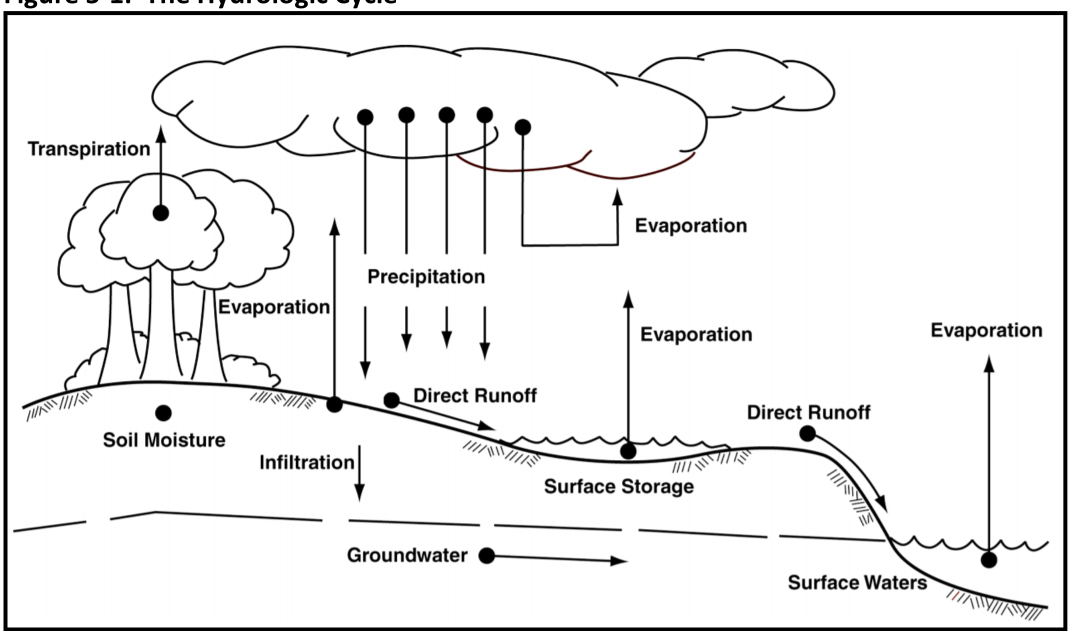
5. STORMWATER MANAGEMENT QUANTITY AND QUALITY STANDARDS AND COMPUTATIONS

This chapter discusses the fundamentals of computing stormwater runoff rates and volumes from rainfall using various mathematical methods. To do so effectively, the chapter also describes the fundamentals of the rainfall-runoff process that these methods attempt to simulate. Guidance is also provided in the use of the Natural Resources Conservation Service (NRCS) method, the Rational Method and the Modified Rational Method that are specifically required by the NJDEP Stormwater Management rules at N.J.A.C. 7:8 *et seq*.

Fundamentals of Stormwater Runoff

In general, stormwater runoff can be described as a by-product of the interaction of rainfall with the land. This interaction is one of several processes that the earth's water may go through as it continually cycles between the land and the atmosphere. This cyclical process is scientifically known as the *hydrologic cycle*. Stormwater runoff is only one of many forms water may take. Figure 5-1 below depicts the primary forms that water can take during the hydrologic cycle and the various processes that produce these forms. In addition to runoff, these processes include precipitation, evaporation from surfaces or the atmosphere, evapotranspiration by plants and infiltration into the soil and or groundwater. As such, water that precipitates as rainfall can wind up, or at least spend time, on ground or plant surfaces, in the atmosphere, within the various soil layers or in waterways and water bodies.

Figure 5-1: The Hydrologic Cycle



Source: Fundamentals of Urban Runoff Management.

The physical processes that convert rainfall to runoff are both complex and highly variable. As such, these processes cannot be replicated mathematically with exact certainty. However, by making simplifying assumptions and using empirical data, there are several mathematical models and equations that can simulate these processes and predict resultant runoff volumes and rates with acceptable accuracy. Before any of the computation methods can be discussed, it is necessary to define two terms used extensively throughout this chapter.

Time of concentration – As defined in N.J.A.C. 7:8-2.4(g)4, time of concentration is the time it takes for runoff to travel from the hydraulically most distant point of the drainage area to the point of interest within a watershed.

Hydrograph – In the context of a stormwater runoff analysis, the graph depicting the flow rate of runoff versus the time passed at a specific point of analysis is a hydrograph. A hydrograph can provide much information about stormwater runoff, including the time of concentration, the time at which peak flow occurs, the peak flow rate and the volume of runoff generated.

In general, all runoff computation methods are mathematical expressions attempting to replicate the hydrologic cycle. Many hydrological models have been developed to compute the flow rate or volume of the runoff from an individual event. However, the Stormwater Management rules at N.J.A.C. 7:8-5.7 allow only the following three modeling methodologies to be used, and each will be discussed, including any drainage area limitations, in later sections of the chapter:

1. The USDA Natural Resources Conservation Service (NRCS) methodology, including the NRCS Runoff Equation and Dimensionless Unit Hydrograph as described in *Chapters 7, 9, 10, 15* and *16*, *Part 630 Hydrology*, *National Engineering Handbook* (*NEH*), may be used for the computation of runoff volume, peak flow rate of runoff and hydrograph of runoff resulting from specific precipitation depths. This methodology was previously described in *Technical Release 55--Urban Hydrology for Small Watersheds* (*TR-55*), dated June 1986; however, it has been superseded by the aforementioned chapters of the *NEH*. Information regarding the *NEH*, *Part 630 Hydrology,* is available from the United States Department of Agriculture website at:

   https://directives.sc.egov.usda.gov/viewerFS.aspx?hid=21422 or

   at United States Department of Agriculture Natural Resources Conservation Service, 220 Davison Avenue, Somerset, New Jersey 08873.

