# Chapter 2
# Design Criteria

This Chapter presents design criteria for sizing BMPs in the State of New Hampshire to protect the state's waters from the adverse impacts of development. Land development projects should employ site design and Best Management Practices (BMPs) to control peak runoff rates, provide stormwater quality treatment, use stormwater for groundwater recharge, and provide for stream channel protection.

For projects that must comply with the AoT Regulations, specific parameters for sizing BMPs to meet these requirements are stipulated in the Env-Wq 1500. This Manual recommends these parameters for all development projects.

This Chapter addresses the following design criteria for sizing stormwater management practices:

- Water Quality Volume (WQV)
- Water Quality Flow (WQF)
- Groundwater Recharge Volume (GRV)
- Effective Impervious Cover (EIC)
- Undisturbed Cover (UDC)
- Channel Protection (CP)
- Peak Control

A summary of the requirements is included in Table 2-1, with detailed descriptions provided in the text that follows. In addition to these design criteria, other BMP-specific criteria also apply to the design of stormwater management practices. Those additional criteria are provided for each BMP in Chapter 4 – Designing Best Management Practices. Each of the criteria listed in Table 2-1 is further discussed below.

**Table 2-1. Summary of Design Criteria[1]**

| Design Criteria | Description |
|---|---|
| Water Quality Volume (WQV) | WQV = (P)(Rv)(A)<br>P = 1" of rainfall<br>Rv = unitless runoff coefficient = Rv = 0.05 + 0.9(I)<br>I = percent impervious cover draining to the structure converted to decimal form<br>A = total site area draining to the structure |
| Water Quality Flow (WQF) | WQF = $(q_u)$(WQV)<br>WQV = water quality volume calculated in accordance with Design Criteria above<br>$q_u$ = unit peak discharge from TR-55 exhibits 4-II and 4-III<br>Variables needed for exhibits 4-II and 4-III:<br>Ia = the initial abstraction = 0.2S<br>S = potential maximum retention in inches = (1000/CN) – 10<br>CN = water quality depth curve number<br>= 1000/ (10+5P+10Q–10[$Q^2$ + 1.25(Q)(P)]$^{0.5}$)<br>P = 1" of rainfall<br>Q = the water quality depth in inches = WQV/A<br>A = total area draining to the design structure |
| Groundwater Recharge Volume (GRV) | GRV = $(A_I)(R_d)$<br>$A_I$ = the total area of effective impervious surfaces that will exist on the site after development<br>$R_d$ = the groundwater recharge depth based on the USDA/NRCS hydrologic soil group, as follows:<br>Hydrologic Group — Rd (inches)<br>A — 0.40<br>B — 0.25<br>C — 0.10<br>D — 0.00 |
| EIC & UDC | %EIC = area of effective impervious cover/total drainage area within a project area X 100<br>%UDC = area of undisturbed cover/total drainage area within a project area X 100 |
| Channel Protection (CP) | If the 2 yr, 24-hr post-development storm volume *does not increase* due to development then: control the 2-year, 24-hour post-development peak flow rate to the 2-yr, 24-hr pre-development level.<br>If the 2yr, 24-hr post development storm volume *does increase* due to development then:<br>Control the 2-yr, 24-hr post-development peak flow rate to ½ of the 2-year, 24-hr pre-development level or to the 1-yr, 24-hr pre-development level. |
| Peak Control | Post-development peak discharge rates can not exceed pre-development peak discharge rates for the 10 & 50-yr, 24-hr storm events. |

[1] Appendix A provides rainfall data for New Hampshire, for use with these design criteria.

## 2-1. Water Quality Volume (WQV)

### Criteria

The Water Quality Volume (WQV) is the amount of stormwater runoff from a rainfall event that should be captured and treated to remove the majority of stormwater pollutants on an average annual basis. The recommended WQV

is the volume of runoff associated with the first one-inch of rainfall, which is equivalent to capturing and treating the runoff from the 90th percentile of all rainfall. WQV should be calculated using the following equation:

**WQV = (P)(Rv)(A)**

Where:

P = 1 inch
Rv = the unitless runoff coefficient, Rv = 0.05 + 0.9(I)
I = the percent impervious cover draining to the structure, in decimal form
A = total site area draining to the structure

### Rationale

Development impacts the water quality of streams, ponds, lakes and wetlands. Pollutant deposits on the land surface increase as the intensity of land use increases. These materials are then washed off by rain and runoff, increasing the pollutant load to receiving waters. Usually, the stormwater that initially runs off an area, often referred to as the 'first flush' will be more polluted than the stormwater that runs off later, after the rainfall has 'cleansed' the catchment.

Based on early studies in Florida that determined that the first flush generally carries 90 percent of the pollution from a storm (Novotny, 1995), treatment of the first half-inch of runoff was adopted as a water quality volume sizing criterion throughout most of the United States. However, more recent research has shown that pollutant removal achieved using the half-inch rule drops off considerably as site imperviousness increases.

Other water quality sizing methods were developed to achieve higher pollutant removals, including the "90 Percent Rule", in which the water quality volume is equal to the storage required to capture and treat 90 percent of annual runoff and consequently 90 percent of the pollutant load. In the Northeastern United States, capturing 90 percent of the annual runoff is on average, roughly equivalent to capturing and treating the first one-inch of stormwater runoff for each rainfall event.

