are used to distribute flow entry across the cell. [For concentrated inflow points, refer back to *Steps 4* and *5* to provide proper pretreatment at entry points and to reduce the potential of local erosion within the ponding area.] Non-uniform shapes fitted into the contours of the finished landscape may be more aesthetically pleasing, where possible.

- *Minimum widths are established to ensure that side slopes don't encroach into the level bottom. Minimum widths do not need to apply near the extreme ends of the ponding area. Maximum widths are required to allow the cell to be constructed from the edges (no heavy equipment placed on excavated subsoils), and a true level bottom is maintained. Cells that are too large may be not be truly level, leading to low points where runoff collects, minimizing the real area dedicated to infiltration.*
- *If you cannot reach the required ponding area (A_f) from Step 8 using the dimensions above, it is recommended to use multiple bioretention cells or use other water quality BMPs to treat the remaining volume. Bioretention cells can be used in series or parallel.*

Grades around the perimeter of the cell are recommended to be 6:1 or flatter; however slopes may be steeped to 3:1 where space is limited. Review the need for adequate sediment and erosion controls on steeper slopes to prevent side slope erosion into the modified soil layer (turf reinforcement mats, wattles, or sod are examples of practices that could be employed for surface stabilization).

- *After preparing a preliminary grading plan for the bioretention area, double check to make sure that the area ponded to the desired depth is greater than or equal to (A_f).*

Step 10: Subdrain System Design

For a bioretention cell, the subdrain system is needed to drain the aggregate layer over a 24 hour period. The design flow rate can be determined from the following equation:

$$Q = kA_f\left(\frac{1day}{24hours}\right)\left(\frac{1hour}{3{,}600sec}\right)$$

Solve for (Q) = Average subdrain flow rate (in cubic feet per second)

Where:
A_f = Required ponding area to treat WQv, in square feet (from *Step 8*)
k = coefficient of permeability, in feet/day (from *Step 8*, based on modified soil - minimum k)

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

- *Subdrain materials should comply with requirements for Type 1 Subdrains under SUDAS Specifications Section 4040. A minimum size of 8 inches is recommended for cleaning and inspection.*
- *The length of pipe should be determined, so that the area within 1 foot either side of the subdrain is at least 10% of the required ponding area (A_f). (i.e. A cell with a ponding area of 1,000 SF would need (1,000 SF) x 10% / (1 feet x 2) = 50 feet of subdrain.)*
- *Subdrains should be installed at least 3 inches above the bottom of the aggregate layer. Note that the portion of the aggregate layer below the invert of the subdrain can only be drained through infiltration into the native soils below; refer to notes within Step 7.*

Step 11: Staged Outlet Design for Online Systems

Offline systems may not need a staged outlet structure, as flows to the bioretention cell are limited at the inlet of the system. Review the rest of this step, and then proceed to *Step 12* if warranted.

Online systems will receive flows from larger storms, which will pond water to depths greater than those selected in *Step 6*. Without other means of release, all water diverted to the bioretention cell would need to filter through the soil and aggregate layers. To prevent excessive ponding depths and long drawdown periods, a staged outlet is necessary to release larger storms more quickly.

Inlet structures, riser pipes, weirs, or stabilized spillways are options for features that can be used as a second stage for controlled release of stormwater runoff. It is recommended to set an opening for the second stage at or just above the desired maximum WQv ponding depth. Refer to Chapter 3, section 12 on how to correctly size the selected type of control structure.

- *For online systems, it is recommended to complete a stage-storage model of the basin created above the bioretention cell with inflow hydrographs generated in Step 2 to determine storage volumes, depths, and release rates for all relevant storm events. To prevent compaction of the modified soil layer, excessive storage depths and drawdown times should be avoided. For the Cpv, check that ponding depths above the soil layer do not exceed 24 inches and surface drawdown does not exceed 24 hours. For the Q_p-Q_f events, check that ponding depths above the soil layer do not exceed 48 inches and surface drawdown does not exceed 30 hours.*

Step 12: System Outlet and Overland Spillway Design Considerations

Check peak flow velocities near pipe outlets and spillways expected to be overtopped during large storms. For all storm events reviewed, velocities at any pipe outlets should be less than 5 feet per second, and stabilization provided (refer to HEC-14). Overflow spillways should be designed with sufficient width to keep velocities less than 5 feet per second, and be properly stabilized or reinforced to withstand such velocities. Refer to Chapter 3, section 12, H for additional information.

Calculation Example

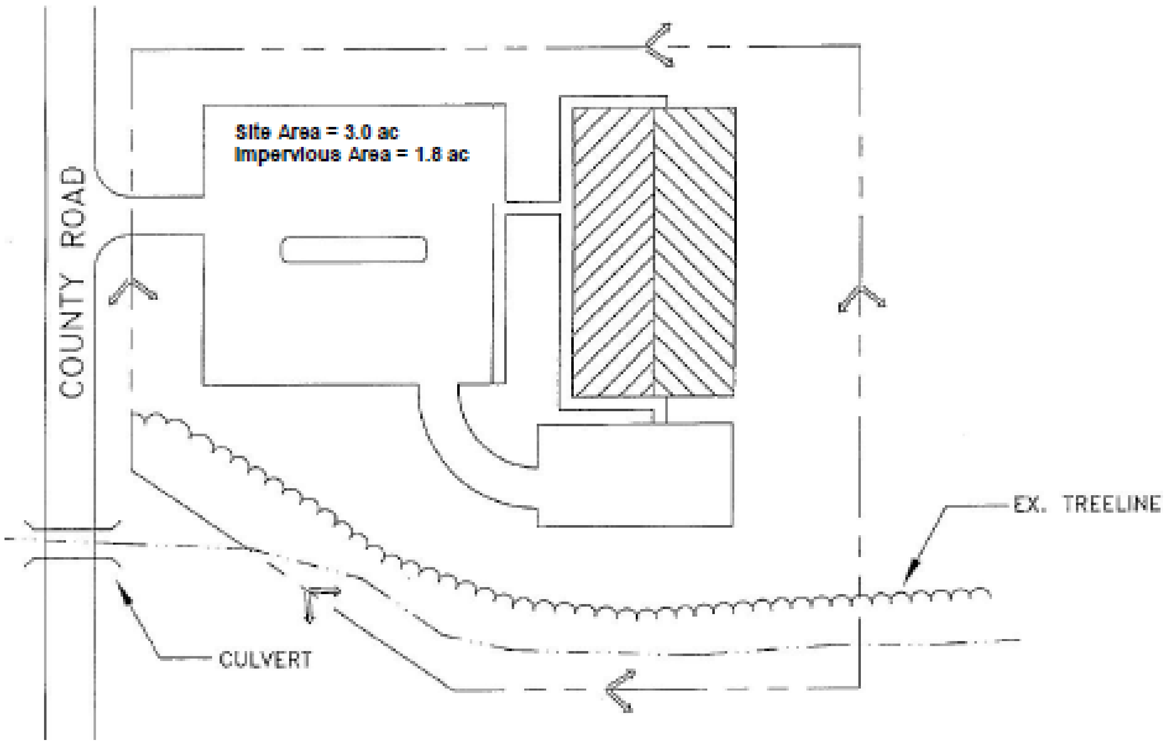


Figure C5-S4- 4: Recreation Center

Table C5-S4- 7: Site data

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

Step 1: Compute the required WQv treatment volume

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

Step 1a: Compute $\boldsymbol{R_v = 0.05 + 0.009(I)}$

$= 0.05 + 0.009(60) = 0.59$

Step 1b: Compute $\boldsymbol{WQv = R_v(P)(DA)(43,560\ sf/ac)\left(\frac{1ft}{12in}\right)}$

$= (0.59)(1.25")(3ac)(43{,}560\ sf/ac)\left(\frac{1ft}{12in}\right)$
$= 8{,}031\ cubic\ feet$

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

Use method outlined in Chapter 3, section 6, C, to compute the peak rate of flow (in cubic feet per second) and volume of stormwater runoff (in cubic feet) for the Channel Protection Volume (Cpv).

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

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

Condition	CN	T_c minutes	Peak rate cfs	Volume watershed inches	Volume cubic feet
Pre-developed	58	25	0.06	0.10	1,100
Post-developed	88	10	5.5	1.3	14,400

Use methods such as the NRCS TR-20, TR-55 (Chapter 3, section 7) or other acceptable methods to generate hydrographs to determine peak rates of flow (in cubic feet per second) and runoff volumes (in cubic feet) for the following events:

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

Step 3: Identify if the bioretention system is intended to be an online or offline system.

This facility is planned to be an offline system. To size the diversion structure, we need to calculate peak rates of flow (in cubic feet per second) and runoff volumes (in cubic feet) for the WQv event. For this example, we will complete TR-55 calculations, using adjusted curve numbers (CNs) for this small event. Refer to Chapter 3, section 6 for additional information.

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

- *Using an adjusted CN value, the volume of runoff from this calculation should be close to the value of WQv calculated in Step 1. (8,031 CF ≈ 8,035 CF)*

Assume for this example that runoff is directed from the site through a pipe to a manhole where the diversion weir will be placed. Refer to Chapter 6, section 1, F for additional design information.

Step 3a: Size outlet pipe to bioretention practice

- *To reduce the potential for erosion, it is recommended to have outlet velocities of less than 10 fps at the pipe outlet. Try a 10 inch outlet pipe (Area of pipe = 0.545 square feet).*

Rearranged continuity equation: $V = \frac{Q}{A} = \frac{3.1cfs}{0.545sf} = 5.7fps < 10fps$

Step 3b: Set diversion weir elevation

Determine the head required to divert all flow from the WQv event toward the practice. Use the orifice equation:

$Q = CA(2gh)^{0.5}\ where\ C = 0.60\ and\ g = 32.2\ ft/s^2$

Rearranged:

$$h = \frac{\left(\frac{Q}{CA}\right)^2}{2g}$$

$$h = \frac{\left[\frac{3.1cfs}{0.60 \times 0.545sf}\right]^2}{2 \times 32.2\,ft/s^2}$$

$$h = 1.40ft$$

The top elevation of the weir should be set 1.40 feet above the center of the 10 inch outlet pipe (or 1.82 feet above the flowline of the 10 inch outlet pipe).

Step 3c: Set diversion weir width

The width of the weir needs to fit within the diversion structure, allowing most of the flows that exceed WQv to bypass the system. Best to check this using the largest storm that the pipe is expected to handle. In many cases, this may be a 5- to 10-year event. In this example, we will use the 10-year event, and assume that larger storms surcharge the storm system and flow overland on a path that will bypass the bioretention practice.

- *If surcharge flows are directed toward the practice, then the system should be designed as an online system as a diversion structure will fail to route large storms around the practice.*

Use the weir equation: $QCLh^{3/2}\ where\ C = 3.33$

Assume L = 4 feet (weir is to fit within a standard manhole diameter)

Use Q = peak runoff from 10-year event from Step 2 - WQv event peak flow
= 13 cfs - 3.1 cfs = 9.9 cfs

Rearranged:

$$h = \left(\frac{Q}{CL}\right)^{2/3}$$

$$h = \left[\frac{9.9cfs}{3.33 \times 4ft}\right]^{2/3}$$

$$h = 0.82ft$$

This is the expected high water level above the top of the weir crest, inside the 4 foot diameter manhole during a 10-year storm event.

Step 3d: Double check flow through the diversion pipe to the practice during the maximum storm event, to avoid overloading the practice.

- *It is best to double check the flow through the outlet pipe to the bioretention area, to calculate the maximum expected peak flow to the practice.*

$$Q = CA(2gh)^{0.5}$$

$$Q = 0.60 \times 0.545sf \times \{[2 \times 32.2\,ft/s^2 \times (0.82ft + 1.40ft)]\}^{0.5}$$

$$Q = 3.90cfs$$

During the 10-year event (4.27 inches in 24-hour for Central Iowa) flow to the practice only increases about 0.8 cfs (25%) over the WQv design flow, meaning at least 9.1 cfs would bypass the practice (70% of the peak flow). This appears to be acceptable.

Step 4: Select, Locate, and Size Pretreatment Practice(s).

Alternatives to evaluate for pretreatment are:

For grass swales, refer to Chapter 9, section 2, E for sizing requirements.

Using a site imperviousness of 60%, and a slope of less than 2%; a 45 foot long, 2 foot wide swale is needed to meet pretreatment requirements.

If the 10 inch discharge pipe is connected to a level spreader to convert concentrated flow to sheet flow, a filter strip could be used. For filter strips, refer to Chapter 9, section 4, C, 4 for sizing requirements. The chart uses a maximum inflow approach length for impervious areas of 75 feet. To have an equivalent impervious approach length maximum of 75 feet, the 1.8 acres (78,408 square feet) of impervious surfaces in this example needs to be spread over a width of 1,045 feet (= 78,408 sf/75 feet). Providing this length does not seem feasible. A filter strip might be a better option with a level spreader in a smaller watershed area, or as an online system receiving sheet flow runoff from paved areas that are less than 75 feet in length.

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

Storage required:

$$= DA\left(\frac{I}{100}\right)(0.1in)\left(\frac{1'}{12"}\right)(43{,}560\, sf/ac)$$

$$= (3ac)\left(\frac{60}{100}\right)(0.1in)\left(\frac{1'}{12"}\right)(43{,}560\, sf/ac)$$

$$= 653cf\ (or\ 8\%\ of\ WQv)$$

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

A combination of practices could also be considered to meet pretreatment requirements, with each practice meeting a certain portion of the requirement. For this example, it is assumed that only the grass swale option will be chosen.

Assume a 4 foot wide swale is used (larger than required, but easier to construct) that is 45 feet long and has a longitudinal slope of 1.5% and side slopes of 4:1.

- *It is recommended to double check that the maximum flow velocity for water quality treatment of 1 fps is met during the WQv event. The methods described in Chapter 9, section 2 can be used modify Manning's equation to evaluate shallow flow in grass swales.*

An iterative procedure, spreadsheets or analysis software may be used.
For the channel section selected and an estimated depth of flow of 7 inches (0.583 feet):

Manning's coefficient (n) = 0.105 Area = 3.67 SF Wetted Perimeter = 8.78 feet Velocity = 0.97 fps (< 1.0 fps)
Q = 3.5 cfs (> WQv = 3.1 cfs)

- *If WQv velocity > 1.0 fps, try widening the swale, or decrease the longitudinal slope.*

Step 5: Review Entrance Designs

For larger events, solve the Manning's equation at the end of the pretreatment swale selected in *Step 4*. Again, an iterative procedure, spreadsheets or analysis software may be used.

For the channel section selected in *Step 4*, and at a depth of flow of 7.5 inches (0.625 feet):

Manning's coefficient (n) = 0.098 Area = 4.06 SF Wetted Perimeter = 9.15 feet Velocity = 1.08 fps (< 3.0 fps)
Q = 4.4 cfs (> WQv = 3.9 cfs)

Step 6: Select Desired WQ Event Ponding Depth

A WQv ponding depth of 6 inches (0.50 feet) has been selected for this example.

Step 7: Design Cross-Sectional Elements

Use the following:

- A 3 inch depth layer of fine shredded hardwood mulch.
- A modified soil layer should be 18 inches deep.
- A stone aggregate layer is recommended of 12 inches deep.
- Total depth = 0.25 + 1.50 + 1.00 = 2.75 feet

Step 8: Calculate the Recommended Footprint of WQ Ponding Area

The footprint area for temporary ponding of the WQv can be determined by the following equation:

$$A_f = \frac{WQv \times d_f}{\left[k\left(h_f + d_f\right)t_f\right]}$$

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

Where:
WQv = 8,031 cubic feet (from *Step 1*)
d_f = 2.75 feet (from *Step 7*)
h_f = 0.50 feet / 2 = 0.25 feet (value from *Step 6*, divided by 2)
t_f = 1 day (recommended drain time of soil layer for WQv event)
k = 2 feet/day (used recommended modified soil mix)

$$A_f = \frac{[8{,}031cf \times 2.75ft]}{[2\,ft/day \times (0.25ft + 2.75ft) \times 1day]}$$

$$A_f = \frac{22{,}085cf \times ft}{6.00sf}$$

$A_f = 3{,}681sf(4.7\%\ of\ impervious\ area)$ Recommend to round up to 3,700 square feet

Step 9: Design Surface Geometry of WQv Ponding Area

The bottom of the ponding area should be level, typically ranging from 10-30 feet in width. The cell should typically be at least two times longer than it is wide, as measured along the direction of flow (longer flowpaths through the system increase filtration and percolation).

Start with a cell twice as long as wide:

$$L = 2W$$
$$WL = 3{,}700sf$$
$$W \times 2 \times W = 3{,}700sf$$
$$W^2 = 1850sf$$
$$W = 43.0ft$$

Preliminary rough dimensions: Width = 43 feet, Length = 86 feet.
Check minimum and maximum widths, maybe adjust to: Width = 25 feet, Length = 148 feet.

Step 10: Subdrain System Design

For a bioretention cell, the subdrain system is needed to drain the aggregate layer over a 24-hour period. The design flow rate can be determined from the following equation:

$$Q(in\ cfs) = kA_f\left(\frac{1day}{24hours}\right)\left(\frac{1hour}{3{,}600\text{sec}}\right)$$

Where:
(A_f) = 3,700 square feet (from *Step 8*)

(k) = 2 feet/day (from *Step 8*, based on modified soil - minimum k)

$$Q = \left(2\frac{feet}{day}\right)(3{,}700sf)\left(\frac{1day}{24hour}\right)\left(\frac{1hour}{3{,}600\text{sec}}\right)$$

$$Q = 0.09cfs$$

The minimum recommended diameter of 8 inches will have sufficient capacity.

The length of pipe should be determined, so that the area within 1 foot either side of the subdrain is at least 10% of the required ponding area (A_f).

Length of subdrain = $A_f \times 10\%/(1ft \times 2)$

$$3{,}700sf \times 10\%/(1ft \times 2) = 185.0ft$$

Use at least 185 feet of 8-inch subdrain, set 3 inches above the bottom of the aggregate layer. Either the cell dimensions will need to be changed to be at least 185 feet long (i.e. 20’ x 185’ = 3,700 CF) and a single run of subdrain used, or parallel/perpendicular runs of subdrain will be needed to get to 185 feet of subdrain length (i.e. two parallel runs of 93 feet each). The upstream end of each subdrain should have a cleanout, extended to the surface for maintenance.

For this example, adjust ponding area size to 20 feet wide by 185 feet long, with a single 185 foot long subdrain and cleanout.