2. The Rational Method may be used for the computation of peak flow rate under specific rainfall intensity.

3. The Modified Rational Method may be used for hydrograph computations, which can be further utilized for the computation of runoff volume for a specific rainfall intensity and the required storage volume of a detention BMP. The modified rational method is discussed further online at:

   http://www.nj.gov/agriculture/divisions/anr/pdf/2014NJSoilErosionControlStandardsComplete.pdf.

Predicting Storm Events

Even though precipitation events are, by nature, random in their duration and rainfall depths, historical data shows that large storm events occur less frequently than small storm events. No one can predict exactly when a certain size storm event will occur. However, through a frequency analysis of rainfall depths and intensities from past precipitation events, one can determine the likelihood of a storm occurrence using probability analysis.

The rainfall depth and intensity of past precipitation events are sorted into a probability distribution that gives the likelihood of the occurrence of different sized events.

- For example, a storm event producing a rainfall depth of 3.5 inches or greater has about a 50% chance of happening in a given year whereas a storm event with a rainfall depth of 8.5 inches or greater that has only a 1% chance of occurring in the same given year.

The probability of the occurrence of a certain size of storm event can be alternatively expressed as a recurrence interval, which is the inverse of the probability.

- For example, the recurrence interval of a rainfall event that has a 50% chance of occurrence in a given year is expressed as the 2-year (= 100 ÷ 50) recurrence interval, which is also known as the 2-year storm.

- For a storm event with a 1% chance of occurrence, it has a 100-year (= 100 ÷ 1) recurrence interval and is referred to as the 100-year storm.

Referring to a precipitation event as the "X-year storm" does not mean that this storm can only happen once every X years. Nor does it mean that a larger storm event cannot also occur that year. The table below lists the probability of a particular occurrence and its corresponding chance of occurring, expressed as a percentage, in a single year.

Recurrence Intervals and Probabilities of Occurrences		
Recurrence Interval, in years	**Probability of Occurrence in any Given Year**	**Percent Chance of Occurrence in any Given Year**
100	1 in 100	1
50	1 in 50	2
25	1 in 25	4
10	1 in 10	10
5	1 in 5	20
2	1 in 2	50

Another aspect of the frequency analysis is the duration of rainfall events. The frequency analysis may use the rainfall depths observed in events having various durations of precipitation, such as 1 hour, 6 hours, or even 3 days, although a 24-hour duration is typically used.

There are many organizations that collect and publish hydrological data, such as National Oceanic and Atmospheric Administration's (NOAA) National Weather Service (NWS). NOAA's NWS publishes and updates hydrological data and frequency analysis of rainfall depth and intensity constantly, under normal operating conditions. The National Engineering Handbook (*NEH*) produced by the NRCS uses NWS data due to its availability and lengths of record. Therefore, in this chapter, NWS data is referenced in the calculations involving the rainfall depths and intensities for the 2-, 10- and 100-year storm events. A more detailed discussion of using NWS data is found beginning on Page 12.

Regulatory Requirements of the Stormwater Management Rules

The Stormwater Management rules set forth stormwater runoff quantity, stormwater runoff quality and groundwater recharge standards for stormwater runoff generated by major developments as defined in N.J.A.C. 7:8-1.2. These projects must demonstrate compliance with those standards, as follows.

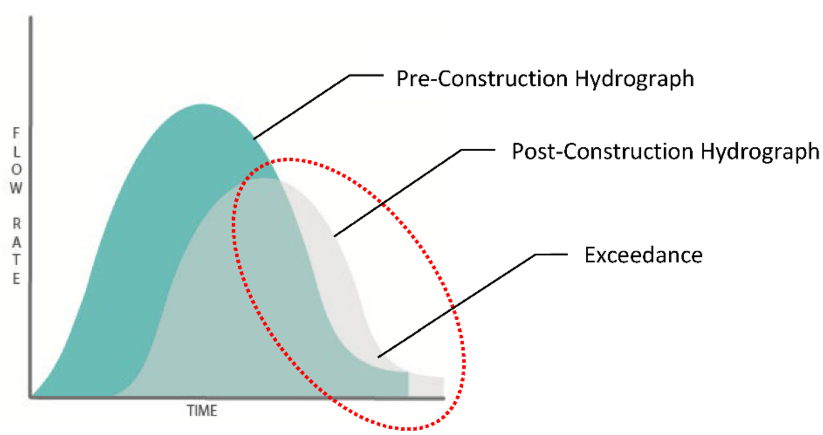
Stormwater Runoff Quantity Control Design and Performance Standards

In order to control stormwater runoff quantity impacts, the design engineer shall use the assumptions and factors for stormwater runoff calculations at N.J.A.C. 7:8-5.7(a). Unless the project is granted a variance pursuant to N.J.A.C. 4.6(a)3.ix, or is exempted pursuant to 5.2(d) or 5.6(b)4, the design engineer must demonstrate the compliance of the quantity standards in one of the three options in N.J.A.C. 7:8-5.6(b)1 to 3:

i. *Demonstrate through hydrologic and hydraulic analysis that for stormwater leaving the site, post-construction runoff hydrographs for the two-, 10- and 100-year storm events do not exceed, at any point in time, the pre-construction runoff hydrographs for the same storm events.*

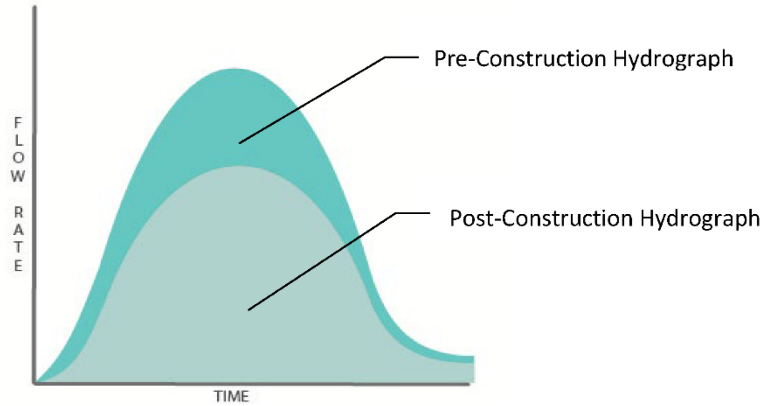
   Below is an illustration demonstrating noncompliance with the requirement under N.J.A.C. 7:8-5.6(b)1, followed on the next page by a second image demonstrating compliance:

Figure 5-2: Post-Construction Hydrograph Exceeds the Pre-construction Hydrograph



In the preceding graphic, the peak of the post-construction hydrograph, shown in grey, is lower than the peak of the pre-construction hydrograph, shown in teal, and some points of the post-construction hydrograph lie outside the pre-construction hydrograph, shown within the dashed oval area; therefore, the post-construction hydrograph does not meet the requirements set forth at N.J.A.C. 7:8-5.6(b)1.