*Example Calculation: Water Quality Volume (WQV)*

Given

P = 1" for New Hampshire
A = 0.80 acres draining to the structure, 0.60 acres of this area is impervious

Solution

I = 0.60 ac/0.80 ac = 0.75
Rv = 0.05 + 0.9(I) = 0.05 + 0.9(0.75) = 0.725
WQV = (1")(0.725)(0.80ac) = 0.58 ac-in

To convert to cubic feet, if desired:
WQV = 0.58 ac-in * 43,560 $ft^2$/ac * 1ft/12 in
= **2,100 $ft^3$**

**WQV = 2,100 $ft^3$**

**Exhibit 4-II** Unit peal discharge ($q_u$) for NRCS (SCS) type II rainfall distribution

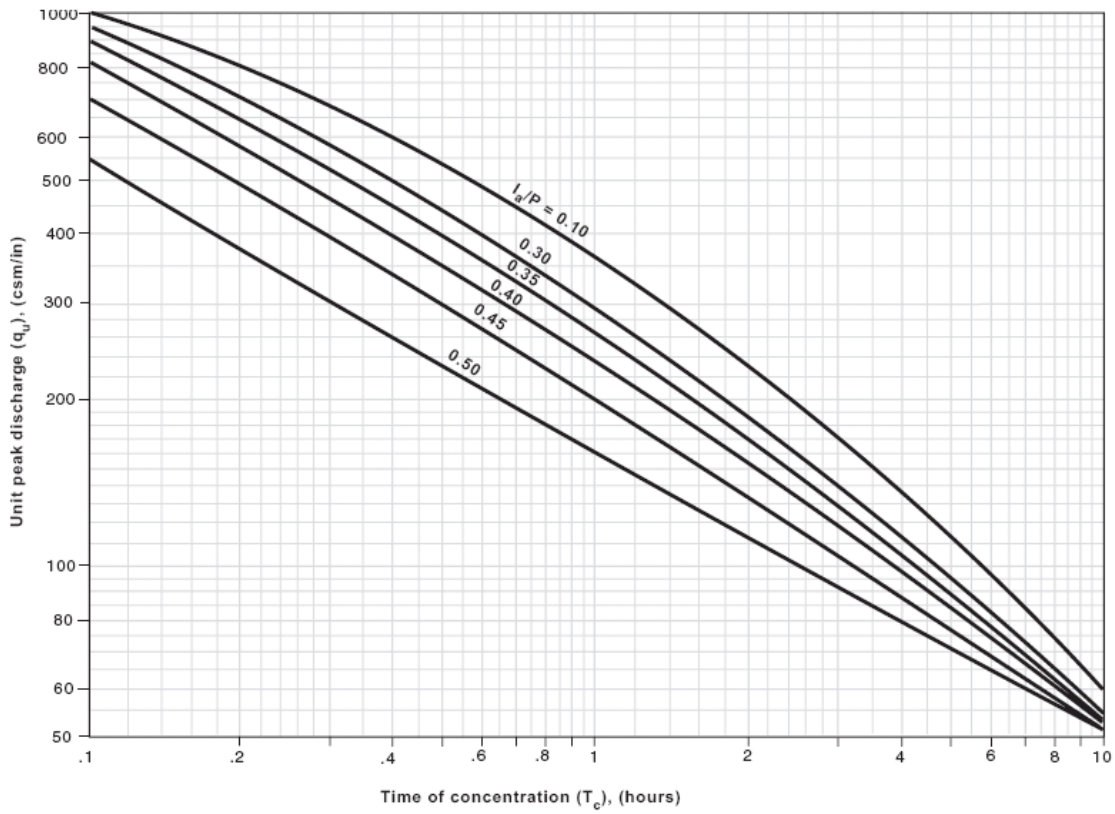


**Exhibit 4-III** Unit peal discharge ($q_u$) for NRCS (SCS) type III rainfall distribution

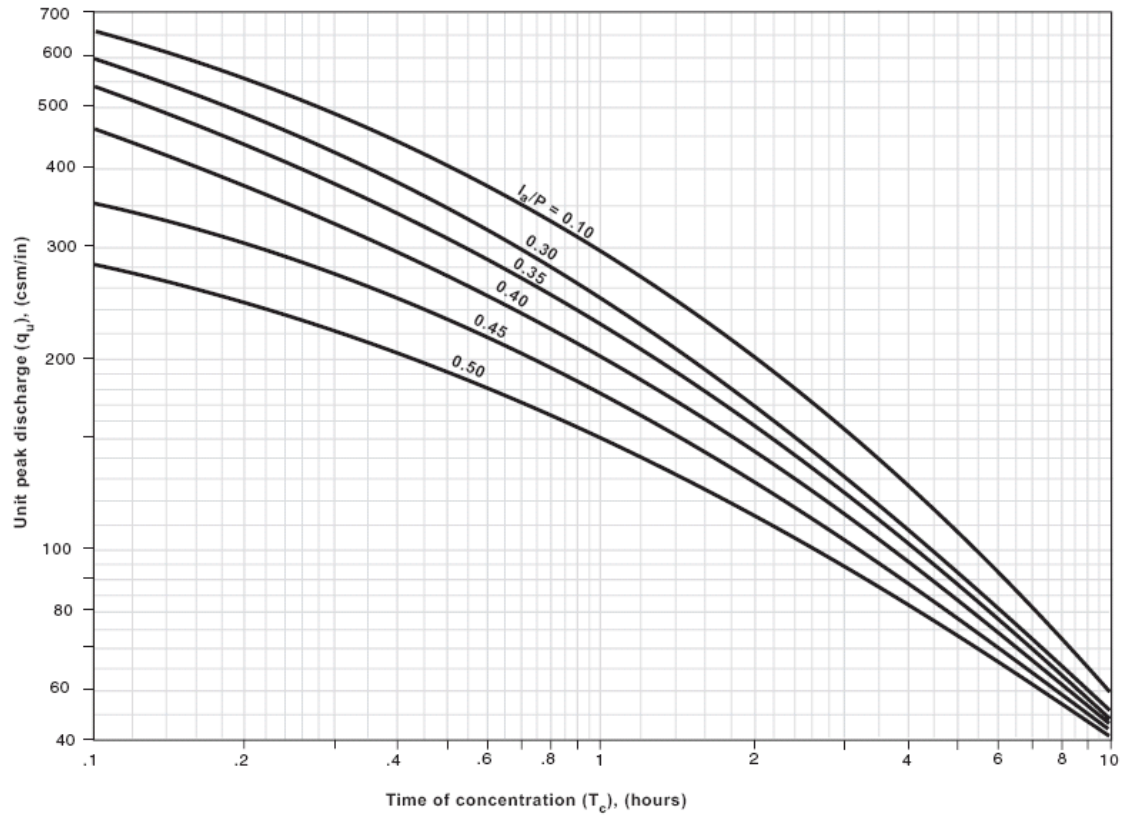


Figure 2-1. Exhibits 4-II and 4-III: Unit Peak Discharge for NRCS Rainfall Distributions