Step 11: Staged Outlet Design for Online Systems

This calculation is an example of an offline system. A staged outlet structure would not be needed in this case. However, an overflow spillway should be provided to prevent ponding deeper than the desired ponding depths. Referring to Chapter 3, section 12, design an earthen spillway to crest 9 inches above the level surface of the mulch layer. The spillway should have a minimum bottom width of 10 feet, a section depth of 2 feet, side slopes of 3:1 or flatter and longitudinal slopes ranging from 1-10%. Steeper slopes may require additional stabilization measures.

Step 12: System Outlet and Overland Spillway Design Considerations

This example is an offline system, with the subdrain discharging to a storm sewer system. If the subdrain did daylight, flows of 0.09 cfs would likely require minimal erosion protection.

For an online system, check exit velocities at pipe outlets and overflow spillways.

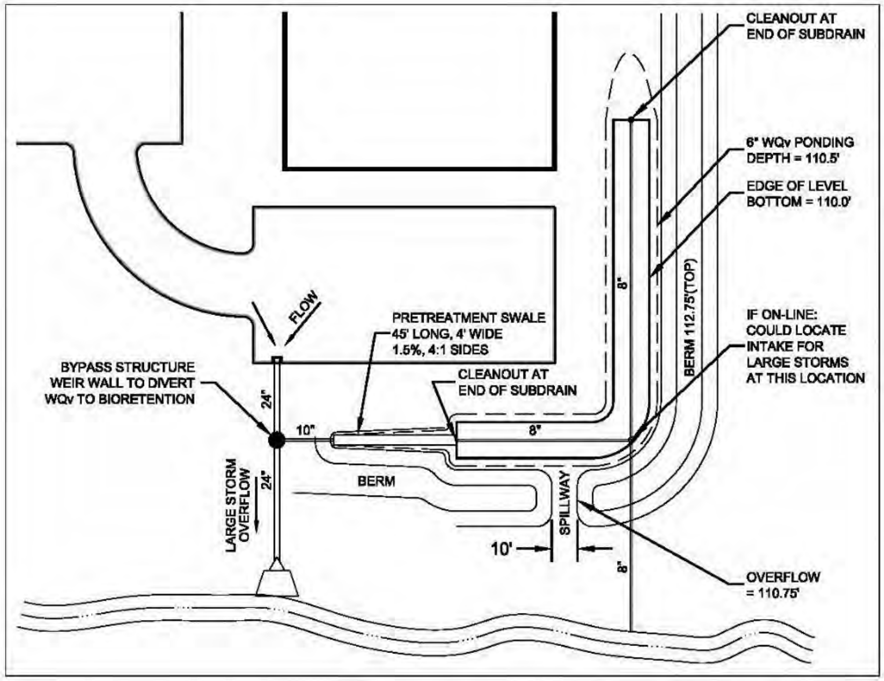


Figure C5-S4- 5: Site plan for the bioretention cell

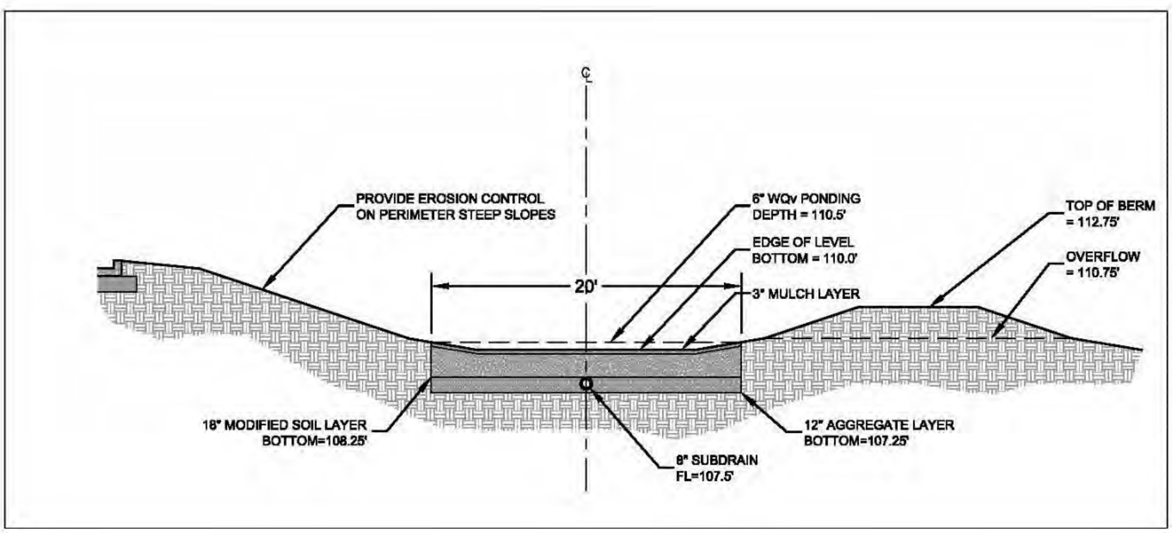


Figure C5-S4- 6: Section view of bioretention cell

C. Construction

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

2. **Staging.** The construction project should be staged so the bioretention cell is installed during the final construction stage. Prior to bioretention cell installation, all soils within the area that will drain to the bioretention cell must be stabilized with permanent vegetation and/or other erosion, sediment, and velocity controls. If the

bioretention facility is to be used as a sediment basin prior to use as a bioretention facility, it should be excavated to the dimensions, side slopes, and 1 foot above the bottom of the modified soil layer elevations shown on the drawings.

3. **Construction considerations:**
   a. **Staking.** The bioretention cell area should be staked prior to any site construction to minimize traffic and compaction. This would not apply to situations where a sediment basin is converted to a bioretention cell.

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

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

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

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

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

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

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

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

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

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

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

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

Plants may require watering over several months to aid establishment, especially during drought periods.

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

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

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

5. **Maintenance.** Bioretention cells require seasonal maintenance. It is imperative that they be maintained to function properly and provide continuous visual aesthetics.

Table C5-S4- 8: Bioretention cell maintenance requirements

Activity	Schedule
• Prune and thin out plants when needed. Remove weeds throughout the growing season, preferably by pulling or trimming. Replace plants when needed. • Replace mulch when erosion is evident and/or weed growth is excessive. • Remove trash and debris from pretreatment area and bioretention cell.	Fall, spring, as needed
• Inspect inflow points for clogging (offline systems). Remove any sediment. • Inspect filter strip/grass channel for erosion or gullying. Re-seed or sod as necessary. • Trees and shrubs should be inspected to evaluate their health and remove any dead or severely diseased vegetation.	Semi-annually
• Look for evidence of standing water in the observation port. This may be a sign of hydraulic failure.	Annually
• Replace pea gravel diaphragm when necessary. • Replace modified soil layer when ponding greatly exceeds the design drainage time.	As necessary

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

*Insufficient Data

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

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

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

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

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

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

A. Overview

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

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

Figure C5-S5- 1: Bioswale

   Bioswales are not to be confused with a filter strip or grass swale, which are limited-application structural controls, used mostly for pretreatment and are not considered acceptable for meeting the water quality volume goals. Ordinary grass swales are not engineered to provide the same treatment capability as a well-designed bioswale with filter media and subdrainage systems. Filter strips and grass swales may be used for pre-treatment or included in a treatment train approach where redundant treatment is provided. Figure C5-S5- 2 through Figure C5-S5- 5 provide several views and configurations of bioswales with rock and earthen check dams.

   When bioswales are installed at sites such as gas stations or other sites where spills of hazardous materials are more likely to occur, the practices should be lined with an impermeable membrane. A shutoff valve should be installed at the downgradient end of the subdrain so that the materials can be contained within the practice and handled according to the proper protocol for managing and disposing of such materials.

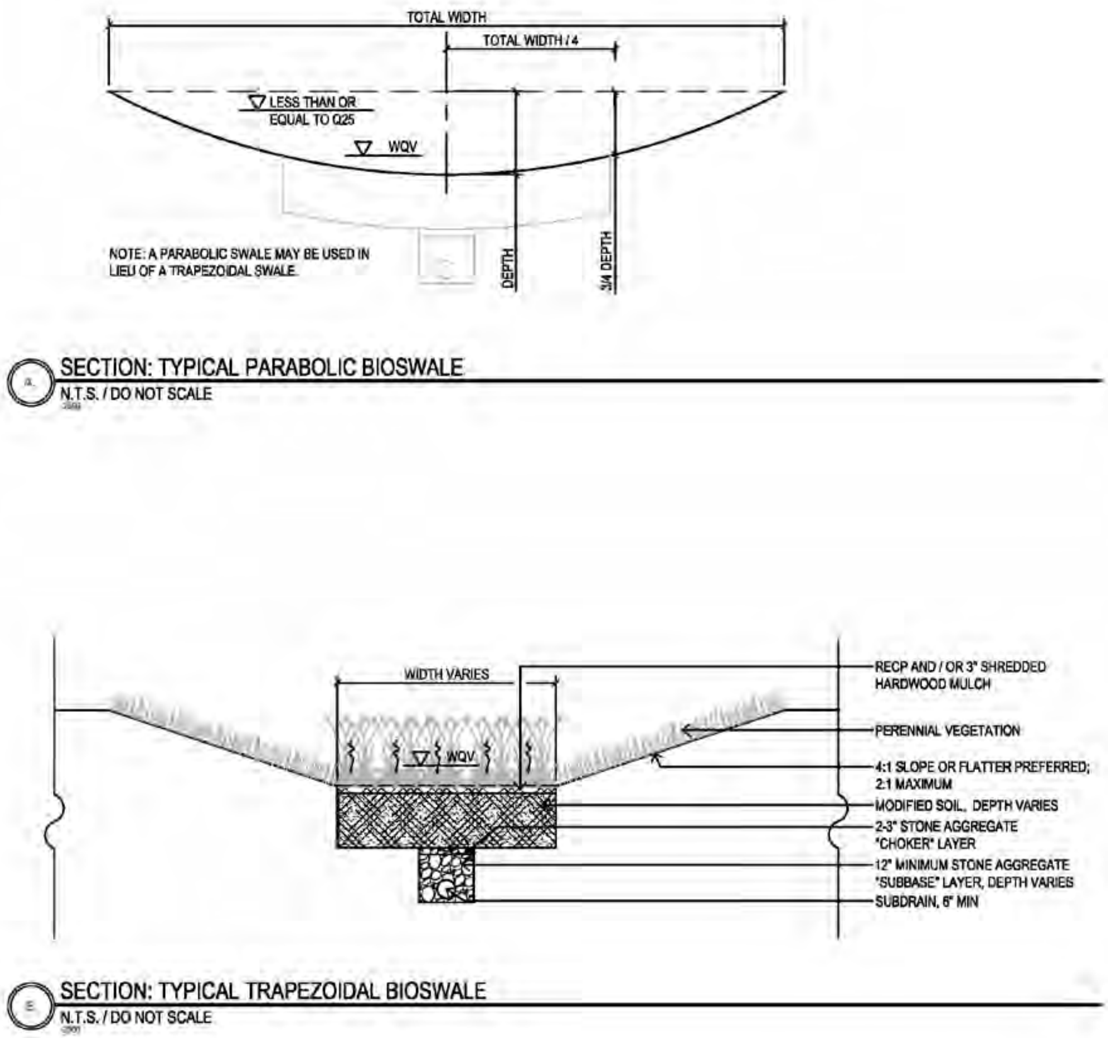


Figure C5-S5- 2: Configuration and design components of a bioswale. A is a cross-section of a parabolic-shaped bioswale and B a trapezoidal bioswale.

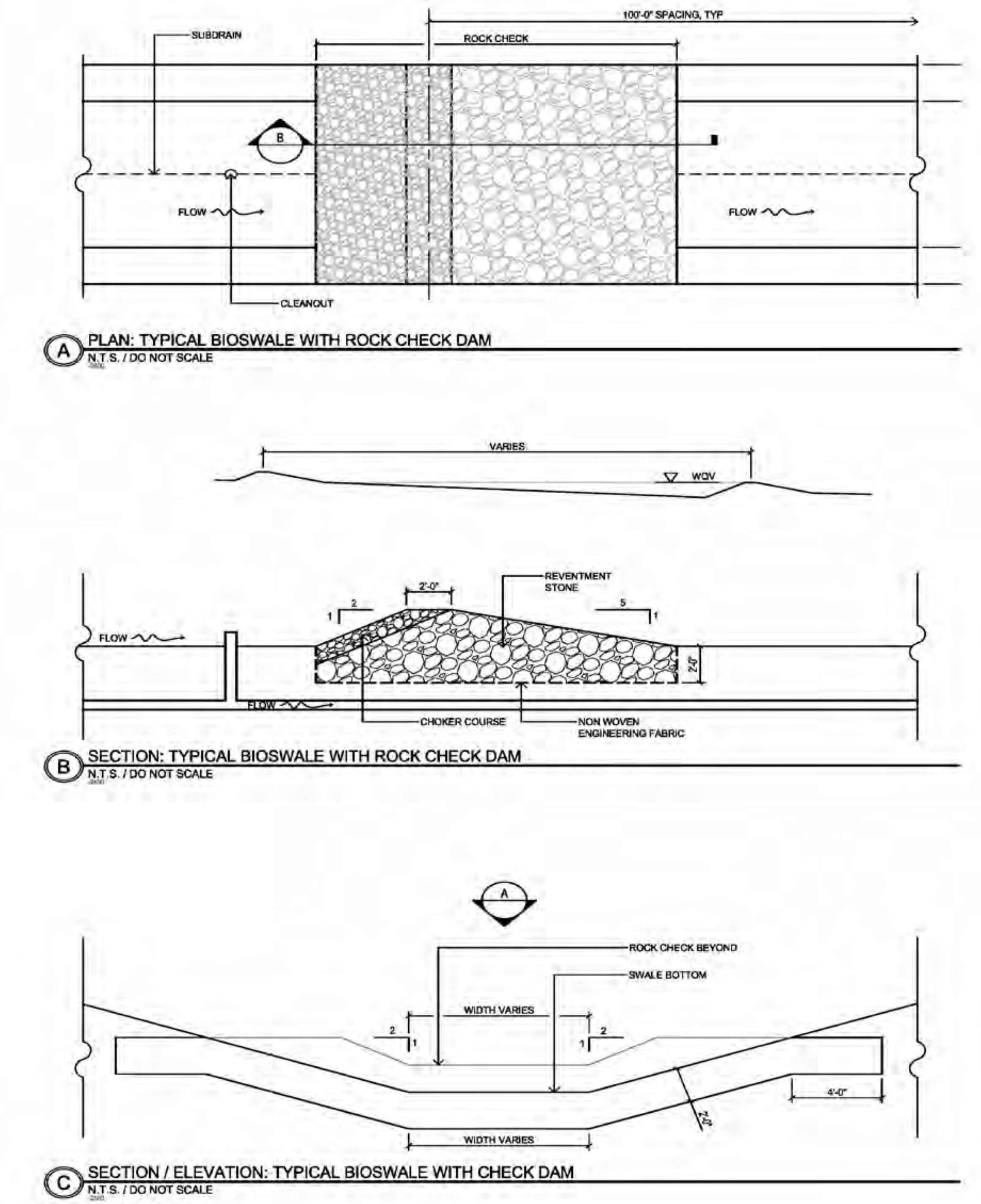


Figure C5-S5- 3: Configuration of a bioswale with a rock check dam showing a top view (A), side view (B), and cross-section (C).

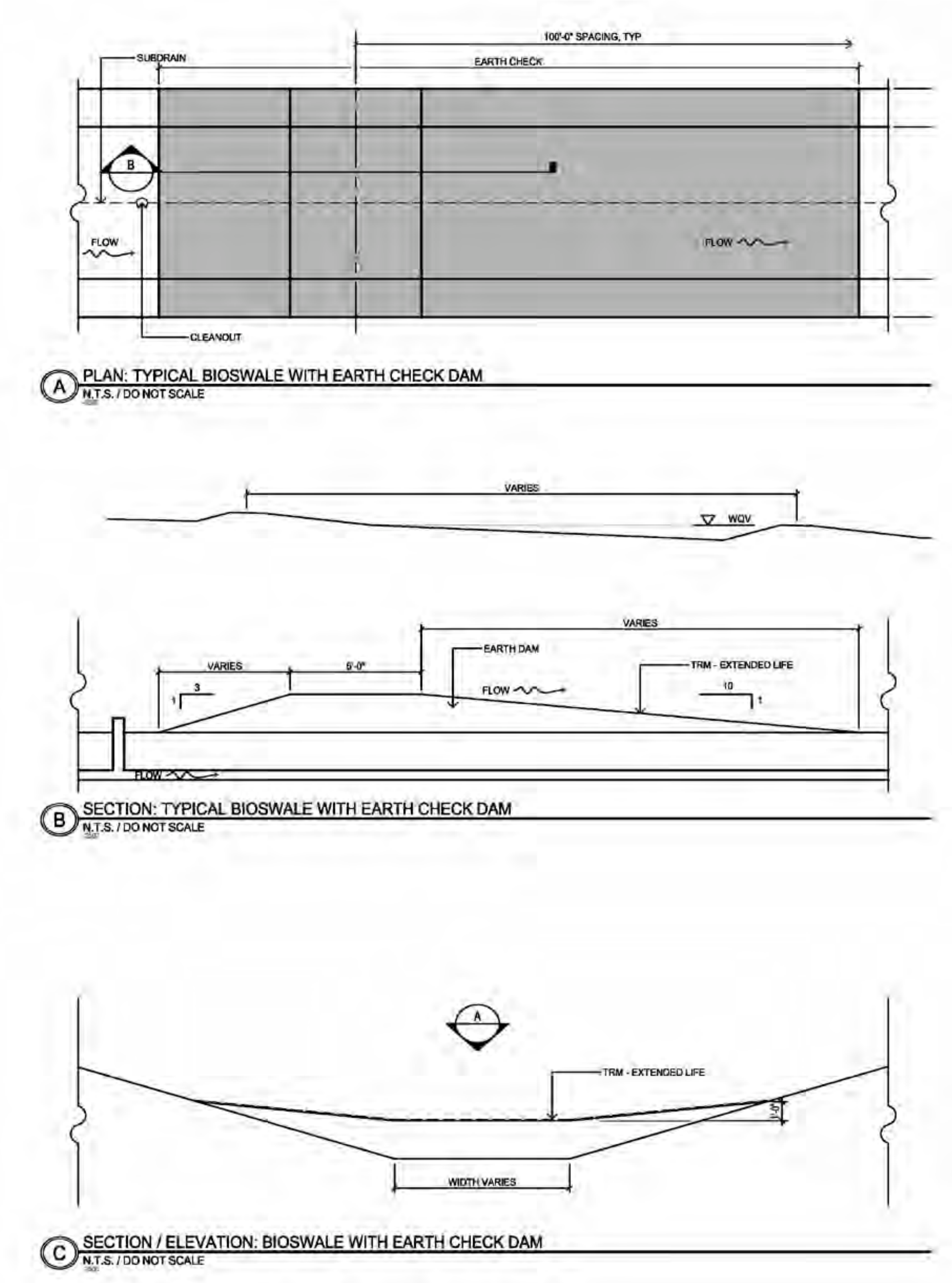


Figure C5-S5- 4: Configuration of a bioswale with an earthen check dam showing a top view (A), side view (B), and cross-section (C).