Figure 5-3: Post-Construction Hydrograph does not Exceed the Pre-construction Hydrograph at any Point



In the above graphic, the post-construction hydrograph meets the aforementioned requirement since every point of the post-construction hydrograph is under the pre-construction hydrograph.

It is important to note that the area under the hydrograph represents the volume of the stormwater runoff. In order to comply with this option for meeting the stormwater runoff quantity standards, the post-construction runoff volume must be equal to or lower than the pre-construction runoff volume. Otherwise, the post-construction hydrograph will exceed the pre-construction hydrograph at some point.

ii. *Demonstrate through hydrologic and hydraulic analysis that there is no increase, as compared to the pre-construction condition, in the peak runoff rates of stormwater leaving the site for the two-, 10- and 100-year storm events and that the increased volume or change in timing of stormwater runoff will not increase flood damage at or downstream of the site. This analysis shall include the analysis of impacts of existing land uses and projected land uses assuming full development under existing zoning and land use ordinances in the drainage area.*

This demonstration requires the following calculations and demonstrations be provided, at a minimum:

- Calculation of pre- and post-construction conditions for the 2-, 10- and 100-year storms, where post-construction peak flow rates leaving the site must not be higher than the pre-construction peak flow rates leaving the site.

- A hydrologic and hydraulic analysis of the receiving waterbody, which demonstrates that the increased volume of stormwater runoff and/or change in timing from pre- to post-construction conditions for the 2-, 10- and 100-year storms does not result in increased flood damage at or downstream of the project. This should be conducted for both of the following scenarios:

- ☐ Pre-construction conditions and post-construction conditions with the project calculations based on the existing land uses.
- ☐ Pre-construction conditions and post-construction conditions with the project calculations based on the assumption of full development in the drainage area allowed by existing zoning and land use ordinances.

iii. *Design stormwater management measures so that the post-construction peak runoff rates for the two-, 10- and 100-year storm events are 50, 75 and 80 percent, respectively, of the pre-construction peak runoff rates. The percentages apply only to the post-construction stormwater runoff that is attributable to the portion of the site on which the proposed development or project is to be constructed.*

Under the third option, the design engineer may use stormwater management measures, either nonstructural and/or structural, to control the post-construction peak flow rates to be 50, 75 and 80 percent of the pre-construction peak flow rates for the 2-, 10- and 100-year storms, respectively.

The methodologies allowed under N.J.A.C. 7:8-5.7 are discussed in the section which begins on Page 9.

Applicability of Stormwater Runoff Quantity Control Standards

- For municipal review under the requirements of the Municipal Separate Storm Sewer System (MS4) permits, the threshold under which a project is considered to meet the definition of major development is dependent upon each municipality's adopted stormwater management ordinances(s). According to N.J.A.C. 7:8-4.2(a), major development reviewed under Municipal Stormwater Control Ordinances is limited to projects that ultimately disturb one or more acres of land. However, municipal ordinances can be more stringent than the requirements of the Stormwater Management rules, but cannot be less restrictive. The Residential Site Improvement Standards (RSIS), under N.J.A.C. 5:21 *et seq*., allow municipalities to require stormwater runoff controls for development falling below the major development threshold to address groundwater recharge and stormwater runoff quantity control, but not for stormwater runoff water quality control.

- In accordance with N.J.A.C. 7:8-5.6(b)4, in *tidal flood hazard areas, stormwater runoff water quantity analysis in accordance with N.J.A.C. 7:8-5.6(b)1, 2 and 3 is required unless the design engineer demonstrates through hydrologic and hydraulic analysis that the increased volume, change in timing, or increased rate of the stormwater runoff, or any combination of the three will not result in additional flood damage below the point of discharge of the major development*. This provision, however, does not provide a blanket exemption from having to provide stormwater quantity control requirements for the sites located in the tidal flood hazard area. It, instead, requires a demonstration that there are no increases in flood damages below the point of discharge by the increased volume of stormwater runoff before the quantity control requirement stated in N.J.A.C. 7:8-5.6(b)1, 2 and 3 can be waived.

  - ☐ For example, when a site located in a tidal flood hazard area discharges stormwater runoff directly into a bay, there is no increase of the water level or flood damage below the point of discharge. Therefore, the project is not required to meet the stormwater quantity control requirement.

- ☐ However, if a site located in a tidal flood hazard area will discharge the runoff so that it flows over or past a neighboring property before reaching the tidal water, the stormwater runoff from the site could increase flood damages to the neighboring property. This project will be required to meet the quantity control requirement.
- ☐ Similarly, if the stormwater runoff from a site will discharge to a storm sewer or other conveyance, meaning it will flow past or through other properties before reaching the tidal water, the stormwater discharge could increase flood damages below the point of discharge. Under such circumstances, the stormwater runoff quantity control requirement must be satisfied.

The demonstration analysis is not required when the stormwater is discharged directly into any ocean, bay, inlet or the reach of any watercourse between its confluence with an ocean, bay or inlet and downstream of the first water control structure.