## 2-2. Water Quality Flow (WQF)

### Criteria

The Water Quality Flow (WQF) is used to determine a flow rate associated with the WQV, for sizing flow-based treatment and pre-treatment practices (e.g., Treatment Swales, Pre-treatment Swales, Flow-Through Devices – see BMP descriptions in Chapter 4). The WQF is calculated using the WQV and the Natural Resource Conservation Service (NRCS), TR-55 Graphical Peak Discharge Method. WQF should be calculated using the following equations and steps:

1. Compute the NRCS Curve Number (CN) using the following equation:

   $$CN = 1000/ (10+5P+10Q–10[Q^2 + 1.25(Q)(P)]^{0.5})$$

   Where:
   CN = Runoff Curve Number
   P = 1 inch
   Q = the water quality depth in inches = WQV/A
   WQV = water quality volume (calculations shown in previous section)
   A = total area draining to the design structure

   *NOTE that this CN is not the same as the subcatchment's CN which is selected based on the land use and soil type. Rather it is a representative CN used to convert the water quality depth to a flow rate.*

2. Compute the time of concentration (tc) using the methods described in Chapter 3 of TR-55.

3. Calculate potential maximum retention (S) in inches using the following equation:
   $$S = (1000/CN) – 10$$

4. Calculate initial abstraction (Ia) using the following equation:
   $$Ia = 0.2S$$

5. Read the unit peak discharge (qu) from TR-55 Exhibits 4-II or 4-III (reproduced below) based on the project's location.

6. Compute the water quality flow (WQF) using the following equation:
   $$WQF = (q_u)(WQV)$$

*Example Calculation: Water Quality Flow (WQF)*

Given

Location of project: Concord, New Hampshire
P = 1" for New Hampshire
A = 1 acre draining to the structure,
60% impervious cover
Tc = 12 minutes (0.2 hours)

Solution

WQV = (P)(Rv)(A)
Rv = 0.05 + 0.9(I) = 0.05 + 0.9(0.60) = 0.59
WQV = (1")(0.59)(1ac) = **0.59 ac-in**

Q = WQV/A = 0.59 ac-in / 1 ac = 0.59 in
*Note that Q is not a function of area, as it may appear. Q reduces down to:*
*Q= (P)(Rv)(A)/(A) = P x Rv so when P = 1",* ***Q = Rv****.*

$CN = 1000/ (10+5P+10Q–10[Q^2 + 1.25(Q)(P)]^{0.5})$
$= 1000/(10 + 5(1") + 10(0.59) – 10[0.59^2 + 1.25(0.59")(1")]^{0.5})$
= 95.4

S = (1000/CN) – 10
= 1000/95.4 – 10 = 0.48 in
Ia = 0.2S = 0.2(0.48) = 0.10 in
Ia/P = 0.10 in / 1 in = **0.10**

Concord is in the Type III rainfall distribution zone, therefore use exhibit 4-III from TR-55 and find qu in (csm/in) where Ia/P = 0.10 (solution above) and Tc = 0.2 hrs (given information) *Note csm = cfs/mi²*

$q_u$ = 560 cfs/mi²/in

**WQF** = ($q_u$)(WQV)
= 560 cfs/mi²/in x 0.59 ac-in x (1 mi²/640 ac)
**= 0.52 cfs**

**WQF = 0.52 cfs**

### Rationale

Some treatment practices such as treatment swales and flow-through devices are more appropriately designed based on peak flow rate, rather than water quality volume, since they are designed to treat higher flow rates, thereby requiring less storage volume. The use of the NRCS, TR-55 Graphical Peak Discharge Method in conjunction with the water quality volume is the preferable method for computing the peak flow associated with the water quality design storm, since it can more appropriately estimate peak flows associated with smaller storm events and can also be used to predict runoff volumes.

## 2-3. Groundwater Recharge Volume (GRV)

### Criteria

The purpose of the groundwater recharge volume criterion is to protect groundwater resources by minimizing the loss of annual pre-development groundwater recharge as a result of the proposed development. The Groundwater Recharge Volume (GRV) should be based on the site soils and the following equation:

$$\mathbf{GRV = (A_I)(R_d)}$$

Where:
$A_I$ = the total effective area of impervious surfaces that will exist on the site after development
$R_d$ = the groundwater recharge depth based on the USDA/NRCS hydrologic soil group, as follows:

| Hydrologic Group | $R_d$ (inches) |
|---|---|
| A | 0.40 |
| B | 0.25 |
| C | 0.10 |
| D | 0.00 |

The following criteria should also apply:

(a) If more than one soil type is present at the site, a weighted recharge depth should be computed based on the area of each soil group present.

(b) Infiltration rates for designing Groundwater recharge practices should be in accordance with Section 2-4 and the Alteration of Terrain regulations (Env-Wq 1500).

(c) No recharge is allowed within the setback areas provided in Table 3-3 or within 100 feet of a surface water that defines a water supply intake protection area, unless the recharge system receives stormwater from less than 0.5 acre and is not from a high-load area.