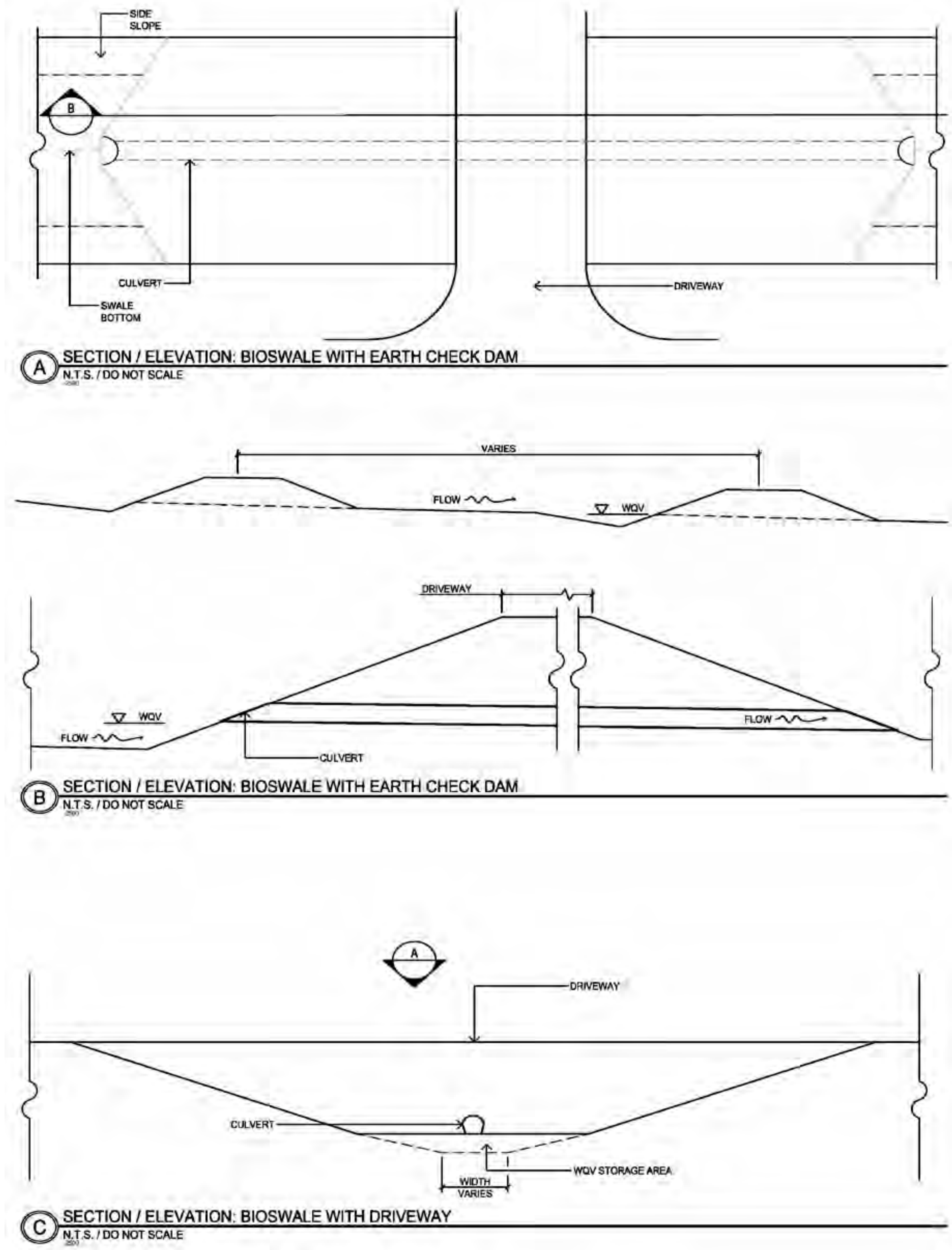


Figure C5-S5- 5: Configuration of a bioswale with an earthen check dam in a driveway application showing a top view (A), side view (B), and cross-section (C).

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

removal of stormwater contaminants. Properly designed bioswales are capable of addressing the WQv requirements for a given site. Chapter 9, section 1 provides expected pollutant removal efficiencies that can be used for planning and design purposes.

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

c. **Overbank and Extreme Flood Protection (Q_f and Q_p).** Bioswales should be able to convey runoff from larger flood events up to the 25 year storm (Q_{p25}) without eroding channel linings or large scale re-suspension of sediments. High water elevations for the 100 year storm event (Q_{p100}) should be checked to determine any flood impact to adjacent structures or properties.

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

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

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

The topography and soils of a site will determine the applicability of the use of swale designs. Overall, the topography should allow for the design of a swale with sufficient slope and cross- sectional area to maintain required treatment velocities. The following criteria should be evaluated to ensure the suitability of a bioswale for meeting stormwater management objectives on a site or development. Table C5-S5- 1 provides a list of considerations when planning for a bioswale.

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

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

- **Soils.** Modified soil sand and compost mixture.

Table C5-S5- 1: Planning criteria for bioswales

Distributed Placement and Location	It is preferred to consider stormwater management during initial site design. These practices can treat more manageable amounts of runoff closer to its source. They should be integrated into the site planning process, and aesthetic considerations should be taken into account in their siting and design. Elevations must be carefully worked out to ensure the desired runoff flow enters the facility with no more than the maximum design depth.
Site Integration	Can be placed close to the source of runoff generation. Stormwater management site integration is a preferred alternative to end-of-pipe BMP design, where feasible.
Drainage Area	Bioswales are best used in conjunction with upland practices through a “treatment train” approach. They are suited to both small and large drainage areas.
Online or Offline	They are almost always designed as an online system. A bioswale is considered online if all runoff from the upstream area enters the practice. Offline systems employ some type of diversion structure, which typically diverts the first flush of flow to the treatment practice, but allows flows from larger events to bypass the practice. This can prevent erosion within the practice and re-suspension of captured sediments.
Flow Diversion for Offline Swales	A diversion weir, flow splitter, or other practice needs to be designed to route flows from the WQ event to the bioswale, while allowing most of the flows from larger events to bypass the system (via parallel storm sewer system or other conveyance). Refer to Chapter 6, section 1, F for additional design information.
Intermittent Flow	They are designed for intermittent flow and must be allowed to drain and re-aerate between rainfall events.
Storm Events	Typically, bioswales are used to manage the WQv but convey larger storms. Refer to Chapter 3, section 6 for additional information about small storms. Bioswales may offer the possibility to attenuate or detain flows from larger storm events. Design to prevent: • Erosive flow velocities • Deep ponding that could compact soil layers • Extended drawdown periods that could affect desired plants

B. Design Methods

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

   Before choosing to employ a bioswale, review the feasibility information included earlier in this section. If feasible, proceed with designing this practice, starting with a review of the initial design considerations listed in Table 2, as well as the preliminary investigation information in Table C5-S5- 3.

Table C5-S5- 2: Initial design considerations

Limiting Conditions	Determine the depth to bedrock or typical groundwater elevation. Verify with geotechnical explorations or other methods.
Local Requirements	Determine if there are any local restrictions and/or surface water, groundwater, watershed, or water quality requirements that may apply.
Separation Distances	Ensure that room is available for installation, including any other local setback and/or separation requirements. Recommended setbacks are 25 feet from the foundation of a building; 5 feet from a property line; 50 feet from a private well; 20 feet from a geothermal well field; 100 feet from a municipal well.
Intermittent Flow	Design for intermittent flow and must be allowed to drain and re- aerate between rainfall events.
Character of Runoff	Determine the land use and the percent imperviousness that will generate runoff directed to the bioswale. Identify the pollutants of concern
Treatment Train	Consider multiple practices as part of a treatment train system for pollutant removal and maintenance considerations.
Quality Control	Design for water quality volume and conveyance of larger storm events.
Quantity Control	Not intended for stormwater detention but should be designed to safely convey larger storm events.
Aesthetics and Site Plans	Bioswale locations should be integrated into the site planning process, and aesthetic considerations should be taken into account in their siting and design.
Plant Materials	Native species are recommended. Consider plant establishment, flow conditions, and salt tolerance.
Maintenance	Develop a maintenance plan that includes access paths for equipment required for maintenance. See Table C5-S5- 7.

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

Table C5-S5- 3: Preliminary investigations

Properties of the Drainage Area to a Bioswale	Determine the expected drainage area to be routed to the bioswale and the projected amount of impervious surfaces. Bioswales should be sited such that the topography allows for the design of a channel with sufficiently mild slope (unless small drop structures are used) and cross-sectional area to maintain required treatment velocities. Surface properties required to determine time of concentration will be needed for final design (refer to Chapter 1, section 4). The area upstream of the bioswale should be stabilized prior to construction or adequate controls put in place to prevent high sediment loads being delivered to the bioswale.
Space Required	The relationship between percent of impervious area and required size of a bioswale is not easily defined. The target velocity will set the length required to achieve the desired residence time, then a cross-section developed that can convey the flow for the Water Quality event (WQv) below the maximum allowable velocity for treatment.
Slope	Channel slopes of less than 2% are recommended unless topography necessitates a steeper slope, in which case control structures can be placed to limit the slope to within the desired range. Flow control devices may include check dams and/or drop structures. Energy dissipation may be required below the control structures.
Minimum Head	Make sure there is sufficient elevation difference to pond water as needed and drain the soil and aggregate layers through a subdrain and/or outlet works to a finished surface, swale, or storm sewer system.
Existing Site Soils	Evaluate the existing site soils to determine existing percolation rates.

2. **Typical Components of Bioswales.** Before proceeding with final design, it is important to understand the function and purpose of the elements that make up this type of practice. Table C5-S5- 4 provides a summary of bioswale components and their function.

Table C5-S5- 4: Bioswale design components

Inlet Structures	Stormwater may be routed to bioswales in many ways, such as sheet flow off hard surfaces, or as concentrated flow from curb openings, downspouts, and pipe outlets. Inlet structures may also include features that divert only a portion of stormwater runoff to the bioswale (known as an offline configuration). Level spreaders can be used to disperse concentrated flows to sheet flows reducing flow depths and velocities, enhancing pretreatment possibilities.
Pretreatment Area	Use pretreatment areas when sediment and debris are anticipated. These areas reduce the potential for clogging and future maintenance.
Energy Dissipation	Energy dissipation controls may be needed at inlets and outlets.
Ponding Area	Bioswales are sized to store and infiltrate the entire WQv. Water is intended to be temporarily ponded below the crest elevation of each check dam or drop structure, then allowed to infiltrate into the modified soil layer.
Channel	The channel shape should be parabolic or trapezoidal.
Check Dams	Toe of upstream check should be at same elevation as crest of next check dam downstream. They can be constructed of rock or earth materials.
Modified Soil Layer	The modified soil layer filters stormwater. Typically this layer is 6-12 inches deep and consists of a uniform mixture of 75-90% washed concrete sand, 0-10% approved organic material, 0-25% soil with a soil texture that includes A- horizon characteristics, and meets specifications.
Choker Aggregate Layer	The choker layer separates the modified soil layer and aggregate subbase and prevents the modified soil from entering into the aggregate subbase. The 2-3 inch layer consists of clean, durable ⅜ inch diameter chip.
Stone Aggregate Subbase Layer	The aggregate layer at the bottom of the structure provides additional temporary storage capacity for the captured runoff after filtration. The layer consists of an open-graded, clean, durable aggregate generally of 1-2 inches diameter with a porosity of 35-40%.
Subdrain	A minimum 6 inch diameter perforated pipe is required, but generally an 8 inch diameter pipe is recommended to accommodate cleaning and maintenance. They provide the outlet for filtered water in areas with soils with poor percolation rates and act as a secondary outlet where soil percolation rates are better.
Outlet Structures	To avoid excessive ponding depths and drawdown times, outlet controls are needed to manage runoff from larger storm events. An overflow spillway set above the ponding depth can release flows in a non-erosive manner (velocities less than 5 feet per second below the outlet). For online configurations, riser pipes, intakes, or weirs may be used to release runoff from larger storms more rapidly than it could infiltrate through the soil layers. The underdrain system should discharge to the storm drainage infrastructure or a stable outfall. The riser can be used as an observation port.
Hydrologic Design	The primary goal of the practice is to treat runoff from the WQv event while having the ability to convey the larger storms. The bioswale should be sized to convey runoff from the largest design storm with a minimum of 6 inches of freeboard and without damage to adjacent property.

Table C5-S5- 5: Summary design criteria for bioswales

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

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

Without positive grade, Manning’s formula or other methods to calculate velocity in open channels will not apply. For such an application, we would recommend referring to the design section for bioretention cells (Chapter 5, section 4). Collectively, these bioretention cells would need to provide the required surface area for ponding to allow for proper infiltration of the WQv volume to be addressed by that series of “bioretention cells.”

Refer to Chapter 5, section 4 for the design procedure for bioretention cells, making sure to check for velocities and erosion potential when larger storms are expected to overtop each check dam or drop structure (Step 12 of that design procedure).

3. **Bioswale Sizing and Design Calculations.** The following design procedure assumes that the designer has completed preliminary investigations, and understands the design components of a bioswale, as outlined in Table C5-S5- 4 and Table C5-S5- 5. It is recommended that these calculations be completed as early as possible in the design process so that adequate room is reserved for stormwater management as site design development continues. Calculations can be adjusted as final site design is completed.

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

Step 1: Compute the WQv peak runoff rate.

To calculate peak rates of flow (in cubic feet per second) for the WQv event, use NRCS TR-20 or TR-55 calculation methods, using adjusted curve numbers (CNs) for this small event (1.25” in 24-hours). Refer to Chapter 3, section 6 for additional information.

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

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

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

Use the method outlined in Chapter 3, section 6, C, to compute the peak rate of flow (in cubic feet per second) and volume of stormwater runoff (in cubic feet) for the Channel Protection Volume (Cpv).

Use methods such as the NRCS TR-20, TR-55 (Chapter 3, section 7) or other acceptable methods to generate hydrographs to determine peak rates of flow (in cubic feet per second) and runoff volumes (in cubic feet) for the following events:

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

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

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

Step 3: Identify if the bioswale system is intended to be an online or offline system.

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

If planning for an offline system, a diversion weir, flow splitter, or other practice needs to be designed to route flows from the WQ event to the bioswale, while allowing most of the flows from larger events to bypass the system (via parallel storm sewer system or other conveyance). Refer to Chapter 6, section 1, F for additional design information. Include calculation details for the diversion structure with this design procedure.

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

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

For grass swales, refer to Chapter 9, section 2, E for general sizing requirements. The target flow velocity for bioswale treatment is 1.0 fps during the WQv event. Chapter 9, section 2 includes methods on how to modify the value of “n” for Manning’s equation to evaluate shallow flow in grass swales.

For filter strips, refer to Chapter 9, section 4, C for sizing requirements.

Forebays should have a storage volume of 0.1 inches of runoff volume per impervious acre drained (Chapter 3, section 11). Sediment will need to be mechanically removed from the forebay over time, so a depth marker and durable, solid materials are recommended for the bottom (to be certain when excavation is complete). The volume of WQv to be used to size the ponding areas of the bioswale can be reduced by the amount addressed in the pretreatment area(s) (typically no more than 10% of WQv).

Step 5: Review entrance designs.

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

Step 6: Design geometric elements.

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

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

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

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

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

Rock Check Dams

Stone materials should be “clean” or free of fine sediments, rock chips, and other small materials, which could clog the dam prematurely and not allow free percolation through the check dam. It is recommended to use erosion stone (smaller watersheds), Class E revetment stone (larger watersheds), or another similarly sized “clean” stone as the base material and a 12 inch layer choker course of 1 inch clean stone material on the upstream face. Refer to the Chapter 9, section 3 for more detailed descriptions of these stone materials.

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

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

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

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

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

The choker layer is a 2-3 inch layer that separates the modified soil layer and aggregate subbase and prevents the modified soil from entering into the aggregate subbase. It consists of clean, durable ⅜ inch diameter chip.

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

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

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

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

Reminder that Chapter 9, section 2 includes methods on how to modify the value of “n” for Manning’s equation to evaluate shallow flow in grass swales.

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

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

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

Where:
$L_{bioswale}$ = required length of bioswale, feet
T_{res} = average hydraulic residence time, minutes
V = peak velocity within the bioswale, during the WQv event, fps

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

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

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

Reminder that Chapter 9, section 2 includes methods on how to modify the value of "n" for Manning's equation to evaluate shallow flow in grass swales.

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

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

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

$$Q_{inf} = kA\left(\frac{1ft}{12in}\right)\left(\frac{1hr}{3600s}\right)$$

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

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

Percolation through a check dam: The rate of percolation through a rock check dam can be calculated similar to a method developed by the NAHB/NRC Designated Housing Research Center at Penn State University. This method assumes that the check dam is constructed out of clean stone (see Figure C5-S5- 3).

$$Q_{perc} = \frac{h^{3/2}W}{\left(\frac{L}{D} + 2.5 + L^2\right)^{1/2}}$$

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

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

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

Water quality inlets, notch weirs, orifices, etc.: Inlet structures, riser pipes, weirs, or orifice restrictions are options for features that can be used to control the release rate for the WQv from the ponding areas. Refer to Chapter 3, section 12 on how to correctly size the selected type of control structure.

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

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

Step 11: Subdrain system design.

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

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

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

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

Step 12: Additional provisions for large storm flows.

Bioswale systems are typically designed as online systems. They will receive flows from larger storms, which need to be safely conveyed through the swale system and possible into outlet structures (i.e. culverts, storm sewers, etc.) at the downstream end of the project. Flow across check dams in larger storms should be checked using weir equations (refer to Chapter 3, section 12) to make sure that larger flows can be passed over the dams and remain within the cross-section of the swale and be kept at non-erosive velocities.

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

Step 13: System outlet and overland spillway design considerations.