- Stormwater runoff from agricultural development meeting the definition of major development must meet the performance standards established in these rules. Development on agricultural land means: any activity that requires a State permit, any activity reviewed by the County Agricultural Boards (CAB) and/or the State Agricultural Development Committee (SADC) and any activity that requires municipal review that is not exempted by the Right to Farm Act, N.J.S.A. 4:1C-1 *et seq*. This does not conflict with the Right to Farm Act, which recognizes the State's continuing authority to regulate agricultural development at N.J.S.A. 4:1C-9.

- "Disturbance" means the placement or reconstruction of impervious surface or motor vehicle surface, or exposure and/or movement of soil or bedrock or clearing, cutting, or removing of vegetation. Milling and repaving is not considered disturbance for the purposes of this definition. Milling and/or repaving of an existing impervious surface that will not expose or move soil or bedrock beneath the existing surface do/does not count as disturbance or redevelopment and do/does not trigger the Stormwater Management rules, provided there are no changes to the existing stormwater drainage system. The reconstruction of these areas, however, does constitute disturbance.

- N.J.A.C. 7:8-5.6(c) requires that the *stormwater runoff quantity standards shall be applied at the site's boundary to each abutting lot, roadway, watercourse or receiving storm sewer system*. Stormwater quantity control requirements are applicable to each discharge point leaving the boundary of the development site separately unless the stormwater runoff generated by different areas within the site converge into one discharge point before leaving the development site.

Conditions Regarding the Use of Exfiltration in Stormwater Runoff Routing Computations

Exfiltration can be used in the design of the small-scale green infrastructure BMPs, as listed in Table 5-1 of N.J.A.C. 7:8-5.3(f). Exfiltration, meaning discharge of runoff into the subsoil, may be included in stormwater runoff routing computations under certain conditions, provided all of the conditions, as outlined below, are satisfied.

1. All soil testing must be fully compliant with *Chapter 12: Soil Testing Criteria* of this manual.

2. The design of the BMP must comply with all of the design criteria within the respective subchapter of *Chapter 9* of the BMP Manual.

3. **Pretreatment**, in the form of a forebay or any of the other BMPs found in the BMP Manual, **must be incorporated into the BMP design, unless specifically stated otherwise** in the corresponding subchapter of the BMP Manual. This pretreatment requirement does not apply to BMPs with a contributory drainage area of 1 acre or less, except when pretreatment is a design requirement even without using exfiltration in the routing (such as a subsurface infiltration basin.)

4. Exfiltration cannot be used in any BMP designed with an underdrain system, since the runoff discharged through the underdrain will be discharged to the down-gradient surface water or sewer system and will not be infiltrated into the subsoil.

5. Infiltration of the **entire** 2-, 10- or 100-year storm is allowed only when:

    a. existing site conditions are such that no runoff leaves the site for the pre-construction condition scenario, thereby constraining the design to infiltrate 100% of the volume produced by the post-construction condition for the same design storm. In this case, the maximum storm that can be entirely infiltrated is the largest storm event with no runoff leaving the site in pre-construction conditions, or

    b. the volume of stormwater runoff to be fully infiltrated is required by law or rule implemented by the Pinelands Commission, Highlands Council, or any other stormwater review agency with jurisdiction over the project.

6. The analysis of groundwater hydrology and the hydraulic impact due to the exfiltration, required pursuant to N.J.A.C. 7:8-5.2(h), must be conducted in conjunction with the design using exfiltration. The design soil permeability rate, also referred to herein as the design vertical hydraulic conductivity, of the most hydraulically restrictive soil horizon below an infiltration type BMP may be used as the exfiltration rate in the routing calculations only when the soil is tested strictly in accordance with *Chapter 12*. This analysis must be performed using the method outlined in *Chapter 13: Groundwater Table Hydraulic Impact Assessments for Infiltration BMPs*.

7. The runoff volume discarded as exfiltration and the design vertical hydraulic conductivity of the most hydraulically restrictive soil horizon below an infiltration BMP must be used, in the initial model, to calculate the duration of infiltration period in the groundwater mounding analysis. The groundwater mounding analysis has determined that an adverse impact will occur if the resulting groundwater mounding reaches the bottom of the BMP or if the temporary localized increase in the water table encroaches upon a building or another structure, including any septic systems. When an adverse impact is the result, further modifications to the size of the infiltration area of the BMP or reductions in the exfiltration rate must be performed until the adverse impacts are eliminated. Further, when the groundwater mounding reaches the bottom of the BMP, the hydraulic gradient is reduced, thereby reducing the exfiltration rate. To reflect the impact on the hydraulic gradient, the reduced exfiltration rate must also be used to re-run the routing calculation(s) to check the peak flow rate(s) produced for the respective design storm(s) through the proposed outlet structure of the infiltration BMP used to meet the Stormwater Runoff Quantity Standards. If adverse impacts cannot be avoided, the infiltration BMP cannot be used.

For additional information on performing the groundwater mounding analysis, see *Chapter 13: Groundwater Table Hydraulic Impact Assessments for Infiltration BMPs* of this manual. Examples 5-6 and 5-7, which begin on Page 44, illustrate the methodology to be used.

Stormwater Runoff Computation Methods

The following is an introduction to the computation methods allowed by the Stormwater Management rules, followed by a brief overview of any limitations an individual method may have and the respective drainage area limits for each of these methods. The chapter will then provide separate detailed discussions, with examples, for each of the methods allowed. Special page headers have been incorporated into this portion of the chapter to indicate the method under discussion. As stated above, for the purposes of managing potential flooding, stormwater runoff quantity and quality, plus groundwater recharge issues, it is essential to calculate the volume and peak flow of the stormwater runoff produced by a storm event. N.J.A.C 7:8-5.7 states the following methods are the only methods acceptable for use in the computation of stormwater runoff:

1. The U.S. Department of Agriculture NRCS methodology, for which the discussion begins on Page 10, and
2. The Rational Method for peak flow, beginning on Page 70, along with the Modified Rational Method for hydrograph computations, beginning on Page 75.