On some sites, existing soils or other conditions may severely constrain the use of infiltration systems for recharging groundwater. Examples include sites underlain by marine clays, sites in areas of karst geology, and urban redevelopment areas. In these areas, the recharge volume requirement may be reduced. However, stormwater management systems should still be provided to treat the full WQV and non-structural practices should be implemented to the maximum extent practicable to reduce runoff (e.g., filter strips that treat rooftop or parking lot runoff, sheet flow discharge to forested buffers, and grass channels that treat roadway runoff).

Additional requirements applicable to systems that infiltrate stormwater and that

*Example Calculation: Groundwater Recharge Volume (GRV)*

Given:

Total project area = 10 acres
Total impervious cover = 4 acres
Total effective impervious area = 1.5 acres

The effective impervious area will cover 1 acre of hydrologic group A soil and 0.5 acres of hydrologic group C soil.

Solution:

$GRV = (A_I)(R_d)$

Weighted $R_d$
= [(1 ac)(0.40 in) + (0.5 ac)(0.10 in)]/1.5 ac = 0.30 in

GRV = 1.5 ac * 0.30 in = **0.45 ac-in**

To convert to cubic feet, *if desired:*
GRV = **0.45 ac-in** * 43,560 $ft^2$/ac * 1ft/12 in
= **1,630 $ft^3$**

**GRV = 1,630 $ft^3$**

would contribute to groundwater recharge are listed with the specific BMP descriptions included in Chapter 4.

### Rationale

The groundwater recharge criterion is intended to maintain pre-development *annual* groundwater recharge volumes by capturing and infiltrating a portion of runoff from the post-development impervious surfaces for each individual storm event. Under this approach, a portion of runoff from larger storms, and all runoff from smaller precipitation events, is captured and infiltrated using appropriate BMPs.

The objective of the groundwater recharge criterion is to maintain water table levels, stream baseflow, and wetland moisture levels and to provide a filtering mechanism to "clean" surface water. Maintaining pre-development groundwater recharge conditions can also reduce the volume of runoff that must be managed to meet other design criteria (i.e., water quality, channel protection, and peak flow control), and thus the overall size and cost of stormwater management practices.

The objective of the groundwater recharge criterion is to mimic the average annual recharge that occurs on a site before it is developed. The recommended approach for calculating the GRV is a function of post-development site imperviousness and the prevailing infiltration capacity of existing soils. The hydrologic soil group approach uses the widely available NRCS Soil Survey maps and estimates of average annual infiltration rates for each hydrologic soil group. This method has been adopted in several other northeastern states with similar climates and average annual precipitation.

For each soils hydrologic group, the NHDES considers the recharge depth ($R_d$) the amount of runoff that must be captured from an impervious surface and infiltrated for each storm, in order to make up for the loss of recharge that would otherwise result from that impervious surface. For example, if a site development creates impervious surfaces on an area with soils in Hydrologic Group A, then for every storm event, the stormwater system should capture and infiltrate the first 0.4 inches of runoff from all pavements and roofs; for small storm events that generate less than 0.4 inches of runoff, the system should capture and infiltrate all runoff from the new impervious surfaces. The cumulative effect of capturing and infiltrating the initial volume of runoff from multiple events is to approximate the annual recharge occurring during pre-development conditions.

## 2-4. Design Infiltration Rate

Chapter 4 presents information on a number of Best Management Practices that rely on stormwater infiltration (e.g.; infiltration practices, filtering practices, and groundwater recharge practices). This section outlines the procedures for selecting a design infiltration rate.

### Site Feasibility Confirmation Testing

Initial screening identifies the potential for using infiltration methods and determines potential locations on the site for infiltration facilities. Initial screening establishes the feasibility of installation of infiltration methods on the site and identifies where fieldwork may be needed for subsequent field verification.

## » INITIAL SCREENING PARAMETERS

The initial stormwater infiltration screening evaluation involves seven screening parameters, to identify site-specific characteristics of the proposed development site. Information regarding the following seven parameters should be obtained and evaluated relative to applicable regulations, the BMP descriptions provided in Chapter 4, and the guidelines discussed in this Chapter:

1. Site topography and slopes greater than 15%.
2. Site hydrologic soil groups or Ksat values. If a site specific soil map as defined in accordance with the Society of Soil Scientists of Northern New England (SSSNNE) Special Publication No. 3, Site-Specific Soil Mapping Standards for New Hampshire and Vermont, December 2006 (or most recent), has been created for the developed site area, this will be very useful in the initial screening process.
3. Potential depth to bedrock and seasonal high water table (SHWT).
4. Presence of potentially vulnerable groundwater areas (Water Supply Well Setback areas, Groundwater Protection Areas, and Water Supply Intake Protection Areas).
5. Presence or nearby proximity to known areas with identified soil or groundwater contamination, including but not limited to:
    - Existing or closed remediation sites, or
    - underground storage tanks within or adjacent to the project parcel.
6. Presence of sensitive ecological habitat (including wetlands and threatened or endangered species habitat).
7. Presence of flood plains.

The following list of soils typically have infiltration rates too rapid to provide treatment; therefore, stormwater should already be treated prior to discharging to these soils or these soils should be field tested to ensure that the infiltration rate, prior to adding a factor of safety, is less than 10 inches per hour. If the infiltration rate is greater than 10 inches per hour, the soils should be amended to an infiltration rate less than 10 inches per hour and then field tested to confirm a maximum infiltration rate of 10 inches per hour:

**Abenaki, Adams, Agawam, Boscawen, Caesar, Champlain, Colton, Croghan, Deerfield, Haven, Hermon, Hinckley, Hoosic, Metallak, Quonset, and Warwick.**

*Standardized Test Pit/Boring Protocol*

Test pits and/or borings are required in the infiltration area to a minimum depth of 5 feet below the proposed bottom of the infiltration facility. The following steps describe the main elements necessary to support test pit/boring requirements:

1. Excavate a test pit or drill a boring to a depth of at least 5 feet below the proposed facility bottom or to the depth of bedrock or the SHWT, whichever is less. Test pits should be of adequate size, depth, and construction to allow a person to enter and exit the pit and complete a morphological soil profile description. If borings are drilled, continuous soil borings should be taken using a bucket auger, probe, split-spoon sampler, or Shelby tube. Samples should have a minimum 2-inch diameter. A minimum number of test pits and/or borings should be provided for each infiltration facility as designated in Table 2-2).
2. Determine depth to SHWT (if potentially within 5 feet below the base of the facility).
3. Determine US Department of Agriculture (USDA) or Unified Soil Classification (USC) System soil textures at the proposed bottom and to 5 feet below the bottom of the infiltration facility.
4. Describe soil horizons and determine depth to bedrock (if within 5 feet of proposed bottom of facility).
5. The location of the test pit or boring should correspond to the BMP location; test pit/soil boring stakes should be clearly labeled and left in the field for inspection and surveyed location.