Check peak flow velocities near pipe outlets and spillways expected to be overtopped during large storms. For all storm events reviewed, velocities at any pipe outlets should be less than 5 feet per second and stabilization provided (refer to HEC-14). Overflow spillways should be designed with sufficient width to keep velocities less than 5 feet per second, and be properly stabilized or reinforced to withstand such velocities. Refer to Chapter 3, section 12, H for additional information.

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

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

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

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

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

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

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

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

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

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

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

2. **Design Example.**

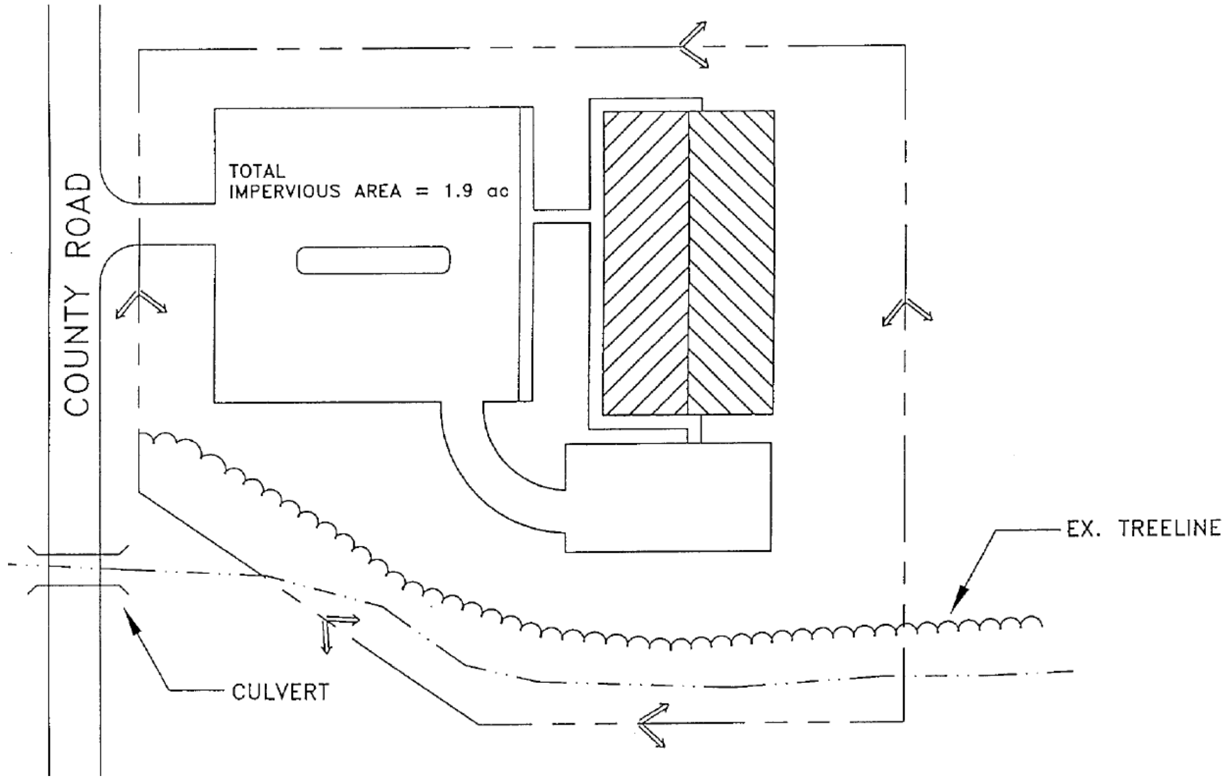


Figure C5-S5- 6: Recreation center, Central Iowa

Table C5-S5- 6: Site data

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

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

Step 1: Compute the WQv peak runoff rate.

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

To calculate peak rates of flow (in cubic feet per second) for the WQv event, use NRCS TR-20 or TR-55 calculation methods, using adjusted curve numbers (CNs) for this small event (1.25 inches in 24hours). Refer to Chapter 3, section 6 for additional information.

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

Using an adjusted CN value, the volume of runoff from this calculation should be close to the value of WQv volume that can be calculated using methods from Chapter 3, section 6. (8,031 CF ≈ 8,035 CF)

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

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

Use method outlined in Chapter 3, section 6, C, to compute the peak rate of flow (in cubic feet per second) and volume of stormwater runoff (in cubic feet) for the Channel Protection Volume (Cpv).

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

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

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

Use methods such as the NRCS TR-20, TR-55 (Chapter 3, section 7), or other acceptable methods to generate hydrographs to determine peak rates of flow (in cubic feet per second) and runoff volumes (in cubic feet) for the following events:

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

Step 3: Identify if the bioswale system is intended to be an online or offline system.

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

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

Alternatives to evaluate for pretreatment are:

Grass swale (without modified soil layers): For grass swales, refer to Chapter 9, section 2, E for sizing requirements. Using a site imperviousness of 60%, and a slope of less than 2%; a 45 foot long, 2 foot (minimum) wide swale is needed to meet pretreatment requirements.

Filter strip: If the 10 inch discharge pipe is connected to a level spreader to convert concentrated flow to sheet flow, a filter strip could be used. For filter strips, refer to Chapter 9, section 4, C-4 for sizing requirements. The chart uses a maximum inflow approach length for impervious areas of 75 feet. To have an equivalent impervious approach length maximum of 75 feet, the 1.8 acres (78,408 square feet) of impervious surfaces in this example needs to be spread over a width of 1,045 feet (= 78,408 SF/75 feet). Providing this length does not seem feasible. A filter strip might be a better option with a level spreader in a smaller watershed area, or as an online system receiving sheet flow runoff from paved areas that are less than 75 feet in length.

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

Storage required:

$$= DA\left(\frac{I}{100}\right)(0.1in)\left(\frac{1ft}{12in}\right)(43{,}560\, sf/ac)$$
$$= (3ac)\left(\frac{60}{100}\right)(0.1in)\left(\frac{1ft}{12in}\right)(43{,}560\, sf/ac)$$
$$= 653cf\ (or\ 8\%\ of\ WQv)$$

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

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

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

Step 5: Review entrance designs.

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

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

$$Q = CLH^{1.5}$$

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

Rearranging to solve for H yields:

$$H = \left[\frac{Q}{CL}\right]^{2/3}$$

For this example, $H = \left[\frac{21cfs}{3.3\times 15ft}\right]^{2/3}$

So the area of flow over the edge of the forebay would be $HL = 0.56 \times 15ft = 8.4ft^2$

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

Step 6: Design geometric elements.

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

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

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

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

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

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

The choker layer is to be a 3 inch layer that separates the modified soil layer and aggregate subbase and prevents the modified soil from entering into the aggregate subbase. It consists of clean, durable ⅜ inch diameter chip. The stone aggregate subbase layer is recommended to be 12 inches deep. Material should be 1-2 inch clean aggregate. The aggregate subbase layer will contain a subdrain, with the aggregate material extending at least 1 foot on either side of the subdrain (for wide parabolic cross-sections, the aggregate subbase material should extend across at least 10% of the area that would be covered by water during the WQv event).

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

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

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

Reminder that Chapter 9, section 2 includes methods on how to modify the value of "n" for Manning's equation to evaluate shallow flow in grass swales.

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

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

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

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

Where:
$L_{bioswale}$ = required length of bioswale, feet
T_{res} = average hydraulic residence time, minutes
V = peak velocity within the bioswale, during the WQv event, fps

$$L_{bioswale} = (10min)(0.72fps)(60\, sec/\mathrm{min}) = \boldsymbol{432ft}$$

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

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

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

Reminder that Chapter 9, section 2 includes methods on how to modify the value of "n" for Manning's equation to evaluate shallow flow in grass swales.

Assuming free-flow in the channel and solving Manning's equation for various depths (with the adjusted coefficients listed) yields the following results. The flow rate through the bioswale for the 25 year event is expected to be 16 cfs (from *Step 2*), so a peak flow velocity of around 2 fps would be expected for this event. We can also see from the results below that the channel has the capacity to convey even flows larger than the 100 year flow (21 cfs, from *Step 2*) with a velocity of less than 5 feet per second.

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

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

Flow over a rectangular weir is calculated from the following equation:

$$Q = CLH^{1.5}$$

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

Rearranging to solve for H yields: $H = \frac{Q}{CL}^{2/3}$

For this example: $H = \left[\frac{16cfs}{3.3 \times 6ft}\right]^{2/3} = 0.87ft$

So the area of flow over the dam would be $HL = 0.87ft \times 6ft = 5.2ft^2$

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

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

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

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

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

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

$$Q_{inf} = kA\left(\frac{1ft}{12in}\right)\left(\frac{1hr}{3600s}\right)$$

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

$$Q_{inf} = (1\,in/hr)(2592sf)\left(\frac{1ft}{12in}\right)\left(\frac{1hr}{3600s}\right) = 0.06cfs$$

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

$$Q_{perc} = \frac{h^{3/2}W}{\left(\frac{L}{D} + 2.5 + L^2\right)^{1/2}}$$

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

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

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

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

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

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

Step 11: Subdrain system design.

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

Step 12: Additional provisions for large storm flows.

Bioswale systems are typically designed as online systems they will receive flows from larger storms, which need to be safely conveyed through the swale system and possible into outlet structures (i.e. culverts, storm sewers, etc.) at the downstream end of the project. Flow across check dams in larger storms should be checked using weir equations (refer to Chapter 3, section 12) to make sure that larger flows can be passed over the dams and remain within the cross-section of the swale.

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

Flow over a rectangular weir is calculated from the following equation:

$$Q = CLH^{1.5}$$

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

Rearranging to solve for H yields: $H = \frac{Q}{CL}^{2/3}$

For this example, $H = \frac{21cfs}{(3.3\times6ft}^{2/3} = 1.04ft$

So the area of flow over the dam would be $HL = 1.04ft \times 6ft = 6.2ft^2$

From the continuity equation $V = \frac{Q}{A} = \frac{21cfs}{6.2ft^2} = 3.4fps$

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

Step 13: System outlet and overland spillway design considerations.

Assuming the flow is controlled by check dams only in this example, there is nothing more to check in this step. If the bioswale drained into an intake, culvert, or other storm sewer system, the designer should check peak flow velocities near pipe outlets and spillways expected to be overtopped during large storms. For all storm events reviewed, velocities at any pipe outlets should be less than 5 feet per second and stabilization provided (refer to HEC-14). Overflow spillways should be designed with sufficient width to keep velocities less than 5 feet per second and be properly stabilized or reinforced to withstand such velocities. Refer to Chapter 3, section 12, H for additional information.

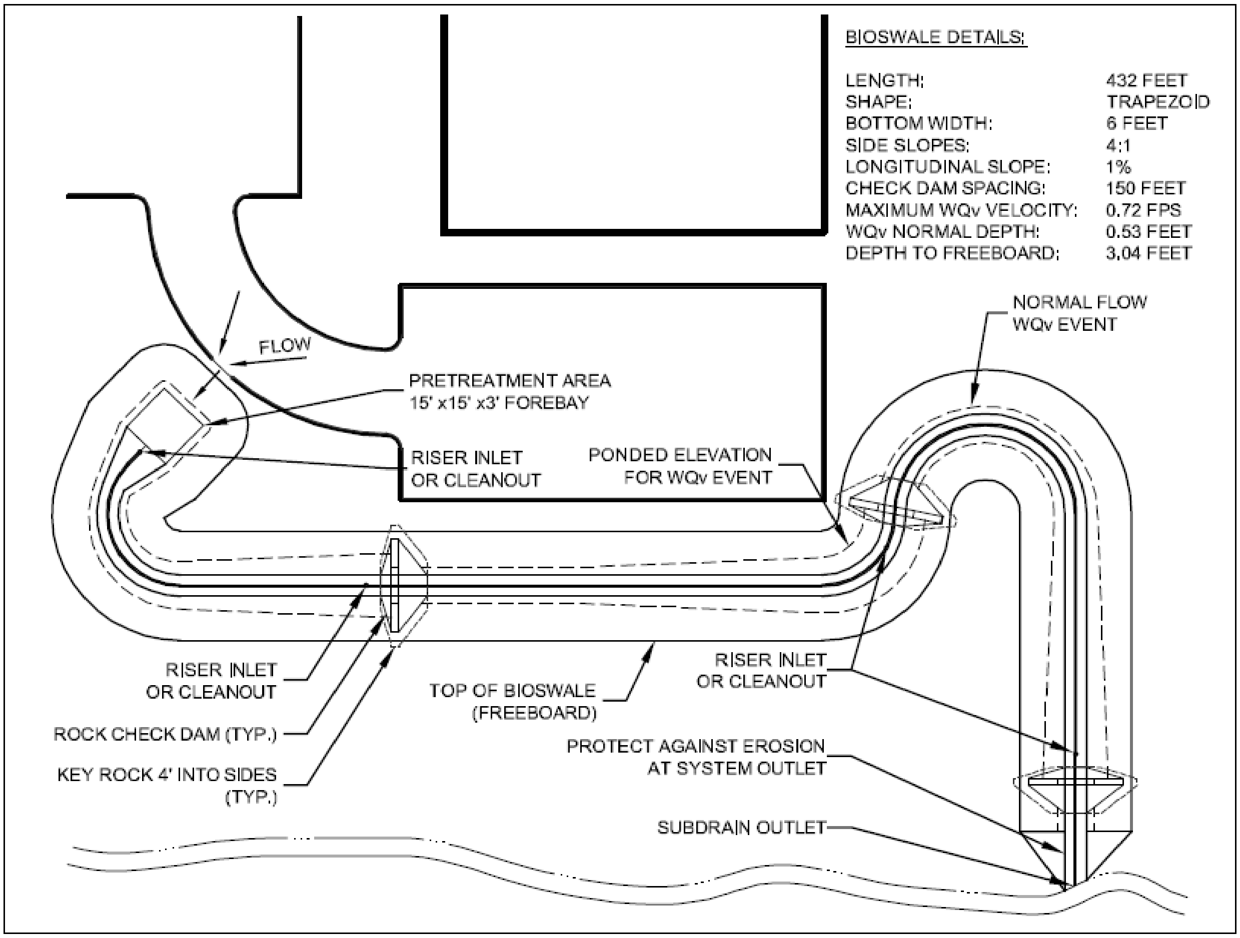


Figure C5-S5- 7: Site plan for project example

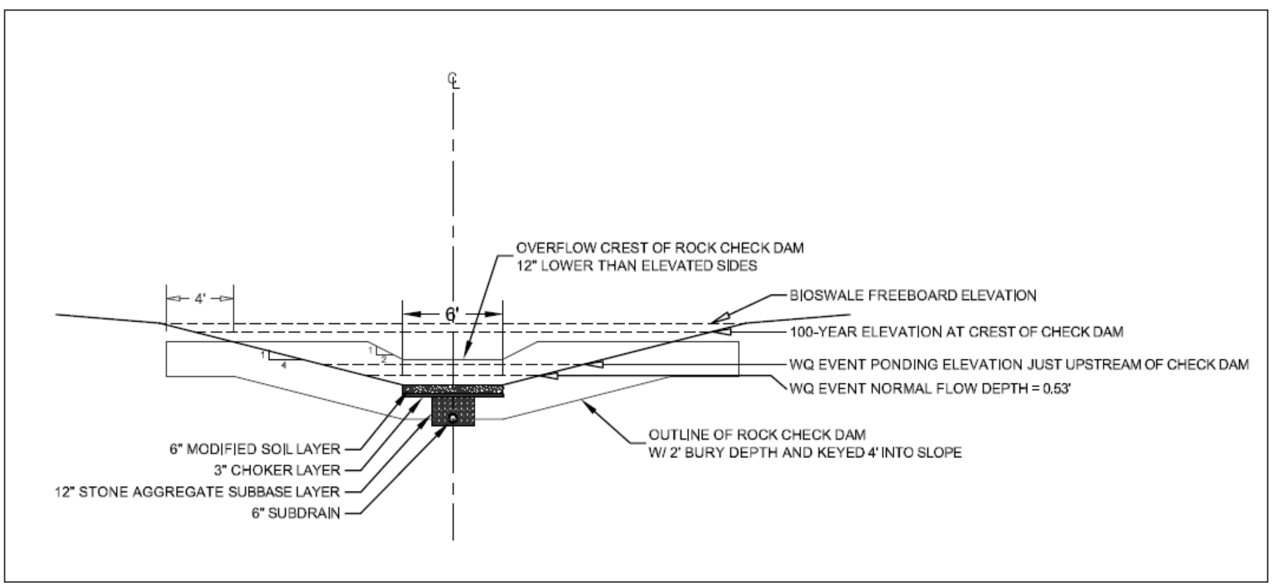


Figure C5-S5- 8: Cross-section for project example

C. Construction

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

D. Maintenance

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

Table C5-S5- 7: Bioswale maintenance requirements

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

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

Description: Soil quality management preserves and protects intact soil profiles from land disturbing activities. These activities often significantly alter the characteristics of the soil profile. Soil quality restoration (SQR) reduces compaction, increases pore space, improves organic matter content, and re-establishes populations of soil dwelling organisms (microbes, worms, insects, etc.) on soils disturbed during construction. Recreating the original soil profile after construction is not likely to be achieved. This chapter describes methods of creating a healthy soil profile which restore the ability of the soil to absorb water after construction is completed.

Typical uses:
- Preservation of intact soil profiles
- Restoration of disturbed soils as part of final grading and stabilization of construction sites.
- Incorporation into lawn care management practices on established landscapes.

Advantages/benefits:
- Reduces stormwater runoff volume.
- Protects water quality by infiltrating and processing pollutants in stormwater runoff.
- Reduces the need for irrigation by increasing water holding capacity and water availability.
- May reduce the need for fertilizers and pesticides.

Disadvantages/limitations:
- Access to soil restoration services may be limited.
- Access to compost may be limited.
- May increase development costs.

Maintenance requirements:
- Annual applications of compost amendments is recommended (but not required).
- Lawn clippings should not be removed, as they decompose they add organic matter.
- Pesticide use should be minimized to maintain healthy populations of earthworms, soil dwelling insects, and soil microbes.
- Strategic use of native landscaping should be considered, as opposed to cool season grasses to maintain and enhance soil quality over time.

A. Description

Chapter 1, section 1 details how land disturbing activities generates additional surface runoff. Creation of impermeable surfaces (rooftops, pavements and other hardscapes) and compaction of soils through grading activities limit the ability of the landscape to infiltrate water during rainfall events. Grading and compaction of the landscape are the primary reasons land development activities increase the volume and rate of stormwater runoff. If soil compaction is a major component in the increase of runoff volume, then restoring soil to better allow rainfall to infiltrate and percolate through the soil profile is a key method to reduce runoff.