The selection of an appropriate method depends upon the limitation(s) of the method under consideration:

- The NRCS method can provide total stormwater runoff volume, the peak flow rate and produce hydrographs. Under the NRCS method, different synthetic rainfall distributions and unit hydrographs can be applied to produce the stormwater runoff hydrograph in accordance with geographical differences that may affect the rainfall pattern in each storm event and the runoff pattern in a region, depending on whether the topographic slope is steep or flat. Further discussion of rainfall distributions and unit hydrographs are found beginning on Page 17.
- The Rational Method can be used to produce estimates of peak runoff rates, but it cannot provide total stormwater runoff volumes nor produce hydrographs.
- The Modified Rational Method can be used for the calculation of runoff volume.

Limitations on the size of the drainage area must also be taken into consideration:

- The NRCS method can be used for a drainage area of any size , but the area is still subject to the N.J.A.C. 7:8-5.7(a)4 requirement that the relative stormwater runoff rates and/or volumes of pervious and impervious surfaces be separately considered to accurately compute the rates and volume of stormwater runoff from the drainage area.
- The Rational Method and Modified Rational Method can be used in a single drainage area measuring 20 acres or less.

A table is provided on Page 81 summarizing the applicability of the methods discussed in this chapter and how the methods are to be used.

NRCS Methodology

The NRCS methodology is perhaps the most widely used method for computing stormwater runoff rates, volumes and hydrographs. It uses both a hypothetical design storm and an empirical nonlinear runoff equation to compute runoff volumes and as well as a dimensionless unit hydrograph to convert the volumes into runoff hydrographs. The methodology is particularly useful for comparing pre- and post-development peak rates, volumes and hydrographs. The key component of the NRCS runoff equation is the NRCS Curve Number (CN), which is based on soil permeability, surface cover, hydrologic condition and antecedent moisture. Watershed or drainage area time of concentration is the key component of the dimensionless "unit hydrograph," which is defined as a discharge hydrograph resulting from one inch of direct runoff distributed uniformly over the watershed resulting from a rainfall of a specified duration. A complete description of the NRCS methodology can be found in the *NRCS National Engineering Handbook,* Part 630 -Hydrology (*NEH*), available at:

https://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/?cid=stelprdb1043063.

Information Required for the NRCS Methodology

The index below and continued on the following page lists of all the information required in order to use the NRCS methodology of computing stormwater runoff. Examples are provided and begin on Page 30.

Information Required to use the NRCS Methodology (*cont'd.*)	Page No.
Runoff Hydrographs	24
Directly Connected Impervious Cover	27
Unconnected Impervious Cover	28
Reduced Curve Numbers	29

1. **Hydrologic Soil Group of the drainage area soil:** Under the NRCS classification, soils are classified into hydrologic soil groups (HSGs) to indicate the minimum rate of infiltration obtained for bare soil after prolonged wetting. The HSGs, which have the designations A, B, C and D, are arranged from highest to lowest in order of soil permeability, or infiltration rate, which is the rate at which water enters the soil at the soil surface. Infiltration is controlled by the surface condition. HSG also indicates the transmission rate—the rate which the water moves within the soil.

   The U.S. Department of Agriculture's (USDA) Soil Surveys by county or the soil survey data from USDA's Soil Survey website can be used in the preliminary or conceptual design. Currently, the information regarding the location of the HSGs present at a location, and the specific soil properties, is available online at:

   https://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm.

   However, during the design process, if soil boring samples and/or field tests of permeability show that the soil of the site has a different HSG soil than the information obtained from the USDA soil survey, the calculation of stormwater runoff and groundwater recharge must be adjusted to the HSG designation obtained from field soil testing. Soil Permeability Testing requirements and procedures can be found in *Chapter 12* of this manual.

2. **Sub-drainage areas:** Each sub-drainage area having different flow patterns and drainage points by which stormwater runoff leaves the sub-drainage area, must be individually identified, and the hydrological analysis of each sub-drainage area must be individually performed. When a site consists of impervious areas and pervious areas, the impervious areas and pervious areas must be separated into sub-different drainage areas in accordance with N.J.A.C. 7:8-5.7. Some hydrologic modeling software packages may allow the user to calculate the runoff separately from impervious surfaces and pervious surfaces that exist in one drainage area. **However, the design engineer may only use this modeling option if the impervious area time of concentration is the same as the pervious area time of concentration**.

3. **Land cover**: The types of vegetation present, the density of the vegetation, the types of development and the percentage of impervious cover are all characteristics that factor into the CN value. For the

pre-development condition, the presumed state is wooded land use in good hydrologic condition unless it is proven otherwise as set forth in the N.J.A.C. 7:8-5.6. Take note that the cover types for streets and roads, urban districts and residential districts by average lot size in Table 9-5, of *Chapter 9, NEH Part 630*, are intended for modeling large watershed on a watershed-wide scale. They are not intended for use in modeling runoff from individual development sites. For runoff from individual sites involving a directly connected or unconnected impervious surface, it may be necessary to compute runoff from the impervious surface separately from any pervious surfaces.

For a site that has more than one land cover existing on the site during the five years immediately prior to the time of application, the land cover with the lowest runoff potential must be used for the computations, as specified at N.J.A.C. 7:8-5.7(a)2. For example, if a site had an existing asphalt paved parking lot removed in 2012 and vegetation was established after the removal of the pavement, the application for stormwater management approval in 2015 cannot claim the removed asphalt parking lot as an impervious surface on the site since the surface with the lowest runoff potential is the vegetation that was established prior to the time of the application.

4. **Rainfall depth for the stormwater runoff quantity control design storms:** Rainfall depth is an essential parameter in the calculation of stormwater runoff volumes and peak flows when using the NRCS methodology. Two sources of data are available, as follows:

   a. Rainfall depth for a specific location from the New Jersey 24-hour Rainfall Frequency Data for a specific county, as provided in either Table 5-1 provided on the following page or by following this link:

      https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/nrcs141p2_018235.pdf.