## » FIELD VERIFICATION

Field verification of information collected during the initial site feasibility screening process includes further investigation of specific areas on a development site that have been considered potentially suitable for infiltration.

Sites should be tested for depth to SHWT and depth to bedrock to verify findings from initial screening.

For existing soils, natural or man-made, test pits or borings should be performed to verify soil infiltration capacity characteristics and to determine depth to the SHWT and depth to bedrock. A standardized test pit/boring protocol is described below.

The following information should be recorded for field verification of the potential sites as a result of the initial screening:

1. The date or dates the data were collected.
2. A legible site plan/map that:
   a. Is drawn to scale.

b. Illustrates the entire development site.
c. Shows all areas of planned filling and/or cutting.
d. Includes a permanent vertical and horizontal reference point.
e. Shows the percent and direction of land slope for the site or contour lines, and highlights areas with slopes over 15%.
f. Shows all flood plain information that is pertinent to the site.
g. Shows the locations of all test pits/borings included in the report.
h. Shows the locations of wetlands as field delineated and surveyed.
i. Shows the locations of water supply wells and setbacks, groundwater protection areas, and water supply intake protection areas if within 100 feet of the development site.

3. It is recommended that soil profile descriptions be written in accordance with the descriptive procedures, terminology, and interpretations found in the "USDA Field Book for Describing and Sampling Soils" (USDA NRCS 2002, or most recent). In addition to the soil data determined above, soil profiles should include the following information for each soil horizon or layer:
   a. Thickness, in inches or decimal feet.
   b. Munsell soil color notation.
   c. Soil redoximorphic feature color, abundance, size, and contrast.
   d. Using the USDA Textural Triangle, soil textural class with rock fragment modifiers
   e. Soil structure, grade size, and shape.
   f. Soil consistence
   g. Root abundance and size.
   h. Soil boundary.
   i. Occurrence of saturated soil, groundwater, bedrock, or disturbed soil.

NOTE: If the material is frozen, it should be thawed prior to conducting evaluations for soil color, texture, structure and consistency.

**Table 2-2. Minimum Number of Test Pits/Borings Required**

| Facility | Minimum Number of Test Pits / Borings Required |
|---|---|
| Infiltration Basins<br>Less than 2,500 sf | 1 test |
| Infiltration Basins<br>2,500 sf or more | 2,500 sf – 20,000 sf = 2 tests<br>20,000 sf – 30,000 sf = 3 tests<br>30,000 – 40,000 = 4 tests<br>1 additional test for every additional 10,000 sf. |
| Infiltration Trenches | 0 LF – 100 LF = 1 test<br>100 LF – 200 LF = 2 tests<br>200 LF – 300 LF = 3 tests<br>1 additional test for every additional 100 LF. |

## » EVALUATION OF SPECIFIC INFILTRATION AREAS

At specific locations identified for stormwater infiltration facilities, this step consists of soils evaluation to confirm that the locations are suitable for infiltration and provide the required information to design the facilities. The

minimum number of test pits and/or borings should be provided for each infiltration facility as discussed above.

The following information should be recorded for this evaluation:

1. All the information obtained in initial screening and field verification steps.

2. A legible site plan/map that:
   a. Is drawn to scale or fully dimensional;
   b. Illustrates the locations of the proposed infiltration facilities;
   c. Shows the locations of all test pits and borings; and
   d. Shows distance to wetlands.

3. The results and supporting information for one of the following methods used to determine the design infiltration rate:

*A. Default Rate*

Default values may be used for native materials only. Default values maybe easier to obtain, however the designer should note that this method is considered conservative. To select a default rate, first use the Site Specific soil map and determine which soil series are at the location of the practice.

*Example: Selecting a Default Infiltration Rate*

Given:

Location: Concord, NH (Merrimack County)
Elevation of the bottom of the proposed filtering practice: 24 inches below native ground.
SSSS mapped as: 166B - Canterbury and test pits confirm this

Solution Steps:

1. Go to the Merrimack County soil data in NRCS's Soil Data Mart located on the web at: http://soildatamart.nrcs.usda.gov and look up the ranges of values for saturated hydraulic conductivity reported for the Canterbury soil layers at or below the bottom of the infiltration system or contact your local NRCS office.

Result:

| Depth (inches) | Ksat (micrometers/second) | Ksat (inches/hour) |
|---|---|---|
| 0-2 | 4.0 - 42.0 | 0.6 - 6.0 |
| 2 - 6 | 4.2 - 14.1 | 0.6 - 2.0 |
| 6 - 28 | 4.2 - 14.1 | 0.6 - 2.0 |
| 28 - 65 | 0.42 - 4.2 | 0.06 - 0.6 |

Conversion: 1 micrometer/second = 0.1417 inches/hour

Second, determine the limiting layer (slowest Ksat) reported beneath the proposed bottom of the practice using the Physical Soil Properties reported by the USDA NRCS. The reported Ksat for a given layer typically has a range of values. Select the slowest value for the default rate. Use a weighted average by area if more than one soil series is present. Lastly, apply a minimum factor of safety of 2.