Prior to settlement, Iowa soil within the native tallgrass prairie landscape had an 8-10% organic matter content. Rain infiltrating into the historic landscape moved slowly through the soil profile to emerge down-gradient as cool, clean groundwater discharge that fed and maintained stable, clean streams, rivers, wetlands and lakes. A healthy soil profile with 50% pore space should be able to infiltrate anywhere from 0.6-2 inches of water per hour into the soil profile. The water-holding capacity of most prairie soils should be around 0.2 inches of water per inch of soil profile. Therefore, a soil at field capacity with 50% pore space should be able to store a minimum of 2.4 inches of rainfall in the upper 12 inches of the soil profile.

Today, due to tillage-based agriculture and urban land development, many Iowa soils have only 4% soil organic matter (SOM) content or less. Most Iowa soils have about 2% SOM. Organic content can be even less where topsoil has been stripped and exported from development sites. Such soils have lost 60-80% or more of their ability to absorb, infiltrate, and store rainfall.

Undisturbed soils have layers or horizons that form over hundreds of years. Prairie soils have a surface O-horizon that is thin and contains a high concentration of SOM from decayed vegetation. The next layer, the A-horizon is what many refer to as topsoil. It is rich in SOM giving it a darker color and has less clay compared to subsurface horizons. The next is the B-horizon that has a higher clay content and lighter color than the A-horizon. The C-horizon or lowest horizon consists of the parent material in which the soil has been formed such as glacial till, or wind-blown deposits. Figure C5-S6- 1 shows the differences in soil profiles between prairie, corn and urban turf. The O-horizon that once appeared in the prairie soils has been either oxidized or eroded away due to modern agricultural practices as shown beneath the corn soil profile.

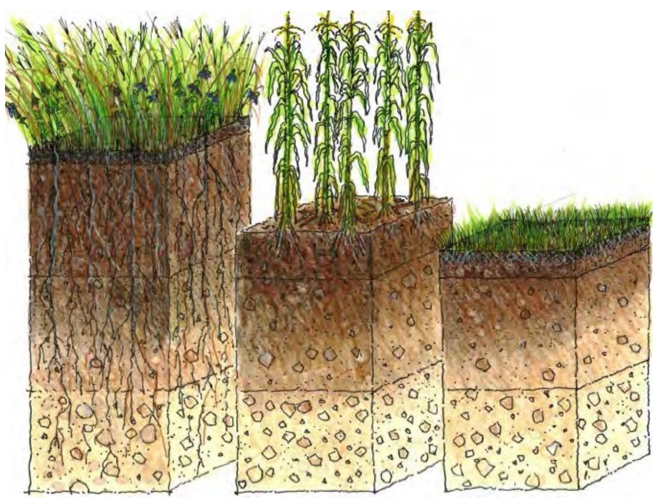


Figure C5-S6- 1: (L to R) Prairie soil profile, soils under modern agriculture and soils after urban development under turf grass.

Compaction of site soils through urban development have an NRCS curve number of nearly 90 (see open space, poor condition, soil group D; Table C3-S5-2). Compare that with soils, with little to no compaction, where curve numbers of 40-60 may be expected (see open space good/fair condition, soil group A/B; same table). Consequently, heavily graded urban green space is typically “hydrologically dysfunctional,” and the ability of these landscapes to absorb and infiltrate water is extremely limited.

Figure C5-S6- 2 provides results of research conducted in Ankeny, Iowa by Iowa State University that shows changes in soil quality indicators, bulk density and organic carbon, over time with changes in development practices. Bulk density, or the mass per given volume, is a measure of the compaction of soils. Organic carbon is related to organic matter content. Figure C5-S6- 1 shows bulk density has increased with time and organic carbon content has decreased with time. These marked changes began occurring in the 1980s onward when topsoil was being stripped and not replaced from sites and more mass grading and on-site vehicular traffic became common.

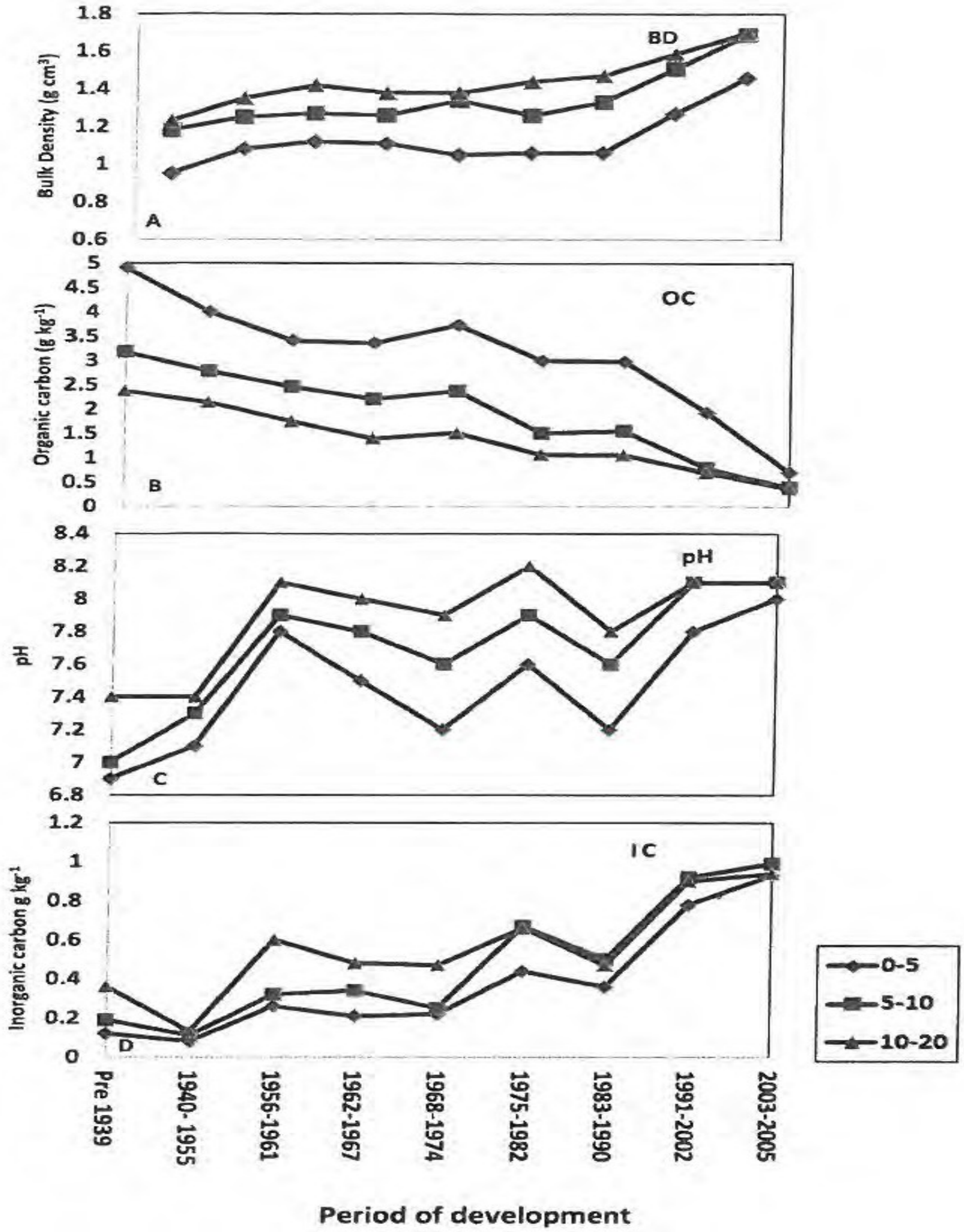


Figure C5-S6- 2: Variation of (A) bulk density (BD), (B) organic carbon (OC), (C) pH, and (D) inorganic carbon (IC) values in soils by period of development of Ankeny, Iowa at depths of 0-5, 5-10, and 10- 20 cm

Source: A. Langner, Manu, A. and Nath D., SSSAJ, 2012

The goal of soil quality management and restoration (and other infiltration-based stormwater management practices) is for urban landscapes to mimic the hydrologic functionality of the native pre-settlement landscapes, at least for the more commonly occurring small storm events. This is achieved through the protection or creation of soil profiles having at least 40% pore space, 2% (3-5% preferred) organic matter content, and a healthy population of soil microbes and other species of soil dwellers.

To reduce the effects of SOM loss and soil compaction, minimize mass grading activities. The first step in site design should be a review of site conditions prior to preparing a conceptual layout. Topography, existing soils, including soil depth and organic matter content, drainage paths, watershed boundaries, delineated wetlands and flood hazards should all be considered *before* site design begins. (Refer to Part F of this section for further details)

Soil quality is best maintained by minimizing the area impacted by construction. A designer should use the information gathered during the site review to determine ways to lay out the proposed development so the most permeable soils are preserved and the area disturbed or compacted by construction is reduced as much as possible. A "building envelope" should be delineated to confine grading activities, construction traffic, stockpiling of materials, and other construction activities within a defined area. An additional benefit to this step could be reduced grading and infrastructure development costs.

Where land disturbing activities are necessary, soil quality restoration (SQR) should be performed as part of final landscaping, prior to seeding or installation of sod. For single-family residential developments, this will generally be accomplished on a lot-by-lot basis after all construction activities are complete.

Foundation and basement excavation generates soil stockpiles, which along with other building activities can compact soils on a large portion of a typical residential lot. Soil conditions worsen when basement excavated subsurface soils are spread over the lot, prior to re-spreading topsoil.

When SQR techniques are used to counteract soil compaction, the soil profile under green space areas will be a water management and water storage resource. Locate pervious areas strategically so that stormwater runoff can be dispersed across it via sheet flow, where and when possible.

B. Stormwater management suitability

Where green space is limited, or transitioning runoff to sheet flow is not possible, another BMP will be necessary to manage stormwater runoff (ISWMM Chapter 2).

The water quality volume (WQv) may be managed on site with significant green space, when deep tillage practices are used. The WQv is the runoff generated by a 1.25" rain event. Runoff volume reductions for larger storms can be modeled through appropriate reductions in NRCS curve numbers by accounting for open spaces with higher quality soils and vegetation.

C. Pollutant removal capabilities

Good soil quality will generally provide for the capture of most of the major pollutants of concern, and would be comparable to bioretention for pollutant removal. Hydrocarbons, bacteria, sediment, metals, and other pollutants are generally captured in the top part of the profile when runoff is infiltrated. A healthy microbial population will degrade and utilize many of the pollutants as a food source. A pollutant, such as nitrogen moving in solution could move past the root zone of turf landscapes with high percolation rates. Incorporating strategic native landscaping along with soil quality restoration is recommended for increased evapotranspiration and more nutrient uptake.

D. Application and feasibility

Some component of SQR is applicable to almost all Iowa soils. Conditions such as hydric soils and surface ponding could render SQR not feasible. A hydric soil is permanently or seasonally saturated to cause anaerobic conditions in the upper part of the soil. Development on sites with hydric soils or surface ponding is discouraged as part of Low Impact Design due to the inability to infiltrate stormwater and generally because hydric soils are unsuitable for dwellings and other structures.

The potential infiltration and storage capacity of healthy soils makes an infiltration-based and groundwater driven hydrology feasible, which was the case back when the prairies and other native ecosystems were intact. If this type of a hydrology for 90% or more of rainfall events in Iowa can be replicated, then the potential for water quality enhancement and stabilization of stream flows seems quite feasible, as well.

SQR methods are not to be used on areas to be covered by structures or impervious surfaces nor on nearby areas where soil compaction is necessary for structural stability.

E. Planning and design criteria

A soil management plan (SMP) should be created for each new development. It should be created and initiated before any site layouts or designs are prepared, and it should be modified as the design process continues. Soil management plans are needed to treat landscapes as mass grading is completed and infrastructure is installed. A SMP will be needed as well for individual lots.

Table C5-S6- 1: Process for developing Soil Management Plans (SMPs)

1. Investigate existing site soil conditions.
2. Identify areas where soils and vegetation will not be disturbed on a site map or scale drawing as part of Contract Documents or site Storm Water Pollution Prevention Plan (SWPPP).
3. Identify areas where topsoil will be stripped and stockpiled on a site map or scale drawing.
4. Select which method(s) of Soil Management and Soil Quality Restoration are to be used and identify where they will be employed.
5. If using tillage for Soil Quality Restoration (SQR) then determine the depth of tillage and provide recommendations for suitable moisture conditions. Tillage of wet soils can cause smearing.
6. Determine and quantify types and amounts of materials needed to complete SQR requirements.
7. Specify methods for establishing permanent vegetative cover (i.e. sodding, seeding rates).
8. Incorporate SMP into site specific SWPPP if one is required to be implemented from initial disturbance to final stabilization.

Step 1. Prior to site design, soils information from resources such as county soil maps, geotechnical reports or other available data should be reviewed. Identify areas expected to have higher quality soils with intact soil layers. If possible, determine the type, quality and organic content of topsoil available on site. County soil map information can be viewed through the USDA at the following link: http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm

Step 2. First and foremost, areas having higher quality soils should be protected. By minimizing land-disturbing activities, soil profiles are left intact and compaction does not occur. Compaction, which increases bulk density and reduces pore space, is a primary culprit in the creation of hydrologically dysfunctional landscapes. During preliminary site design, orient improvements to minimize disturbance of higher quality soils. Plan grading activities to avoid compacting, filling or deep tilling under the drip line of any trees that are intended to be saved.

Step 3. Stripping and removing topsoil is another key aspect of post-construction soil quality problems. Topsoil contains organic matter that is the key to soils being able to absorb water. Soils with 2% or greater organic matter have the ability to absorb water like a sponge; lower than 2% and soils will absorb less rainfall, generating runoff quicker. Existing topsoil should be stripped, stockpiled and returned as part of final grading. Topsoil may need to be amended with compost to achieve the desired organic matter content of a minimum of 2%.

The SMP should identify where topsoil is to be removed, stockpiled and replaced. Stockpiles may need to be stabilized with temporary seed and mulch during construction and protected by perimeter sediment control measures (identified on the SWPPP and/or grading plan).

Step 4. Where land-disturbing activities cannot be avoided, SQR should be performed as part of final grading. Scarification and tillage should be done to a depth specified in the SMP.

Step 5. On sites where the majority of the landscape will be left as open space, deep tillage depths may be specified to address the water quality volume treatment requirements for adjacent open space. This is described in more detail later.

Step 6. Based on the selected method of SQR, the SMP needs to quantify the amount of materials (i.e. imported topsoil, compost, etc.) necessary to complete the work. Incorporate these quantities into the overall quantity estimate and/or bid documents for the project.

Step 7. Select the type of permanent stabilization to be provided for all disturbed areas (i.e. sodding, seeding, native turfgrass, lawn, etc.). The area where each measure is to be applied should be identified on at least one of the following: landscaping plan, seeding plan, or site schematic plan as part of the SWPPP; and quantified within the overall quantity estimate and/or bid documents for the project.

Note: Methods of SQR and vegetation establishment included within the SMP should be considered “non-structural practices” which are required to be identified within a SWPPP, when one is prepared. If a given project meets state or local thresholds that require a SWPPP to be prepared for a given site, the SMP needs to be made part of the SWPPP, either as an attachment or incorporated into the SWPPP.

Table C5-S6- 2: SQR design components, new construction

Topsoil	Topsoil used for SQR should contain or be amended to contain a minimum of 2% organic matter content. Topsoil depth may include the depth of topsoil with any sod that is laid.
Tillage Depth	Compacted soil should be tilled to a specified depth based on the SQR method to be employed at a given site. Selecting the method of SQR depends on quality and availability of topsoil, subsoil properties and if WQv is intended to be managed for adjacent impervious areas. Use ripping tillage tools for deep tillage.
Soil Moisture Conditions	Do not remove or respread topsoil or perform tillage operations when soils are wet.
Compost Application	Apply compost as specified in soil management plan to achieve a minimum of 2% organic matter content. Apply compost before deep tillage to incorporate organic matter into the soil profile. Refer to later portions of this section for application rates.
Sand	Sand may be used to alter soil texture and increase infiltration/percolation rates.
Native Landscaping	Penetrometer readings should be no greater than 200 psi to a depth of at least 8 inches.
Runoff from Adjacent Impervious Areas	Concentrated flows must be avoided. Disperse runoff from impervious areas across the SQR area in sheet flow, using a level spreader or other means to evenly distribute runoff across the treatment area.

Table C5-S6- 3: SQR design components, existing vegetation

Aeration	Deep tine or deep plug aeration; do not perform aeration after a rain or on wet soils.
Compost Application	Apply a ½-¾ inch layer of compost over aerated lawn.

NOTE: Direct stormwater runoff toward areas with restored soil quality. Disperse runoff as much as possible to discourage concentrated flow into one area and encourage sheet flow across the entire amended area.

F. Design procedures

For all projects a SMP should be created. Refer to Part E and Table C5-S6- 1 of this chapter for guidance in developing the SMP. The SMP should detail one or more of the following methods for restoring soil quality and identify where these methods will be employed. The SMP should be included within the Storm Water Pollution Prevention Plan (SWPPP) for any site where one is required. (The SMP details methods to preserve or enhance soil health, which would be a non-structural stormwater management practice, required to be detailed within a SWPPP.)

Topsoil, as defined in the Iowa Department of Natural Resources storm water General Permit no. 2, means the fertile, uppermost part of the soil containing significant organic matter largely devoid of debris and rocks and often disturbed in cultivation. When used in SQR methods, "significant organic matter" means the topsoil contains at least 2% organic matter. This is generally the approximate minimum amount of organic matter found in crop ground and pasture land in Iowa.

For the purpose of this section, a healthy soil profile is defined as an A-horizon with a depth of at least 8 inches underlain by intact B- and C-horizons, meeting the following requirements:

i. Soil from an undisturbed A-horizon with a clay content less than 25% and at least 2% organic matter content.
ii. Soil does not have a bulk density that exceeds 80 lb/ft (1.3 gm/cm3) (Michael J Singer and Donald N Munns. Soils and Introduction. 1987, Macmillan Publishing Company, New York) (Edward J Plaster. Soil Science and Management. 2009, 5^{th} Edition, Delmar, Clinton Park, New York)
iii. A penetrometer reading of no greater than 200 psi to a depth of at least 8 inches below the surface.
iv. Soil is not hydric and has at least 2 feet of separation from normal high water table.
v. Soil could also have less than 8 inches of undisturbed topsoil with good structure and still meet the definition of a healthy soil profile if it has percolation rates of 1 inch/hour or more.