Table 5-1: County-Specific, New Jersey 24-Hour Rainfall Frequency Data

NEW JERSEY 24 HOUR RAINFALL FREQUENCY DATA

Rainfall amounts in Inches

County	1 year	2 year	5 year	10 year	25 year	50 year	100 year
Atlantic	2.72	3.31	4.30	5.16	6.46	7.61	8.90
Bergen	2.75	3.34	4.27	5.07	6.28	7.32	8.47
Burlington	2.77	3.36	4.34	5.18	6.45	7.56	8.81
Camden	2.73	3.31	4.25	5.06	6.28	7.34	8.52
Cape May	2.67	3.25	4.22	5.07	6.34	7.47	8.73
Cumberland	2.69	3.27	4.25	5.09	6.37	7.49	8.76
Essex	2.85	3.44	4.40	5.22	6.44	7.49	8.66
Gloucester	2.71	3.29	4.24	5.05	6.29	7.36	8.55
Hudson	2.73	3.31	4.23	5.02	6.19	7.20	8.31
Hunterdon	2.80	3.38	4.26	5.00	6.09	7.02	8.03
Mercer	2.74	3.31	4.23	5.01	6.19	7.20	8.33
Middlesex	2.76	3.35	4.30	5.12	6.36	7.43	8.63
Monmouth	2.79	3.38	4.38	5.23	6.53	7.66	8.94
Morris	2.94	3.54	4.47	5.24	6.37	7.32	8.35
Ocean	2.81	3.42	4.45	5.33	6.68	7.87	9.20
Passaic	2.87	3.47	4.42	5.23	6.43	7.47	8.62
Salem	2.69	3.26	4.20	5.00	6.22	7.28	8.45
Somerset	2.76	3.34	4.25	5.01	6.15	7.13	8.21
Sussex	2.68	3.22	4.02	4.70	5.72	6.60	7.58
Union	2.80	3.39	4.35	5.17	6.42	7.49	8.69
Warren	2.78	3.34	4.18	4.89	5.93	6.83	7.82

Notes: The average point rainfall amounts listed above were developed from data contained in NOAA Atlas 14 Volume 2.

Point rainfall estimates for specific locations may be obtained from the Precipitation Frequency Data Server located at http://www.nws.noaa.gov/ohd/hdsc/

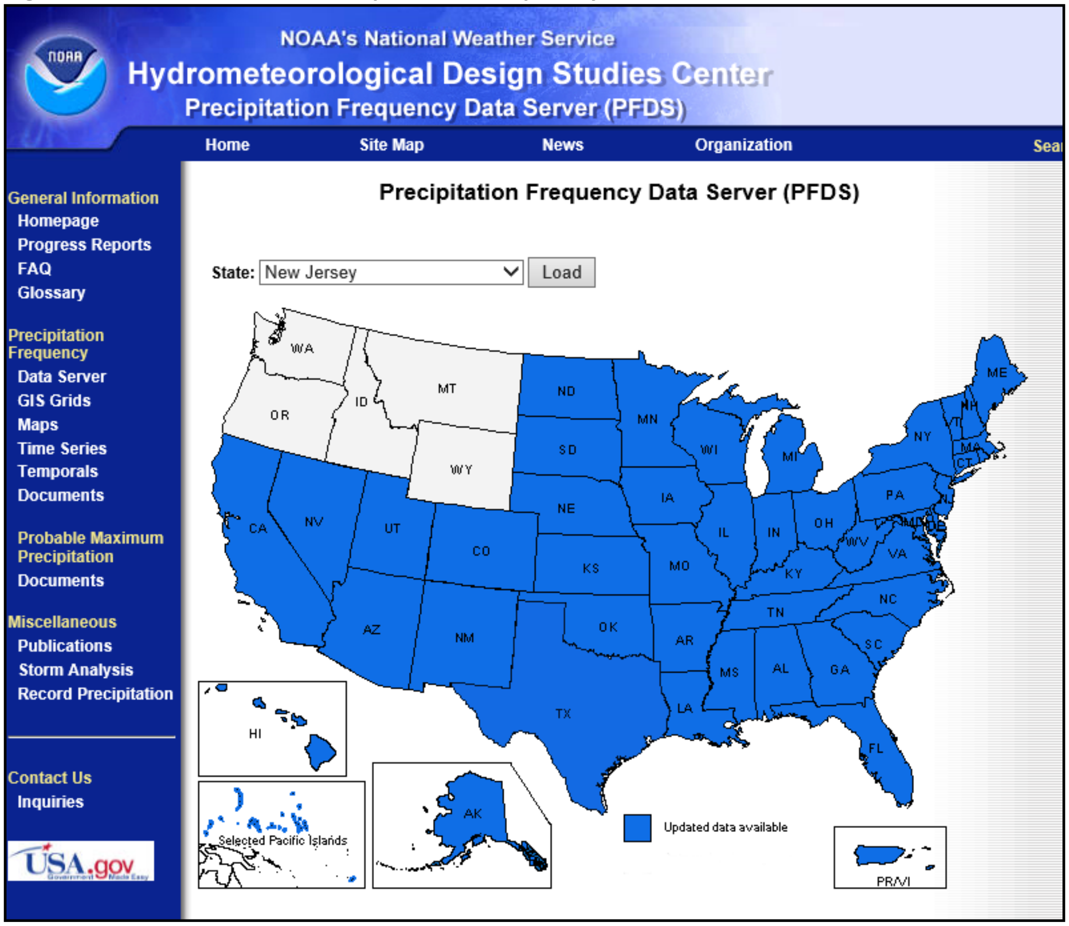
For most hydrologic design procedures, the rainfall amounts listed above may be rounded to the nearest tenth of an inch.

b. Rainfall data obtained from a nearby weather station, as provided by NOAA's NWS, which is available online at: https://hdsc.nws.noaa.gov/hdsc/pfds.