1. Select the slowest value reported below the bottom of the practice:
   Results: The limiting layer, at or below 24", is 0.06 inches per hour.

2. Apply a factor of safety
   Result: design infiltration rate = 0.06 inches per hour/2 = 0.03 inches per hour.

*B. Field Measured Infiltration Rate*

For the purposes of determining a design infiltration rate for stormwater BMPs a saturated hydraulic conductivity (Ksat) test should be performed with the following testing protocol:

- The Ksat should be measured with a Guelph Permeameter; a Compact Constant Head Permeameter; a Double-Ring Infiltrometer (ASTM 3385), where the inner ring is at least 12 inches in diameter; or a Borehole Infiltration test, see Table 2-3 for the testing protocol.
- The test should be performed and/or supervised by a qualified professional such as a certified soil scientist, a professional geologist, or an engineer.
- The test location should be within the footprint of the final location of the infiltration facility.
- The test should be conducted at the proposed bottom elevation of the infiltration facility.
- See Table 2-4 below for the minimum number of testing locations
- If a Guelph Permeameter or Compact Constant Head Permeameter test is used, the test should be performed a minimum of 3 times for each test location.

*Example: Determining a Field Measured Infiltration Rate*

Field testing results on natural soils using a compact head parameter are as follows:

Test location A.

Run 1 = 2.2 inches per hour
Run 2 = 5.8 inches per hour
Run 3 = 2.4 inches per hour
Run 4 = 3.8 inches per hour
Run 5 = 1.4 inches per hour
Average = 3.1 inches per hour

Testing Location B.

Run 1 = 1.2 inches per hour
Run 2 = 2.4 inches per hour
Run 3 = 1.9 inches per hour
Average = 1.8 inches per hour

Testing Location C.

Run 1 = 7.4 inches per hour
Run 2 = 8.1 inches per hour
Run 3 = 9.4 inches per hour
Average = 8.3 inches per hour

Average of the tests
= (3.1 + 1.8 + 8.3 )/ 3
= 4.4 inches per hour

Result: design infiltration rate
= 4.4 inches per hour/2
= 2.2 inches per hour

| Table 2-3. Borehole Infiltration Test Protocol |
|---|
| **Infiltration Testing Requirements** |
| 1. Install casing (solid 4 - 6 inch diameter, 30" length) to 24" below proposed bottom of the practice (see Figure 2-1). |
| 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 two (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" and allow to pre-soak for twenty-four hours |
| 3. Twenty-four hours later, refill casing with another 24" 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. The observations should be averaged. |
| 4. May be done though a boring or open excavation. |
| 5. Upon completion of the testing, the casings should be immediately pulled, and the test pit should be back-filled. |

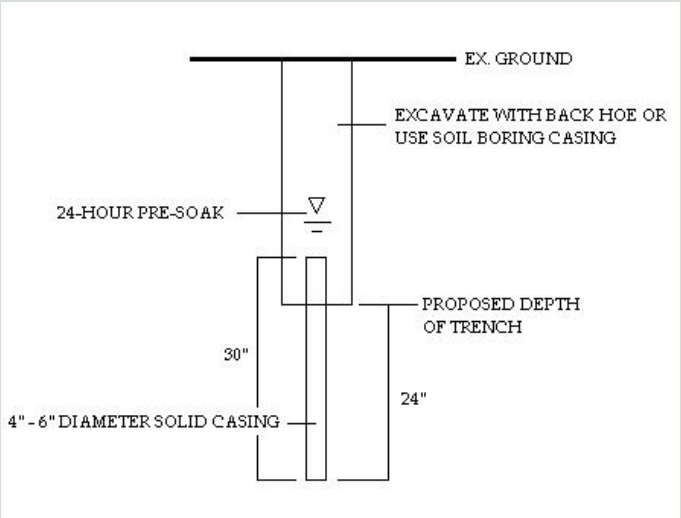


Figure 2-2. Borehole Infiltration Test Setup

- Test locations should be located on plans by survey.
- Final infiltration testing data should be documented, and include a description of the infiltration testing method. This is to ensure that the tester and reviewer fully understand the procedure.
- Apply a minimum factor of safety of 2 to the field measured infiltration rate. See example below:

*C. Lab Measured Infiltration Rate*

The following protocol should only be used for initial design for proposed fill material:

- The Ksat should be measured with test methods described in ASTM D-2434, "Standard Test Method for Permeability of Granular Soils (Constant Head)" or ASTM D-5856, "Standard Test Methods for Measurement of Hydraulic Conductivity of Porous Material Using a Rigid-Wall, Compaction-Mold Permeameter";
- Apply a minimum factor of safety by dividing the representative Ksat by 2.0 and use the result as the design infiltration rate.
- Once the fill is in place, the soil should be field tested to confirm the design rate. To confirm the rate, run the field test in accordance with section B. above.

## » LIMITATIONS & CONSIDERATIONS

The following limitations on discharging stormwater into the ground should be recognized.

Infiltration practices, Unlined Filtering Practices, and Groundwater Recharge Practices should not be installed in the following areas:

1. Within groundwater protection areas, where the stormwater comes from a high-load area;
2. Within areas that have contaminants in groundwater above the ambient groundwater quality standards established in Env-Or 603.03
3. Within areas having soil above site-specific soil standards developed pursuant to Env-Or 600;
4. In any area, if the stormwater comes from areas that have contaminants in soil above site-specific soil standards developed pursuant to Env-Or 600;
5. In any area, if the stormwater comes from areas with underground storage tanks regulated under RSA 146-C or aboveground storage tanks regulated under RSA 146-A, where gasoline is dispensed or otherwise transferred to vehicles;
6. Within areas having slopes greater than 15%, unless the system has been carefully engineered to prevent seepage forces from causing instability;
7. Within areas where the design infiltration rate is less than 0.50 inches per hour. For filtering practices such as a bioretention area or permeable pavement, no minimum infiltration rate should be required if these facilities are designed with a "daylighting" underdrain system.
8. Within areas having soils with infiltration rates greater than 10 inches per hour) unless the stormwater has first been treated by an acceptable BMP, or the soil has been amended to reduce the infiltration rate and the reduction is confirmed by further testing.