Prior to preparing a conceptual layout or site design, review sources of information to identify soil and determine existing soil conditions by reviewing county soil maps, geotechnical reports or field visits. Areas shown as Hydrologic Soil Groups "A" or "B" are best candidates for preservation (see Chapter 3, section 5, page 4 for more information on soil groups).

Additional studies or testing is required to verify the presence of a healthy soil profile. Geotechnical studies, coring or soil pits and percolation tests can be used to evaluate these properties. Soils determined to meet the definition of "healthy" as stated above should not require amending. Verify that at least the upper 8 inches of the soil horizon meets the definition of healthy soil.

Consider location of identified healthy soils in initial site design. Design should layout proposed improvements to avoid disturbing or compacting healthy soils as much as possible. Final design plans and the site SMP should identify which areas are to be preserved using this method. Both the design plans and SMP should note the method of protecting these areas (i.e. construction fence, etc.) to prevent them from being disturbed or compacted by tracking or storage of materials.

The following decision chart can be used to determine the appropriate methods to use for site conditions. Methods 1 and 2 are used to maintain a healthy soil profile when the soil is not to be disturbed. Methods 3-7 are used to recreate a healthy soil profile on areas that have been severely disturbed by construction activities. Method 8 is used on areas where construction activities have been completed in the past, stabilization has been achieved and structures are in place. Results when using method 8 are inferior to other methods and it is to be used only when the use of other methods was not possible.

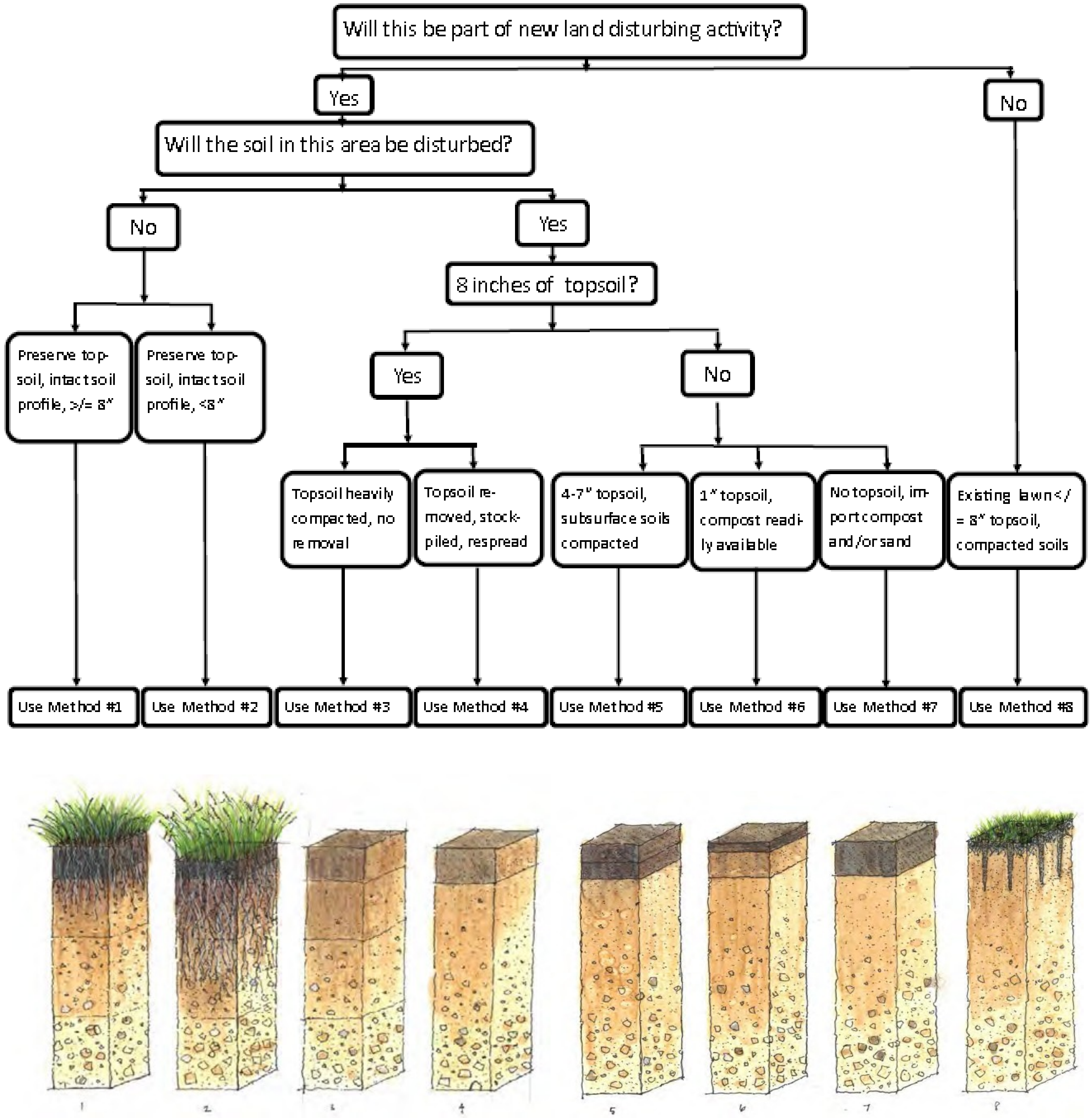


Figure C5-S6- 3: Decision guide for selecting soil management and soil quality restoration methods.

Soil Management on Undisturbed Sites

The goal is to maintain at least 8 inches of healthy soil profile.

Method 1. Preserve existing healthy soil profile: Identify undisturbed areas where topsoil will not be disturbed and protect it from compaction.

This method is intended to preserve areas determined to have a healthy A-horizon to a depth of at least 8 inches with a B- and C-horizon that appear to be intact. Prior to any site design, available information shall be reviewed to determine existing soil conditions. Geotechnical studies, coring, soil pits or other soil tests shall be performed on-site as needed.

Method 2. Preserve undisturbed areas that have intact A-, B- and C-horizons where the topsoil may never have developed to a depth of 8 inches but due to good soil structure and adequate porosity have a Hydrologic Soil Group of A or B and have percolation rates of 1 inch or more per hour. Sandy soils or timber soils may never have developed an 8 inch deep A- horizon but may have adequate infiltration and percolation rates to manage the WQv.

This method is used where the soil profile has the ability to infiltrate and percolate the WQv even though the A-horizon may not be 8 inches in depth.

Soil Quality Restoration on Disturbed Sites

The goal is to recreate a healthy soil profile on areas that have been severely disturbed by construction activities. When tillage is performed, do not till wet soils. If ruts from medium weight equipment traffic form, the soils are too wet. Penetrometer readings of no greater than 200 psi to a depth of at least 8 inches and a minimum organic matter of 2% in the topsoil generally indicate successful SQR.

Method 3. Eight inches or more of topsoil is present but compacted by land disturbing activities.

This method is intended to restore at least 8 inches of compacted A-horizon soils to its previous, uncompacted, functioning state.

Procedure:

i. Topsoil used for SQR must meet the definition of topsoil found in Part F. “Design Procedures” above.
ii. Till disturbed, compacted area to a depth of at least 8 inches.

Method 4. Stockpile topsoil, respread 8 inches or more of topsoil.

This method is applied where topsoil is present at a site prior to construction but is stripped, stockpiled and respread following construction.

Procedure:

i. Topsoil used for SQR must meet the definition of topsoil found in Part F. “Design Procedures” found above.
ii. Site soils should be stripped and stockpiled in an approved location identified in the SMP. The SMP should identify the depth of the topsoil layer to be stripped and replaced. Stripping and stockpiling should occur before other site grading or construction activities are initiated to keep topsoil separate from lower horizon soils.
iii. Soil stockpiles should be protected by appropriate erosion and sediment control measures, identified within the SWPPP.
iv. Respread the topsoil after all grading and trenching activities in the area have been completed. If SQR is completed prior to individual lot construction, the topsoil material should be stripped prior to lot construction and respread as necessary to avoid excessive compaction and so the topsoil will remain on the surface after construction is completed.
v. Remove large clods, roots, litter, stones larger than 1 inch (1/2 inch for residential lawns) and other undesirable material. After respread, avoid placement of basement spoils, fill, other materials or heavy equipment on the restored area.
vi. Perform tillage as necessary to address excessive compaction. No not till wet soils.

Method 5. Combination of 4-7 inches of topsoil and tillage to achieve an 8 inch soil profile depth.

Use a combination of respread of a minimum of 4 inches topsoil and tillage to create an 8 inch thick healthy soil profile**.**

Procedure:

i. Topsoil used for SQR must meet the definition of topsoil found in Part F. “Design Procedures” found above.

ii. Till or scarify the upper surface of the existing soil to a depth of 1-4 inches prior to placement of topsoil. Do not till wet soils.
iii. Respread the topsoil after all grading and trenching activities in the area have been completed. If SQR is completed prior to individual lot construction, the topsoil material should be stripped prior to lot construction and respread as necessary to avoid excessive compaction and so the topsoil will remain on the surface after construction is completed.
iv. Remove large clods, roots, litter, stones larger than 1 inch (½ inch for residential lawns) and other undesirable material. After respread, avoid placement of basement spoils, fill, other materials or heavy equipment on the restored area.
v. Perform tillage as necessary to address excessive compaction. Do not till wet soils.

Table C5-S6- 4: Recommended tillage, topsoil and compost depths for soil quality restoration to get 8 inches of healthy soil that includes 4 inches of topsoil.

Method	Tillage Depth (Inches)	Topsoil Depth (Inches)	Compost Depth (Inches)
4	0	8	0
5	1	7	0
5	2	6	0
5	3	5	0
5	4	4	0
6	6	1	1
7	8	0	2

Method 6. Topsoil blended with compost applied as surface blanket over tilled subsoil.

Use when there is not enough topsoil onsite and compost is readily available. Up to 3 inches of topsoil can be replaced with 1 inch of compost. Thus, the equivalent of 4 inches of topsoil can be achieved through a blend of 1 inch of topsoil and 1 inch of compost. This soil blend is spread as a surface blanket over 6 inches of tilled subsoil. Tillage is again performed to incorporate the topsoil and compost blend into the upper portion of the subsoil to a minimum total depth of 4 inches to create an 8 inch thick healthy soil profile.

Procedure:

i. Till or scarify the upper surface of the existing soil to a depth of 6 inches prior to placement of topsoil and compost blend. Do not till wet soils.
ii. Topsoil used for SQR must meet the definition of topsoil found in Part F. “Design Procedures” found above.
iii. Spread 2 inches of topsoil and compost blend over the tilled subsoil after all grading and trenching activities in the area have been completed. If SQR is completed prior to individual lot construction, the topsoil and compost blend should be stripped prior to lot construction and respread as necessary to avoid excessive compaction and so the blend will remain on the surface after construction is completed.
iv. Remove large clods, roots, litter, stones larger than 1 inch (½ inch for residential lawns) and other undesirable material. After respread, avoid placement of basement spoils, fill, other materials or heavy equipment on the restored area.
v. Perform tillage to a minimum depth of 4 inches to incorporate topsoil and compost blend. Do not till wet soils.

Method 7. Create an engineered healthy topsoil profile onsite where topsoil is absent by importing compost and possibly also sand. Sand alone, without the addition of a suitable amount of organic matter, cannot be used with subsoil to create a healthy soil profile. If sand is to be added with the compost, spread a suitable amount or calculate the depth of sand by volume required to change the soil texture class to that of a loam or sandy loam soil (Table C5-S1- 1 and Figure C5-S1- 4).

Use when topsoil is absent at a site prior to construction, topsoil is otherwise unavailable or when topsoil is exported because space is not available to stockpile stripped topsoil.

Procedure:

i. Upon completion of site grading and construction activities, the area where soil is to be amended should be inspected. Remove large clods, roots, litter, stones larger than 1 inch (½ inch for residential lawns) and other undesirable material. Remove smaller rocks or gravel if they densely cover the surface in a given area.

ii. Spread 2 inches of compost prior to tillage.

iii. Sand may be added to change the texture of topsoil to increase infiltration and percolation rates. Refer to Chapter 5, section 1, Part E for additional information. If sand is to be added with the compost, spread a suitable amount or calculate the depth of sand by volume required to change the soil texture class to that of a loam or sandy loam soil (Table C5-S1- 1 and Figure C5-S1- 4). Sand should be added in a uniform layer before tillage. Refer to Table C5-S6- 5 on recommended additions based on soil texture. It is recommended to test site soils to determine actual sand application rates, which should be less than those listed in the table below. Alternatively, sand and compost can be mixed then spread and tilled.

iv. Incorporate the compost or compost and sand blend through tillage to a minimum depth of 8 inches. If SQR is completed prior to individual lot construction, the engineered topsoil should be stripped prior to lot construction and respread as necessary to avoid excessive compaction and so the topsoil will remain on the surface after construction is completed.

v. Remove large clods, lumps, roots, litter, stone larger than 1 inch (½ inch for residential lawns) and other undesirable material. After respread for individual lot construction, avoid placement of basement spoils, fill, other materials or heavy equipment on the restored area.

vi. Perform tillage as necessary to address excessive compaction. Do not till wet soils.

Table C5-S6- 5: Estimates of Sand Amendment Rates Required to Change Texture Class

Soil Texture Class	Silt* %	Sand* %	Clay* %	Amendment Rate (CY sand/CY original material)
Sandy Loam	30	50	20	None
Loam	50	23	27	None
Silt Loam	68	5	27	0.35/1.00
Sandy Clay Loam	19	44	37	0.80/1.00
Clay loam	40	19	41	0.50/1.00
Silty clay loam	54	5	41	0.50/1.00
Sandy clay	5	39	56	1.75/1.00
Silty clay	40	5	55	1.75/1.00
Clay	5	5	90	3.50/1.00

*Based on ISWMM Figure C5-S1- 4 using percentage values for a given soil requiring the greatest amount of sand amendments to make soil act as a sandy loam or loam textural class Hydrologic Soil Group “B”.

Method 8. Soil enhancement on previously developed areas can be achieved by aeration and compost applications to enable existing soils and vegetation to absorb the WQv.

Use to improve soil quality to support existing vegetation and reduce runoff on areas already developed and built upon. This method will provide for the capacity for the landscape to absorb the WQv, but is not intended to address WQv requirements for adjacent hard surface areas. This is to be used on areas where construction activities have been completed in the past, stabilization has been achieved and structures are in place. Results are inferior to other methods and it is to be used only when the use of other methods was not possible.

Procedure:

i. Mow existing vegetation to a height of approximately 2 inches.
ii. Aerate to a depth of 4 inches (6-8 inches preferred).
iii. Apply ½-¾ inch compost blanket over the mowed area.
iv. Apply seed as specified or incorporate seed into the compost blanket if using a pneumatic blower. If a good stand of grass exists, seeding may be optional if the compost blanket is thin enough for established grass to grow through.
v. Water twice daily (morning and evening) or otherwise as needed until vegetation is established.

G. Water Quality Volume Management Guidelines

The recommended approach to determining the available soil water storage is based on research conducted by Hudson (1994) that provides data on the relationship between available soil water and percent organic matter in soil. Data collected in this research included soils information on samples collected from Iowa. Hudson's research shows that the higher the organic matter content, the greater the available storage capacity of the soil. Essentially, soils with higher organic content have greater ability to swell like a sponge, retaining more water within their pore spaces. Data from the Hudson study can be used to determine the available water storage characteristics for soils which have been preserved or where SQR techniques have been applied. This can be used to determine the depth of soil with a given organic matter content needed to manage the WQv.

If the organic matter content of a soil is known, Table C5-S6- 6 can be used to determine the available water storage for every inch of depth for the restored or preserved soil profile. From that, it projects the available storage within soil profiles of 4, 6 and 8 inches. This table is based on the Hudson research, which included soils with a silt loam texture and a bulk density of 1.25 gm/cm3 (78.0 lb/ft^3).

Table C5-S6- 6: Amount of available water storage in healthy soil profiles based on percent organic matter content. Data is based on research conducted by Hudson (1994).

Assumes bulk density of 1.25 gm/cm^3 and silt loam texture.
Available Water Content (AWC = Field Capacity – Permanent Wilting Point); AWC = Theta * 100; Theta = Volumetric Water Content

% SOM by weight	Bulk Density (gm/cm^3)	Available Water Storage (in/in soil)	Available Water Storage (in/4 in soil)	Available Water Storage (in/6 in soil)	Available Water Storage (in/ 8 in soil)
1	1.25	0.13	0.52	0.77	1.03
2	1.25	0.17	0.66	1.00	1.33
3	1.25	0.20	0.81	1.22	1.62
4	1.25	0.24	0.96	1.44	1.92
5	1.25	0.28	1.11	1.66	2.22
6	1.25	0.31	1.26	1.88	2.51
7	1.25	0.35	1.40	2.11	2.81
8	1.25	0.39	1.55	2.33	3.10

1. To properly use this table, you will need to know the organic matter content for a given soil. Since evaluation and design will be occurring before construction the content for the future restored soils will usually be unknown. An assumed value will need to be used that is supported by actual soil data collected from the site. This value will need to be verified by post-construction testing. Care should be taken in making this assumption, as if testing shows that the assumed value has not been achieved, additional SQR measures will be needed to meet the selected value, or other BMPs will be needed downstream to address the volume of runoff not addressed by this practice.
2. The column in orange can be used to calculate the available water storage for a soil of any depth, by selecting the value for a given organic matter content (%SOM) and multiplying it by the depth of the soil profile in inches.
3. Remember that the first 1.25 inches of water storage are needed to address the volume of rainfall that falls directly

onto the preserved or restored area. When the available storage exceeds this value, there is surplus storage which can be used to manage the WQv requirements of adjacent areas, when their runoff is to be directed across the preserved or restored area. (Profiles with such conditions are highlighted in light blue in Table C5-S6- 6.) Refer to following pages for section titled "Guidance on Using Preserved or Restored Areas to Manage WQv for Adjacent Areas".

For example: An 8 inch soil profile known to have 6% OM has 2.51 inches of available storage. The first 1.25 inches of storage will absorb rainfall from the 1.25 inch WQv event. That means that 1.26 inches of storage remain available to absorb runoff to that soil profile from outside areas.

Organic matter content at a given site may be unknown, or soils may either be compacted or lack adequate organic material to meet WQv requirements. In such a case, the following guidelines may be applied to use compost as a SQR technique to improve the water storage capacity of a soil profile.

Table C5-S6- 7: Water volume managed based on disturbed soils amended with compost.

Assume a soil bulk density of 2 gm/cm^3 (120 lb/ft^3) for soil material (compacted subsoil or B-horizon).