Below is an example of using the link in b above to obtain rainfall depth data for a location in Trenton, NJ.

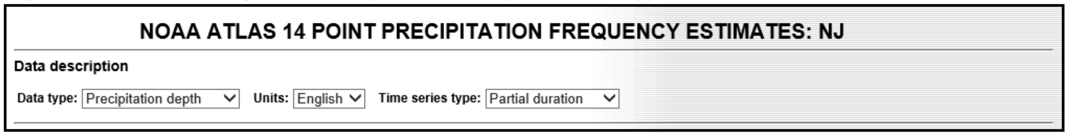
Step 1: Choose New Jersey from the drop-down list shown in the image below.

Figure 5-4: NOAA's NWS Precipitation Frequency Data Server Website



Step 2: In the Data description section of the next window that opens, from the Select Data Type dropdown menu, choose "Precipitation depth" rather than "Precipitation intensity," the latter of which is used more often for the Rational Method and is discussed beginning on Page 70. Then, for the Time series type, select "Partial duration" from that dropdown menu, as shown in Figure 5-5.

Figure 5-5: Selecting the Precipitation Depth Data Type

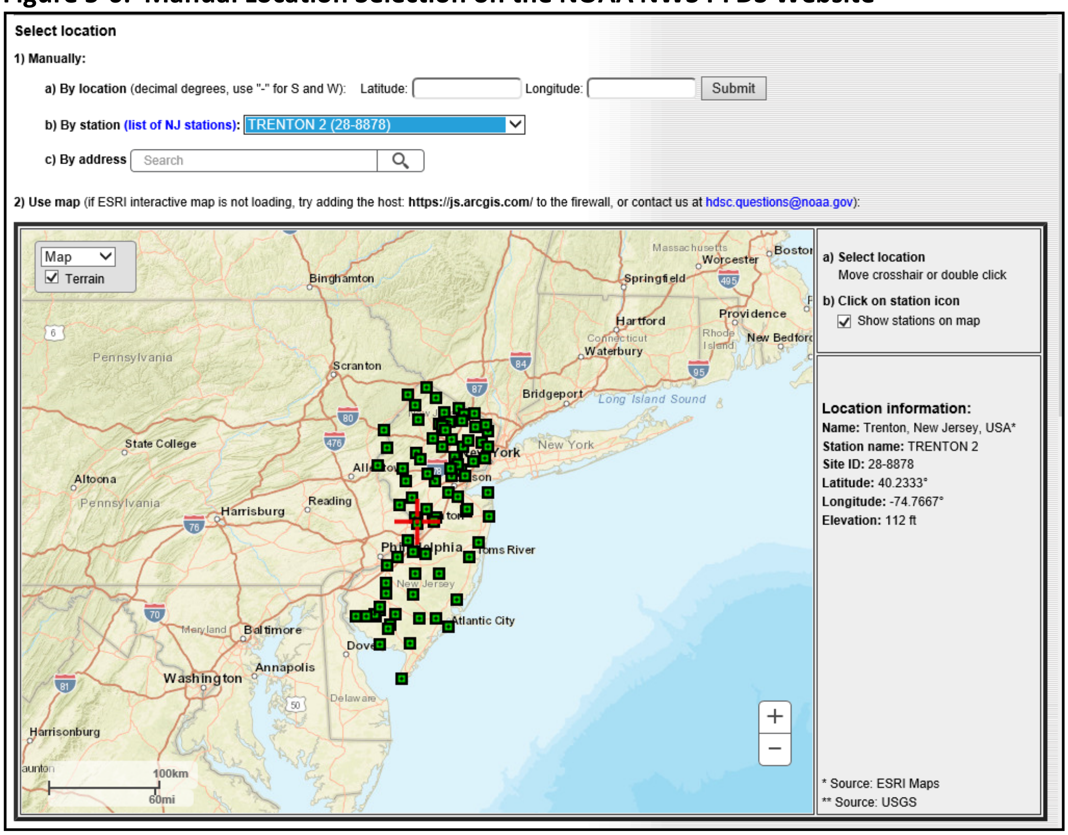


Step 3: In the Select location section, input the location information by one of four methods:

- latitude/longitude,
- station name,
- address or
- left click on the location on the interactive map.

For this example, Trenton Station 2 was selected from the dropdown menu under 1.b):

Figure 5-6: Manual Location Selection on the NOAA NWS PFDS Website



Step 4: Scroll down the page to the Point Precipitation Frequency (PF) Estimates section. Left click on the PF tabular option, if it does not appear on top of the other tabs, which will be highlighted in dark blue, as shown in the following image:

Figure 5-7: Point Precipitation Frequency (PF) Estimates – Tabular Option

POINT PRECIPITATION FREQUENCY (PF) ESTIMATES

WITH 90% CONFIDENCE INTERVALS AND SUPPLEMENTARY INFORMATION
NOAA Atlas 14, Volume 2, Version 3