Infiltration into fill soils should be used with extreme caution!

**Table 2-4. Minimum Number of Test Locations**

| Facility | Minimum Number of Test Pits / Borings Required |
|---|---|
| Infiltration Basins (no manmade soils present) | 1 test for each 2,500 sf of basin area |
| Infiltration Basins (manmade soils present) | 1 test for each 1,000 sf of basin area |
| Infiltration Trenches (no manmade soils present) | 1 tests for each 100 LF of trench |
| Infiltration Trenches (manmade soils present) | 1 tests for each 50 LF of trench |

The following should be considered to enhance the use of, or avoid problems with, an infiltration facility:

1. Groundwater monitoring wells can be used to determine the seasonal high water table. Large sites considered for infiltration systems may need to be evaluated for the direction of groundwater flow.

2. 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. 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.

3. 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 compaction or sediment loading during construction;
   d. Inadequate pretreatment of post-development stormwater flows;
   e. Inadequate maintenance of the infiltration system and pretreatment facilities.

4. No construction-related sediment should enter the infiltration device. This includes sediment resulting from initial site grading as well as subsequent home building and related 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. In addition, infiltration facilities should only be placed into service after the contributing areas are fully stabilized.

## 2-5. Effective Impervious Cover (EIC) and Undisturbed Cover (UDC)

Volume 1, Chapter 5 of the Stormwater Manual describes the concepts of Effective Impervious Cover and Undisturbed Cover. These parameters are used to determine the applicability of proposed Antidegradation Requirements, as discussed in Volume 1.

NHDES has proposed a target of 10% effective impervious cover (%EIC) maximum and a 65% undisturbed cover (%UDC) minimum for development sites to be used as a surrogate to conducting pollutant loading analysis. This is informally called the "1065 Rule." It is proposed that eligible sites[1] that meet the 1065 Rule do not have to perform a loading analysis under the antidegradation requirements.

1 The "1065 Rule" pertains to Tier 2 – High Quality Waters that have useable assimilative

- %EIC – The percent effective impervious cover (%EIC) is computed by dividing the area of effective impervious cover within a project area by the drainage area within a project area, using equal units of measure, and then multiplying the result by 100.
- %UDC – The undisturbed cover (%UDC) is computed by dividing the area of undisturbed cover within a project area by the drainage area within a project area, using equal units of measure, and then multiplying the result by 100.

## 2-6. Channel Protection (CP)

### Criteria

The purpose of this design criterion is to protect stream channels, downstream receiving waters, and wetlands from erosion and associated sedimentation resulting from urbanization within a watershed. This criterion limits the total amount of time that a receiving stream exceeds an erosion-causing threshold based on pre-developed conditions. Off-site flows, or flows into receiving channels within the project area, must meet one of the following criteria to satisfy channel protection requirements:

1. If the 2 year, 24-hour post-development storm volume has not increased over the pre-development volume, then control the 2-year, 24-hour post-development peak flow rate to the 2-year, 24-hour pre-development peak flow rate.
2. If the 2 year, 24-hour post-development storm volume has increased over the pre-development volume, then control the 2-year, 24-hour post-development peak flow rate to 50 percent of the 2-year,

*Example Calculation: Channel Protection (CP)*

Given:

Prior to development, stormwater is collected immediately off-site to one point of analysis (POA). After development, all of the stormwater is collected at the same POA after being treated and recharged, as necessary.

1 year pre-development 24-hour peak discharge rate
= 1.4 cfs with 0.24 ac-ft of runoff

2 year pre-development 24-hour peak discharge rate
= 2.6 cfs with 0.34 ac-ft of runoff

Solution options:

If the 2 yr post-development runoff volume at the POA is ≤ 0.34 ac-ft
Then: the 2 yr post-development 24-hour peak discharge rate should be
≤ **2.6 cfs.**

If the 2 yr post-development runoff volume at the POA is > 0.34 ac-ft
Then: 2 yr post-development 24-hr peak discharge rate should be
≤ ½ x 2.6 cfs ≤ **1.3 cfs**
or the 2 yr post-development 24-hr peak discharge rate should be ≤ **1.4 cfs**

capacity remaining. Volume 1, Section 5-2 includes more information on project eligibility for this surrogate measure.

24-hour pre-development peak flow rate or to the 1-year, 24-hour pre-development peak flow rate.

### Rationale

One of the earliest and most common methods developed to protect stream channels involved the control of post-development peak flows associated with the 2-year, 24-hour storm event to pre-development levels. More recent research indicates that this method does not adequately protect stream channels from erosion and may actually contribute to erosion, since banks are exposed to more frequent and longer duration of erosive bankfull events (MacRae, 1993 and 1996, McCuen and Moglen, 1988).

This is illustrated in Figure 2-3, which compares typical hydrographs for an undeveloped site, the same site developed with no control of peak rates, and the developed site with facilities to attenuate peak rates. As expected, the uncontrolled post-development hydrograph shows a higher peak runoff rate and greater volume of runoff than the pre-development hydrograph. To control peak rates, attenuation facilities are designed to store runoff and release it over an extended period, in order to control the release rate to pre-development levels. While this controls the rate, the period of time during which the receiving water experiences the flow is extended. The extended duration is significant, because flows approaching and larger than the 2-year storm comprise the erosive, channel-forming events. The net result is that receiving channels experience greater erosion due to the increased frequency and duration of bankfull events. The Channel Protection criterion addresses this condition.