Soil (in)/ Compost (in)	Weight Soil (lb)	Weight Compost (lb)	Total Weight (lb)	% OM	Water Volume Managed (in)
7/1	70	3.7	73.7	1.5	1.2
6/2	60	7.4	67.4	3.2	1.7
5/3	50	11.1	61.1	5.5	2.4
4/4	40	14.8	54.8	8.1	3.2

Guidance on Using Preserved or Restored Areas to Manage WQv for Adjacent Areas:

As noted previously, when it can be verified that there is storage capacity in a given soil profile which exceeds the 1.25 inch WQv rainfall depth, there is an opportunity to manage runoff from adjacent areas. Flow from upstream areas could be directed to the preserved or SQR area, in a manner which would allow that runoff to infiltrate into the healthy soil profile. This allows these areas to act as stormwater Best Management Practices (BMPs) for managing the WQv requirements for such adjacent areas.

If this is the intent of a given design, the following requirements should be met:

1. Install and maintain appropriate site and perimeter controls to prevent sediment discharge from the adjacent construction areas to the BMP during construction.
2. After construction, provide for adequate separation between any impervious surfaces and the BMP area for a pretreatment buffer.
3. To act as a BMP for managing runoff from adjacent areas, runoff needs to be distributed evenly as sheet flow across the area to be counted as a BMP so it can infiltrate into the soil profile.
    i. For smaller applications (runoff from adjacent residential roofs, open spaces without SQR and small impervious areas) careful grading at the point of entry can be used to spread out flows.
    ii. For larger areas, it may be necessary to employ a level spreader or other structural method to convert concentrated flow to sheet flow or to prevent runoff from following a concentrated path of flow into the BMP.
    iii. The finished surface of the BMP area should be graded as such that flow to be treated will spread out across it. Flow should remain as sheet flow and not concentrate into low points or swales.
    iv. There can be larger areas preserved or where SQR is applied, but only those areas where sheet flow will pass over the finished surface should be counted as a BMP towards treating the WQv for adjacent areas.

4. To reduce flow velocity, minimize erosion and promote infiltration, the slope within the BMP area being counted toward WQv treatment should not exceed 6%.

5. The BMP area will need to be verified to have healthy topsoil to the desired depth and organic matter content needed to manage the WQv. Refer to Part I, "Construction Observation and Verification Requirements" within

this Chapter.

If items 1-5 above are satisfied, the BMP area will be able to manage part or all of the WQv requirements for adjacent areas. The following table shows the relationship between the excess water storage volume in a soil profile to the size of an adjacent impervious surface which can be managed by the BMP in order to meet 100% of the WQv requirements for the adjacent impervious area. Keep in mind that open space areas without preserved healthy soils or where SQR techniques have not been applied to a depth of at least 4 inches should be treated as if they were 50% impervious (such areas will generate runoff during the WQv event which will need to be treated.

Table C5-S6- 8: Minimum BMP Area Ratios to Fully Manage WQv

Excess Water Storage Volume (inches)	Multiply BMP Area by This Factor to Determine Maximum Impervious Area to be Treated
0.50	0.42
1.00	0.84
1.25	1.05
1.50	1.26
2.00	1.68

Note the following when using Table C5-S6- 8:

1. Excess water storage is determined by finding the available water storage in a soil profile and subtracting 1.25 inches (the rainfall depth for the WQv event, which falls directly onto the BMP area).
2. Areas determined to have excess water storage volume, but that don't meet conditions 1-5 listed prior to Table 8, should not be included as part of the BMP area used to treat runoff from adjacent areas.
3. Upstream open spaces where healthy soils have not been verified and preserved or where SQR techniques have not been applied should be treated as 50% impervious area.
4. The relationship in Table 8 may be calculated for any value, by dividing the excess water storage volume by 1.1875 inches (runoff from a impervious surface generated by the 1.25 inch WQv event).

For example: A soil profile for a 10,000 square foot SQR area has been found to have 1.40 inches of excess storage volume.

$$= \frac{1.40in}{1.1875in} = 1.18$$

The upstream impervious area to be treated by the BMP can be up to 1.18 times as large as the BMP area. 1.18 x 10,000 SF = 11,800 SF. Therefore, the SQR area in this example can manage the WQv for 11,800 SF of upstream impervious area.

Guidance on Partial Credit for Managing WQv with These Techniques:
In cases where a preserved or SQR area to be used as a water quality BMP does not have sufficient area to fully manage the WQv for the upstream impervious area, partial credit may be given. In such a case, other practices will need to be distributed elsewhere on site to fully meet the WQv requirements. This can be done by calculating the WQv stored by the BMP, and determining the remaining volume to be addressed by other practices.

For example: A soil profile for a 10,000 square foot SQR area has been found to have 0.50 inches of excess storage volume. It receives runoff from 8,000 square feet of impervious surfaces, 4,000 square feet of compacted open space and 5,000 square feet of verified healthy open spaces which have been preserved.

1. The effective impervious area within the watershed of the SQR will be:

$$8000sf + (4000sf \times 50\%) + (5000sf + 0\%) = 8000 + 2000 + 0 = 10000sf$$

2. Calculate the required WQv for the adjacent impervious area (refer to Chapter 3, section 6):

$$R_v = 0.05 + 0.009(I) = 0.05 + 0.009(100) = 0.95$$

$$WQv = R_v(1.25in) = 1.1875\ watershed\ inches$$

$$= 1.1875in \times \frac{10000ft^2}{12\ in/ft} = 990ft^3$$

3. Determine the available water storage within the SQR area to serve as the BMP:

$$= 10000sf \times \frac{0.50in}{12\ in/ft} = 417ft^3$$

4. Determine the difference between available storage and required volume:

$$= 417ft^3 - 990ft^3 = -573ft^3$$

Therefore, 573 cubic feet of treatment volume would need to be provided by another practice within the site, in order to meet the site's WQv requirements.

Should there be found to be excess storage available within a given BMP, that surplus should not be used to offset for other impervious surfaces whose runoff leaves the site without measures to address the WQv. Having surplus storage in one area, will not offset the effects of allowing runoff from other areas to leave the site without treatment.

H. Stormwater Modeling Guidelines

Providing healthy soils through preservation or SQR techniques should reduce the amount of runoff generated from that area for all storm events. Conversely, leaving soils in a compacted condition or with a lack of organic matter will likely increase runoff volumes. This section provides guidance on how to apply these principles in stormwater modeling.

Rainfall losses (the amount of rainfall not converted into runoff) can be estimated by the proper selection of curve numbers. These curve numbers (CNs) should be selected based on the Hydrologic Soil Group listed in the County soil survey for the given area, unless site tests indicate another soil group should be used. For open spaces they should also be selected based on the cover type and based on the criteria below:

No Preservation or SQR Technique Implemented	Use POOR condition Healthy Soils Verified to
Minimum 4" Depth	Use FAIR condition Healthy Soils Verified to
Minimum 8" Depth	Use GOOD condition

Refer to Chapter 3, section 5 (NRCS TR-55 Methodology) to select the curve number that applies for the appropriate condition for the expected surface cover of a given area.

I. Construction Observation and Verification Requirements

Methods 1-2:

1. Prior to construction, complete testing methods to determine organic material content in areas proposed to be preserved. Verify that the areas to be preserved are protected by construction fences or other means deemed acceptable by the jurisdictional engineer.

2. Throughout construction, verify that no tracking, storage of materials or other disturbance is allowed within the protected area.

Methods 3-7:

1. Complete SWPPP management and inspections and install pollution prevention measures throughout construction (if required for a given site).

2. Compare site conditions with SMP.
3. When on-site topsoil is used, verify that the topsoil stockpile has been properly located and other site soils, debris, revetment stone or other materials are not being mixed with topsoil stockpile.

4. Verify surface, where SQR is to be completed, has been prepared and is free of debris, rocks larger than 1 inch in diameter (½ inch for turf grass areas) or other areas densely covered with smaller rocks and/or gravel.

5. Where topsoil is to be placed, observe site conditions, that the prepared surface is tilled to the required depth prior to topsoil placement and that it is not wet.

6. Refer also to other requirements of SUDAS Section 2010 related to the stripping, stockpiling and placement of topsoil. Verify that clods, lumps, roots, litter, other undesirable material, or stones larger than 1 inch (½ inch for turfgrass) have been removed prior to placement of any compost / sand or topsoil.

7. Observe that tillage is performed to the depth required. Do not allow wet soils to be tilled.

8. Use visual observation to determine topsoil is placed to the depth specified within the SMP.

9. Use visual observation and collect delivery tickets or tags to determine specified volume of compost is applied to the SQR area. Compare delivery tickets with the SMP to match delivery location, total quantity of material, product description and source of material. Any deviation from specified materials will require laboratory test results to verify that the delivered materials are equivalent to those specified.

10. Verify depth of amended soil and scarification by using a shovel to dig at least one test hole per acre (a minimum of one test hole on smaller sites). The test location should be randomly selected by the site observer. Test holes should extend at least 4 inches below the expected tillage depth and/or topsoil layer and be at least 1 square foot in area. The amended soils and/or topsoil layers should be easy to dig, driven solely by the weight of the observer. The soil should be darker than existing soil below. Particles of organic matter are likely to be visible. Soil that requires vigorous chipping with the shovel to penetrate properly does not meet the specification. Where topsoil has been placed, the next 2 inch depth of soil should be loose enough to penetrate with the shovel. The loosened depth may vary based on pattern of scarification, some sections of the 1 square foot hole should be loosened 2 inches below the topsoil layer. Collect samples from the test hole locations and have tests completed to determine that the organic material content assumed in design has been met or exceeded.

11. Use a rod penetrometer to confirm the soil is uncompacted to the desired tillage depth at a minimum of ten locations per acre (with a minimum of ten on sites less than one acre). Locate test spots by dividing the site (or each acre) in half lengthwise, then dividing each half into five nearly equal sections. Conduct the test near the middle of each section. The rod penetrometer should enter the soil through 2 inches below the amended soil depth and/or topsoil layer, driven solely by the weight of the observer. Irregular scarification or rocks in subsoils may require probing a few spots at each location.

12. Record the results of the shovel and penetrometer tests on a Field Verification Form to be included with Site Record Documents. If a given site does not fulfill the intent of the SMP, corrective action will need to be taken prior to site stabilization.

13. Perform seeding, sodding or other stabilization techniques as specified. Collect tickets or other information as needed to verify that the appropriate materials and application rates are being used.

14. Do not allow vehicular traffic, storage of materials or other disturbance within the SQR area during or after application of stabilization measures.

15. Continue SWPPP management and inspections and install pollution prevention measures until final stabilization. Should surface erosion occur, repair such areas with compost or appropriate topsoil-compost blends. Hand rake and reseed as necessary.

Method 8:

1. Verify that existing vegetation over the identified area for SQR is mowed to a height of 2 inches.

2. Observe the area has been aerated to a minimum depth of 4 inches.

3. Observe the area has been treated with a ½”-¾” layer of compost.

4. Use visual observation and collect delivery tickets or tags to determine that the appropriate volume of compost is applied to the SQR area. Compare delivery tickets to match delivery location, total quantity of material, product description and source of material with SMP. Any deviation from specified materials will require laboratory test results to verify that the delivered materials are equivalent to those specified.

5. Perform seeding as specified.

6. Collect tickets or other information as needed to verify that the appropriate seed and application rates was used.

J. Maintenance Requirements

1. Monitor weekly and after rains of 0.5 inches until vegetation is well established.

2. Long-term maintenance involves maintaining organic matter content. Leave lawn clippings on the yard to decompose and recycle nutrients and organic matter. Annual applications of ½-¾ inches of compost will help maintain or increase organic matter.

K. Reference Information

Soils for Salmon http://www.soilsforsalmon.org/
SUDAS http://www.soilsforsalmon.org/

*Hudson, Berman. 1994. Soil Organic Matter and Available Water Capacity. Journal of Soil and Water Conservation. 49(2), 189-194

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

Description: Native plants are those that grew naturally in Iowa before European settlement, and therefore are well-adapted to this environment. The tallgrass prairie ecosystem developed in Iowa over 10,000 years ago. It was an extremely diverse habitat that consisted of grasses, forbs (flowering plants), insects, and other animals. It adapted to survive conditions that ranged from hot and dry to moist and boggy in any given year; in addition to severe winters, frequent high winds, grazing by buffalo, and routine fire. The response to this ever-changing environment was the development of deep, fibrous root systems commonly reaching 6-12 inches deep. These root systems led to the development of Iowa's fertile soils, and can still contribute significantly to soil quality enhancement. Carefully chosen native plants can be used in a wide variety of infiltration and filtration practices to increase water quality. Landscaping with native plants provides color and habitat, and is an important component for engineered practices to capture and treat the water quality volume and the first flush of runoff from larger storms.

Typical uses:

- Used in conjunction with engineered water management practices.
- Used for runoff management from residential, commercial, and institutional sites.
- Used in rain gardens, bioretention areas, vegetated swales, and basins.

Advantages/benefits:

- Reduces runoff rate and volume from impervious areas in infiltration practices.
- Removes sediment and pollutants to improve water quality.
- Plants are beautiful, hardy, drought-resistant, and low-maintenance.
- Provides aesthetic value and habitat for beneficial insects and animals.
- Reduces the need for inputs from fertilizers, pesticides, water, and mowing.

Disadvantages/limitations:

- Maintenance techniques are not as widely-known as for turf grass.
- Establishment takes longer than turf grass, especially with seeded areas.

Maintenance requirements:

- Annual removal of vegetation growth through burning, or mowing and baling/raking.
- Plantings need to be watered and weeded regularly during establishment.
- Maintenance goes down after establishment (2-3 years).

A. Description

Landscaping with native plants is a simple way to obtain multiple benefits while mimicking the native ecosystems of the tallgrass prairie, oak savannas, woodlands, and wetlands. Native species are low- maintenance once established because they are adapted to Iowa temperatures, wind, and rainfall patterns. Properly-designed native landscaping can improve the value of the site, improve aesthetics, support wildlife, increase soil and water quality, and absorb noise.

Through plant uptake, plants can bind nutrients and other pollutants, and remove water through evapotranspiration. Pathways for rainfall infiltration will be created through root development, which also contributes to a healthy soil structure. Each year, a part of the deep root mass of native plant dies off and decomposes. This annual organic matter deposition helps build soil organic carbon, which in turn helps the soil absorb more water. Soil microbes help bind together particles of sand, silt, and clay, along with organic matter, creating a more granular soil structure, which increases porosity and water holding capacity. An additional benefit of the deep root system is seen when native plants also resist local pests and disease. Natives do not get as stressed as typical non-native species during droughts or other severe environmental conditions common in Iowa.

Native species bloom at a variety of times throughout the growing season and attract butterflies and birds often not seen in non-native landscapes. Native plants attract this variety of beneficial birds, butterflies, insects, and other wildlife by providing diverse habitats and food sources. Conversely, closely-mowed lawns are of little benefit to most wildlife.

B. Stormwater management suitability

Native plants are used in many areas that are designed to infiltrate and temporarily store the water quality volume (WQv).

C. Pollutant removal capabilities

Native plants do not require fertilizers or pesticides, and will eliminate their use when replacing sod lawns. Typically, lawns also require significant amounts of watering to survive, which often results in additional runoff of water and pollutants. Weekly emissions from lawn mowing equipment used on typical sod lawns also contribute to air pollution. Native plants remove carbon from the air and sequester it in the soil. Sequestered atmospheric carbon increases soil organic matter, which increases the soil’s ability to absorb water. Deeply-rooted native plants increase the soil’s capacity to store water and reduce water runoff.

D. Application and feasibility

Native landscaping plants can be used in most of the infiltration and filtration practices. They can also be used as a landscape amenity. Application and feasibility are dependent on the type of application. Various native plants were adapted to dry, mesic, or wet landscapes. Consider your landscape or type of practice and choose plant species that will work best.

1. **Dry soils.** Dry soils are typically found in well-drained, exposed areas. They are more common on south-facing slopes where it is warmest and driest during the summer.

2. **Mesic soils.** Mesic refers to areas that are well-drained, yet moist like a typical vegetable garden. Mesic sites are not overly wet or dry.

3. **Wet soils.** Wet sites often occur low on the landscape and have a high water table. Lists of recommended species and those to avoid are available on the Plant Iowa Native website: http://plantiowanative.com/ Additional guidance is provided in the SUDAS Landscaping Specification (Section 9010).

E. Planning and design criteria

Native prairie plantings can be established from seed or plugs (young, rooted plants). Plugs are better than seed in smaller projects in residential areas because they are easier to establish and maintain. Natives can be incorporated into an existing garden bed, or a new bed can be made by removing sod and loosening the soil. Try to avoid putting native plants in soils that have been fertilized, as this often results in overly tall growth far beyond typical for that species.

In residential settings, it is usually best to use shorter native plants to create an aesthetically-pleasing landscape. This means avoiding species that grow more than 4 feet tall, such as big blue stem, Indian grass, compass plant, and cup plant. Native plants can be designed into any sunny landscape, but rarely do well in deep shade. Woodland species can tolerate shade but often do not have deep root systems.

Native plants can be intermingled in more formal beds and borders, or incorporated as a more natural informal prairie garden. Turf borders should be left to define the area or provide a path through the planting.

Strategically-placed native species plantings can function similarly to engineered infiltration- and filtration-based practices. Choose plants based on site considerations for light, moisture, and soil. Vary plant structure, height, bloom succession, and flower color for seasonal appeal and butterfly habitat. After planting, a shredded-wood mulch layer helps establish natives by retaining moisture and discouraging weeds, but may float if water pools. A few small rocks can help overcome this problem during the establishment phase.

F. Design procedures

Design procedures generally involve matching plant or seed mix selection to the soils, moisture regimes, and aspect of a site. Plant height, color scheme, and shade or open sun tolerance all come into play in plant selection for a site. A number of native nurseries have pre-selected mixes for various conditions; care should be taken with those mixes to ensure the species are appropriate for the site and don't get overly tall. Native seed supplies often provide information on preferences of various species if you want to create your own blend. Species lists for plant suitable for Iowa native landscaping are online at: http://plantiowanative.com/ See the SUDAS Seeding Specification (9010) for more information.

Always plant mowed turf borders or low-growing native turf around native landscaping in an urban setting to provide a border and kept appearance. In plantings such as bio-retention areas, consider a border planting of shorter prairie grasses, such as prairie dropseed, just inside the turf border. Border plantings increase social acceptance of native landscaping sites. Design the planting to accommodate fire management whenever possible.

Develop all mixtures based on pure live seed. Exclude or keep aggressive grasses like switchgrass or other cultivars to a minimum, or eliminate entirely. If the site is within one mile of an existing native prairie (not a reconstructed prairie), local ecotypes are recommended.

1. **Using live plants.** On small urban plantings, it is usually best to buy live plants. Plants should be spaced 12-18 inches apart. Live plantings will establish more quickly than seed, and provide an aesthetically-pleasing site, usually in the first year.

2. **Seeding recommendations.** When seeding a mix of native plants, plan on it taking about three years to get good establishment (ground cover). Native plants spend the first year or two developing deep root systems before putting much energy into above-ground growth. Therefore, a good maintenance plan is essential to keep the site mowed and/or weeded, to protect plants from weedy competition, and avoid unsightly-looking areas that can turn public opinion against native landscaping.

3. **Native turf.** A new alternative showing promise for certain settings is the use of a native turf mix. A native turf features a blend of low-growing native grasses that would provide more a lawn-like appearance, while providing deep, fibrous root systems that will help build and maintain soil quality. Mowing on native turf plantings could be eliminated, and the height of the vegetation would stay in the 8 to 18-inch range. Or, mowing could be done on a limited basis (once a month or less). It should be noted that native grasses are warm-season grasses, which means they respond to the increased sunlight as days grow longer and hotter. Therefore, native turf will not break dormancy and green up as early in the growing season as cool season sod lawns. However, they will be green and growing during the long, hot days of summer when non- native cool season turf often goes dormant in response to the hot, dry conditions. Native turf will not need fertilization or watering after the root systems are established. A way to irrigate should be provided the first, and possibly the second, year to ensure good establishment; but after root development has been achieved, no more irrigation will be necessary.

G. Inspection and maintenance requirements

Native prairie plantings require less maintenance than turf grass and non-native gardens, but still need routine weeding and watering until established. Fertilizer is not recommended for prairie plantings, as it can stimulate excessive growth and cause plants to flop over. Dead vegetation should be removed in the fall or spring. Delaying this step until spring will allow winter landscape interest and provide seed and cover for over-wintering birds. Remove dead vegetation by burning, mowing, raking, and/or baling the residue.

(Note: The following procedures are adapted from the Wisconsin DNR Conservation Practice Standard 1002, and are recommended for use along with the design guidance provided in Chapter 5, section 1 for confirming the suitability of site soils for infiltration practices.)

A. Definition

The requirements in this section define the site evaluation procedures to:

1. Perform an initial screening of a development site to determine its suitability for infiltration.
2. Evaluate each area within a development site that is selected for infiltration.
3. Prepare a site evaluation report.

B. Purpose

1. Establish methodologies to characterize the site.
2. Establish requirements for siting an infiltration device and the selection of design infiltration rates.
3. Define requirements for a site evaluation report that ensures appropriate areas are selected for infiltration and an appropriate design infiltration rate is used.

C. Conditions where practice applies

These requirements are intended for development sites being considered for stormwater infiltration devices. Additional site location requirements may be imposed by other stormwater infiltration device technical standards.

D. Criteria

The site evaluation consists of four steps for locating the optimal areas for infiltration and properly sizing infiltration practices. Steps 1 and 2 are completed as soon as possible in the approval process.

- Step 1: Initial screening
- Step 2: Field verification of information collected in Step 1
- Step 3: Evaluation of specific infiltration areas
- Step 4: Soil and site evaluation reporting

The steps should coincide, as much as possible, for when the information is needed to determine the following:

- Potential for infiltration on the site
- Optimal locations for infiltration devices
- Final design of the infiltration device(s)

1. **Step 1: Initial screening.** The initial screening identifies potential locations for infiltration devices. The purpose of the initial screening is to determine if installation is limited by any of the general restrictions for infiltration practices (Chapter 5, section 1), and to determine where field work is needed for Step 2. Optimal locations for infiltration are verified in Step 2. Information collected in Step 1 will be used to explore the potential for multiple infiltration areas versus relying on a regional infiltration device. Smaller infiltration devices dispersed around a development are usually more sustainable than a single regional device that is more likely to have maintenance and groundwater mounding problems. The initial screening should determine the following:

    a. Site topography and slopes greater than 20%.

    b. Site soil infiltration capacity characteristics as defined in NRCS county soil surveys.

    c. Soil parent material.

    d. Regional or local depth to groundwater and bedrock. Use seasonally high groundwater information where available.

    e. Distance to sites identified as closed remediation sites within 500 feet from the perimeter of the development site.

f. Presence of endangered species habitat.

g. Presence of floodplains and flood fringes.

h. Location of hydric soils based on the USDA county soil survey and wetlands within or adjacent of the project area.

i. Sites where the installation of stormwater infiltration devices would not be recommended due to the potential for groundwater contamination as described below:
   1) An area within 250 feet of a private well.
   2) An area within 1000 feet of a municipal well.
   3) An area within 300 feet upslope or 100 feet downslope of karst features.
   4) A channel with a cross-sectional area equal to or greater than 3 ft^2 that flows to a karst feature.
   5) An area where the soil depth to groundwater or bedrock is less than 2 feet.
   6) Potential impact to adjacent property.
   7) A point system for initial evaluation of a site for infiltration practices is presented in Chapter 5, section 1.

2. **Step 2: Field verification of the initial screening.**
   a. Field verification is required for areas of the development site considered suitable for infiltration. This includes verification of Steps 1a, 1b, 1c, 1d, 1i3, 1i4 and 1i5.

   b. Sites should be tested for depth to groundwater, depth to bedrock and percent fines information to verify any exemption and exclusion found in Steps 1l and 1m. The following is a description of the percent fines expected for each type of soil textural classification: Several textural classes are assumed to meet the percent fines limitations for both 3- and 5-foot soil layers. These classifications would include the sandy loams, loams, silt loams, and all the clay textural classifications. Coarse sand is the only soil texture that, by definition, will not meet the limitations for a 3-foot soil layer consisting of 20% fines. Other sand textures and loamy sands may need the percent fines level verified with a laboratory analysis.

   c. Borings and pits should be dug to verify soil infiltration capacity characteristics and to determine depth to groundwater and bedrock.

   d. The following information should be recorded for Step 2:
      1) The date or dates the data was collected.
      2) A legible site plan/map that is presented on paper that is no less than 8 ½ x 11 inches in size and:
         - Is drawn to scale or fully-dimensional
         - Illustrates the entire development site
         - Shows all areas of planned filling and/or cutting
         - Includes a permanent vertical and horizontal reference point
         - Shows the percent and direction of land slope for the site or contour lines. Highlight areas with slopes over 20%.
         - Shows all floodplain information that is pertinent to the site
         - Shows the location of all pits/borings included in the report
         - Location of wetlands as field delineated and surveyed
         - Location of karst features, private wells within 100 feet of the development site, and public wells within 400 feet of the development site

      3) Soil profile descriptions are written in accordance with the descriptive procedures, terminology and interpretations found in the Field Book for Describing and Sampling Soils, USDA, NRCS, 1998. Frozen soil material must be thawed prior to conducting evaluations for soil color, texture, structure and consistency. In addition to the data determined in Step B, soil profiles must include the following information for each soil horizon or layer:
         - Thickness, in inches or decimal feet

- Munsell soil color notation
- Soil mottle or redox feature color, abundance, size, and contrast
- USDA soil textural class with rock fragment modifiers
- Soil structure, grade size, and shape
- Soil consistence, root abundance, and size
- Soil boundary
- Occurrence of saturated soil, groundwater, bedrock, or disturbed soil

3. **Step 3: Evaluation of specific infiltration areas.** This step is to determine if locations identified for infiltration devices are suitable for infiltration, and to provide the required information to design the device. A minimum number of borings or pits should be constructed for each infiltration device (Table 1). The following information should be recorded for Step 3:

   a. All the information under Step 1.

   b. A legible site plan/map that is presented on paper no less than 8 ½ x 11 inches in size and:
      - Is drawn to scale or fully dimensional
      - Illustrates the location of the infiltration devices
      - Shows the location of all pits and borings
      - Shows distance from device to wetlands

   c. An analysis of groundwater mounding potential is required as per Table 1. The altered groundwater level, based on mounding calculations, must be considered in determining the vertical separation distance from the infiltration surface to the highest anticipated groundwater elevation. References include, but are not limited to Bouwer (1999), Guo (1998, 2001), Hantuch (1967).

   d. One of the following methods should be used to determine the design infiltration rate:
      1) **Infiltration rate not measured.** Table 2 should be used if the infiltration rate is not measured. Select the design infiltration rate from Table 2 based on the least-permeable soil horizon 5 feet below the bottom elevation of the infiltration system.
      2) **Measured infiltration rate.** The tests should be conducted at the proposed bottom elevation of the infiltration device. Two procedures are recommended for the infiltration testing: Infiltration test column and double ring infiltrometer. The procedure of the infiltration test column is summarized below. If the infiltration rate is measured with a double-ring infiltrometer, the requirements of ASTM D3385 should be used for the field test.

   e. **Procedure for infiltration test column.**
      1) Install casing (solid 5-inch diameter, 30-inch length) to 24 inches below proposed BMP bottom (see Figure C5-S8- 1).
      2) Remove any smeared soiled surfaces, and provide a natural soil interface into which water may percolate. Remove all loose material from the casing. Upon the tester's discretion, a 2-inch layer of coarse sand or fine gravel may be placed to protect the bottom from scouring and sediment. Fill casing with clean water to a depth of 24 inches, and allow to pre-soak for 24 hours.
      3) After 24 hours, refill casing with another 24 inches of clean water, and monitor water level (measured drop from the top of the casing) for 1 hour. Repeat this procedure (filling the casing each time) three additional times, for a total of four observations or until there is no measurable change in the readings. Upon the tester's discretion, the final field rate may either be the average of the four observations, or the value of the last observation. The final rate should be reported in inches per hour.
      4) May be done through a boring or open excavation.
      5) The location of the test should correspond to the BMP location.
      6) Upon completion of the testing, the casings should be immediately pulled, and the test pit should be backfilled.

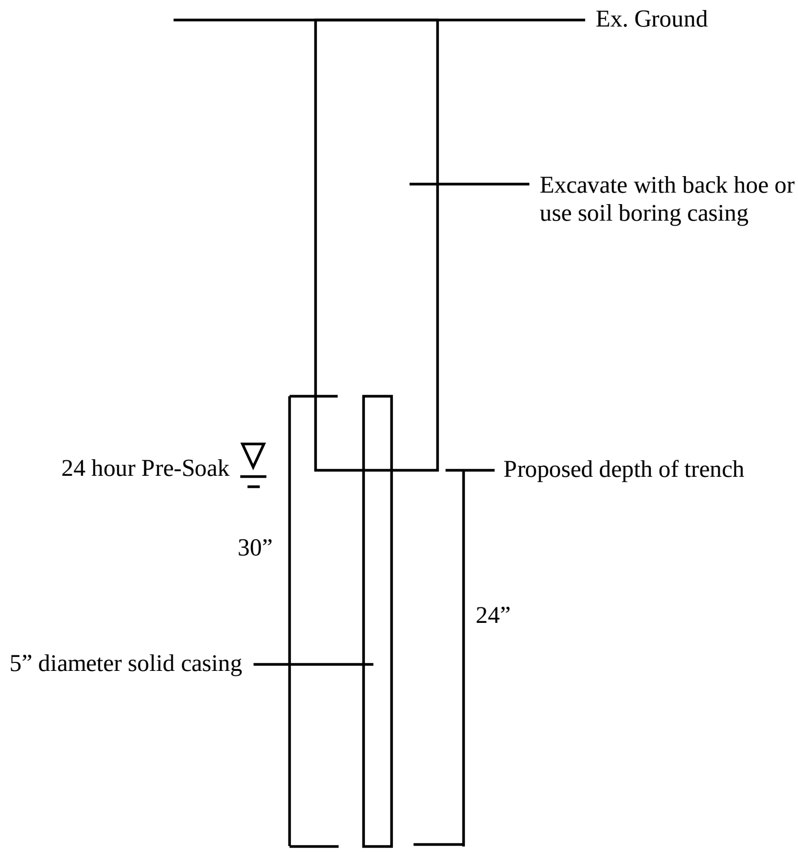


Figure C5-S8- 1: Infiltration testing requirements

The measured infiltration rate should be divided by a correction factor selected from Table 3. The correction factor adjusts the measured infiltration rates for the occurrence of less- permeable soil horizons below the surface and the potential variability in the subsurface soil horizons throughout the infiltration site.

A less-permeable soil horizon below the location of the measurement increases the level of uncertainty in the measured value. Also, the uncertainty in a measurement is increased by the variability in the subsurface soil horizons throughout the proposed infiltration site.

To select the correction factor from Table 3, the ratio of design infiltration rates must be determined for each place an infiltration measurement is taken. The design infiltration rates from Table 2 are used to calculate the ratio. To determine the ratio, the design infiltration rate for the surface textural classification is divided by the design infiltration rate for the least-permeable soil horizon. For example, a device with loamy sand at the surface and a least-permeable layer of loam will have a design infiltration rate ratio of about 6.8 and a correction factor of 4.5. The depth of the least-permeable soil horizon should be within 5 feet of the proposed bottom of the device or to the depth of a limiting layer.

Table C5-S8- 1: Evaluation requirements specific to proposed infiltration devices

BMP type	Design manual reference	Tests required[1]	Minimum number of borings/pits required	Minimum drill/test depth required below the infiltration surface (bottom of BMP)
Rain garden	2E-4	Pits or borings	NA[2]	5 feet or depth to limiting layer, whichever is less.
Infiltration trenches (<2000 ft^2 impervious drainage area)	2E-2	Pits or borings	1 test/100 linear feet of trench with a minimum of 2, and sufficient to determine variability	5 feet or depth to limiting layer, whichever is less
Infiltration trenches (>2000 ft^2 impervious drainage area)	2E-2	Pits or borings Mounding potential	1 pit required and an additional 1 pit or boring per 100 linear feet of trench, and sufficient to determine variability	Pits to 5 feet or depth to limiting layer Borings to 15 feet or depth to limiting layer
Bioretention systems	2E-4	Pits or borings	1 test per 50 linear feet of device with a minimum of 2, and sufficient to determine variability	5 feet or depth to limiting layer, whichever is less
Swales used for infiltration (dry or enhanced swale)	2I-3	Pits or borings	1 test per 1000 linear feet of swale with a minimum of 2, and sufficient to determine variability	5 feet or depth to limiting layer, whichever is less

[1]Continuous soil borings should be taken using a bucket auger, probe, split-spoon sampler, or Shelby tube. Samples shall have a minimum 2-inch diameter. Soil pits must be of adequate size, depth, and construction to allow a person to enter and exit the pit and complete a morphological soil profile description.
[2]Information from Step 2 is adequate to design rain gardens.

4. **Step 4: Soil and site evaluation report contents.** The site's legal description and all information required in Steps 2 and 3 should be included in the Soil and Site Evaluation Report. These reports are completed prior to the construction plan submittal.

Table C5-S8- 2: Design infiltration rates for soil textures receiving stormwater

Soil Texture[1]	Design infiltration rate without measurement[2] (inches/hr)
Coarse sand or coarser	3.60
Loamy coarse sand	3.60
Sand	3.60
Loamy sand	1.63
Sandy loam	0.5
Loam	0.24
Silt loam	0.13
Sandy clay loam	0.11
Clay loam	0.09
Silty clay loam	0.063
Sandy clay	0.05
Silty clay	0.04
Clay	0.02

[1]Use sandy loam design infiltration rates for fine sand, loamy fine sand, very fine sand, and loamy fine sand soil textures.
[2]Infiltration rates represent the lowest value for each textural class presented in Table 2 of Rawls, 1998.
[3]Infiltration rate is an average based on Rawls, 1982.

Table C5-S8- 3: Total correction factors divided into measured infiltration rates

Ratio of design infiltration rates[1]	Correction factor
1	2.5
1.1 to 4.0	3.5
4.1 to 8.0	4.5
8.1 to 16.0	6.5
16.1 or greater	8.5

[1]Ratio is determined by dividing the design infiltration rate (Table 2) for the textural classification at the bottom of the infiltration BMP by the design infiltration rate (Table 2) for the textural classification of the least permeable soil horizon. The least permeable soil horizon used for the ratio should be within five feet of the bottom of the BMP facility or to the depth of the limiting layer.

E. Additional considerations

Additional recommendations relating to design that may enhance the use of, or avoid problems with infiltration practices, but are not required to ensure its function are as follows:

1. Groundwater monitoring wells can be used to determine the seasonal high groundwater level. Large sites considered for infiltration basins may need to be evaluated for the direction of groundwater flow.

2. Cation exchange capacity (CEC) of the soil can indicate the number of available adsorption sites. Sandy soils have limited adsorption capacity and a CEC ranging from 1-10 meq/100g. Clay and organic soils have a CEC

greater than 20, and have a high adsorption rate.

3. Soil organic matter and pH can be used to determine adsorption of stormwater contaminants. A pH of 6.5 or greater is optimal. A soil organic content greater than 1 percent will enhance adsorption. (See SUDAS Section 2E-5).

4. One or more areas within a development site may be selected for infiltration. A development site with many areas suitable for infiltration is a good candidate for a dispersed approach to infiltration. It may be beneficial to contrast regional devices with onsite devices that receive runoff from one lot or a single source area within a lot, such as rooftop or parking lot.

5. Stormwater infiltration devices may fail prematurely if there is:
   a. An inaccurate estimation of the design infiltration rate
   b. An inaccurate estimation of the seasonal high water table
   c. Excessive compacting or sediment loading during construction
   d. No pretreatment for post-development and lack of maintenance
   e. No construction erosion should enter the infiltration device. This includes erosion from site grading, as well as homebuilding and construction. If possible, rope off areas selected for infiltration during grading and construction. This will preserve the infiltration rate and extend the life of the device.

6. The development site should be checked to determine the potential for archeological sites. This search may be conducted by state staff for projects required or funded by the state.

7. Slopes 20% or greater are inappropriate for some infiltration devices.

8. Expect to complete the preliminary design work (Steps 1 through 3) before the approval process (platting). Once required information is compiled, the initial design work for an infiltration device can begin.

9. The approval process requirements for development sites vary across the state, and may also vary within the jurisdiction, depending on the type of project (residential/commercial) or number of lots being developed. The timing of Steps 1, 2, and 3 will need to be adjusted for the type of approval process. Step 1 should be completed before the preliminary plat, and Step 2 should be completed before the final plat is approved. For regional infiltration BMP facilities, and for BMPs constructed on public right-of-ways, public land or jointly-owned land, Step 3 should be completed before the final plat approval.