## 2-7. Peak Runoff Control

### Criteria

The purpose of peak runoff controls is to address increases in the magnitude of flooding caused by development. The following criteria should be met to control peak discharge rates and improve the overall effectiveness of the stormwater treatment systems:

1. The 10-year, 24-hour post-development peak flow rate should not exceed the 10-year, 24-hour pre-development peak flow rate for all flows leaving the site;

2. The 50-year, 24-hour post-development peak flow rate should not exceed the 50-year, 24-hour pre-development peak flow rate for all flows leaving the site;

3. The project should provide supporting information showing that there is no impact to properties as a result of developing within the100-year floodplain;

4. The design must ensure that the conveyance system and land grading direct runoff to the peak control structure for all pertinent storm events. On some sites, detention facilities are designed for one storm event, while pipes are designed for a different event. For example, the control structure may be designed for the 25-year storm, while the drainage system may only be sized to handle a ten-year storm, with larger storms flooding the distribution system and traveling overland. In this case, the design should ensure that this overflow will be directed into the peak control structure;

5. On some sites, stormwater enters the site from adjacent property. If this stormwater must be handled by the project's drainage system, then the system design and supporting calculations should account for this condition for each design storm, in both pre- and post-development conditions;

6. The design should provide for an emergency spillway for any peak rate control structure that requires an embankment (dam). The emergency spillway's purpose is to protect against embankment failure, in the event the primary outlet cannot handle flows discharging form the impoundment (see description of Detention Basin in Chapter 4).

7. Use NRCS (formerly SCS) methods (TR-20 or TR-55) to develop hydrographs and peak flow rates for the proposed development site. The hydrograph time interval (dT) in TR-20 should be no greater than 0.1 hours. All areas should be accounted for in the pre/post

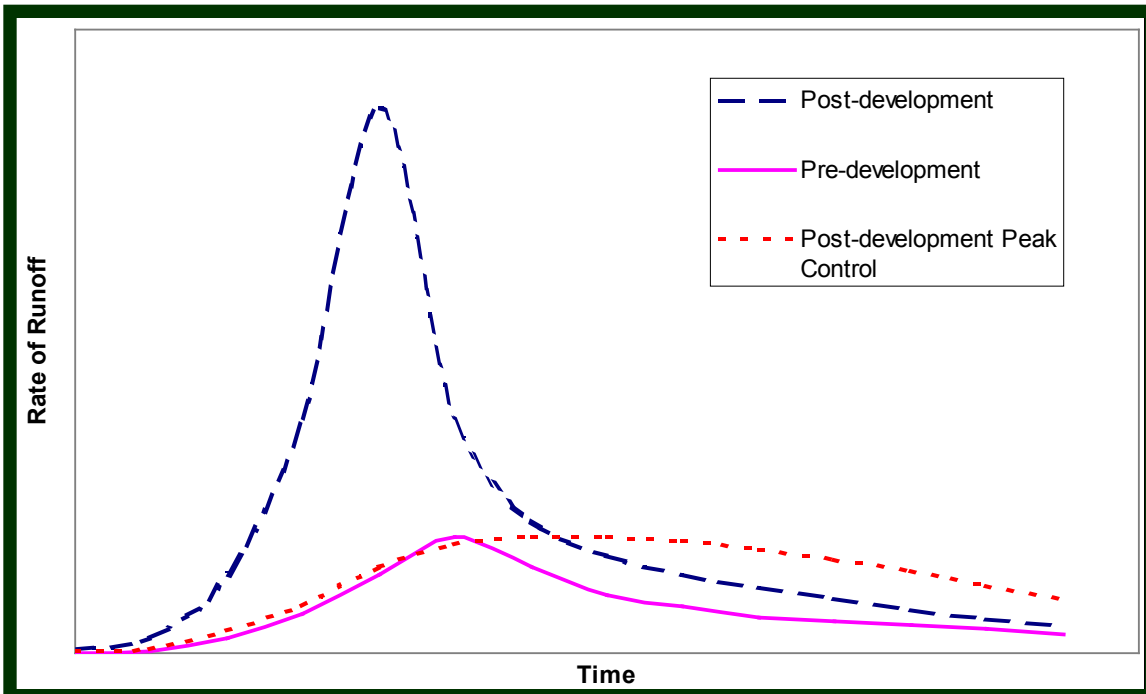


Figure 2-3. Pre- and Post-Development Hydrographs

runoff calculations. The total tributary area that contributes flow to the proposed site, including runoff entering the site through piped drainage or surface runoff from off-site sources, should be included even if a portion does not contribute flow to the site BMPs. The objective is for the development's storm drain design to account for total runoff leaving the site;

8. Any site that was wooded within the last ten years should be considered undisturbed woods for all pre-construction runoff conditions, regardless of clearing or cutting activities that may have occurred on the site during that pre-application period;

9. For all areas that are not modeled in "good" condition, photo documentation should be obtained.

10. Off-site areas should be modeled as present land use condition for all design storm events for both pre and post development calculations; and

11. The length of overland sheet flow used in time of concentration (tc) calculations should be limited to no more than 100 feet for pre- and post-development conditions.

In general, peak runoff controls as described in 1) and 2) above may not be necessary if the project area abuts and discharges to a large receiving waterbody. This typically can be shown through off-site drainage calculations for the 10-year and 50-year, 24-hour storm, showing that at a point immediately downstream from the project site, the post-development peak flow rate from the site and the off-site contributing area does not exceed the pre-development peak flow rate at that point.

### Rationale

This criterion is generally consistent with storm drainage system design in New Hampshire, with some added provisions to help guide the design of peak attenuation structures.

The provision to consider any site that was wooded within the last ten years as undisturbed woods for all pre-construction runoff conditions is incorporated to address properties that are cleared with an intent to develop, before the development application process is triggered. Without this provision, the pre-development peak discharge rate may be overestimated, since cleared land produces more runoff than forested land, resulting in a lesser degree of control when the development actually occurs.