PF tabular | PF graphical | Supplementary information | Print page

PDS-based precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	**0.342** (0.311-0.377)	**0.408** (0.371-0.450)	**0.486** (0.440-0.535)	**0.542** (0.490-0.596)	**0.611** (0.549-0.672)	**0.662** (0.592-0.729)	**0.712** (0.634-0.786)	**0.759** (0.672-0.841)	**0.819** (0.717-0.912)	**0.865** (0.751-0.969)
10-min	**0.547** (0.497-0.602)	**0.653** (0.594-0.720)	**0.778** (0.704-0.856)	**0.866** (0.783-0.953)	**0.974** (0.875-1.07)	**1.05** (0.943-1.16)	**1.13** (1.01-1.25)	**1.20** (1.07-1.33)	**1.30** (1.14-1.44)	**1.36** (1.18-1.53)
15-min	**0.683** (0.621-0.753)	**0.821** (0.747-0.905)	**0.984** (0.890-1.08)	**1.10** (0.991-1.21)	**1.24** (1.11-1.36)	**1.34** (1.19-1.47)	**1.43** (1.27-1.58)	**1.52** (1.35-1.68)	**1.63** (1.43-1.82)	**1.71** (1.48-1.92)
30-min	**0.937** (0.852-1.03)	**1.13** (1.03-1.25)	**1.40** (1.26-1.54)	**1.59** (1.44-1.75)	**1.83** (1.64-2.01)	**2.01** (1.80-2.21)	**2.19** (1.95-2.42)	**2.36** (2.09-2.62)	**2.59** (2.27-2.89)	**2.77** (2.40-3.10)
60-min	**1.17** (1.06-1.29)	**1.42** (1.29-1.57)	**1.79** (1.62-1.97)	**2.07** (1.87-2.28)	**2.44** (2.19-2.68)	**2.72** (2.44-3.00)	**3.02** (2.69-3.33)	**3.32** (2.94-3.67)	**3.72** (3.26-4.14)	**4.04** (3.51-4.53)
2-hr	**1.41** (1.28-1.56)	**1.72** (1.56-1.90)	**2.18** (1.97-2.40)	**2.53** (2.28-2.78)	**3.01** (2.70-3.31)	**3.40** (3.03-3.74)	**3.80** (3.37-4.19)	**4.21** (3.71-4.66)	**4.78** (4.16-5.33)	**5.24** (4.51-5.87)
3-hr	**1.55** (1.40-1.72)	**1.89** (1.71-2.10)	**2.39** (2.16-2.66)	**2.79** (2.51-3.09)	**3.34** (2.98-3.70)	**3.78** (3.36-4.20)	**4.25** (3.75-4.73)	**4.74** (4.14-5.28)	**5.42** (4.67-6.07)	**5.97** (5.08-6.73)
6-hr	**1.96** (1.77-2.19)	**2.38** (2.15-2.65)	**3.01** (2.71-3.35)	**3.52** (3.15-3.91)	**4.26** (3.78-4.73)	**4.87** (4.30-5.41)	**5.54** (4.83-6.16)	**6.25** (5.39-6.96)	**7.28** (6.17-8.16)	**8.14** (6.80-9.19)
12-hr	**2.38** (2.15-2.66)	**2.88** (2.60-3.23)	**3.66** (3.30-4.10)	**4.33** (3.88-4.83)	**5.32** (4.72-5.92)	**6.18** (5.43-6.89)	**7.12** (6.18-7.94)	**8.17** (6.98-9.15)	**9.74** (8.14-11.0)	**11.1** (9.10-12.6)
24-hr	**2.76** (2.55-3.00)	**3.33** (3.08-3.62)	**4.23** (3.90-4.59)	**4.99** (4.59-5.41)	**6.12** (5.58-6.62)	**7.09** (6.42-7.66)	**8.15** (7.32-8.82)	**9.32** (8.28-10.1)	**11.1** (9.67-12.0)	**12.5** (10.8-13.7)
2-day	**3.19** (2.94-3.48)	**3.86** (3.56-4.21)	**4.91** (4.51-5.36)	**5.78** (5.29-6.30)	**7.04** (6.40-7.66)	**8.11** (7.32-8.82)	**9.27** (8.31-10.1)	**10.5** (9.35-11.5)	**12.4** (10.8-13.5)	**13.9** (12.1-15.3)
3-day	**3.38** (3.12-3.68)	**4.08** (3.77-4.45)	**5.16** (4.76-5.62)	**6.05** (5.56-6.58)	**7.33** (6.70-7.95)	**8.41** (7.63-9.11)	**9.56** (8.62-10.4)	**10.8** (9.67-11.7)	**12.6** (11.2-13.7)	**14.1** (12.4-15.4)
4-day	**3.57** (3.31-3.87)	**4.31** (3.99-4.68)	**5.42** (5.01-5.88)	**6.32** (5.83-6.85)	**7.63** (6.99-8.24)	**8.70** (7.94-9.40)	**9.85** (8.94-10.6)	**11.1** (9.98-12.0)	**12.9** (11.5-13.9)	**14.3** (12.7-15.5)
7-day	**4.18** (3.87-4.55)	**5.02** (4.64-5.46)	**6.22** (5.75-6.77)	**7.22** (6.65-7.84)	**8.65** (7.93-9.38)	**9.84** (8.98-10.7)	**11.1** (10.1-12.0)	**12.5** (11.2-13.5)	**14.4** (12.9-15.6)	**16.0** (14.2-17.4)
10-day	**4.76** (4.43-5.14)	**5.69** (5.30-6.15)	**6.96** (6.47-7.51)	**7.99** (7.41-8.61)	**9.43** (8.71-10.2)	**10.6** (9.76-11.4)	**11.8** (10.8-12.7)	**13.1** (11.9-14.1)	**14.9** (13.4-16.1)	**16.3** (14.7-17.7)
20-day	**6.43** (6.04-6.84)	**7.63** (7.17-8.12)	**9.12** (8.57-9.71)	**10.3** (9.66-10.9)	**11.9** (11.1-12.6)	**13.1** (12.3-14.0)	**14.4** (13.4-15.3)	**15.7** (14.5-16.7)	**17.4** (16.0-18.6)	**18.7** (17.1-20.0)
30-day	**8.01** (7.57-8.45)	**9.44** (8.93-9.97)	**11.0** (10.4-11.7)	**12.3** (11.6-13.0)	**13.9** (13.1-14.7)	**15.2** (14.3-16.0)	**16.4** (15.4-17.3)	**17.6** (16.5-18.6)	**19.2** (17.8-20.3)	**20.4** (18.9-21.6)
45-day	**10.2** (9.70-10.7)	**12.0** (11.4-12.6)	**13.8** (13.1-14.5)	**15.2** (14.4-16.0)	**17.0** (16.1-17.8)	**18.3** (17.3-19.2)	**19.5** (18.4-20.6)	**20.7** (19.5-21.8)	**22.2** (20.8-23.4)	**23.3** (21.8-24.6)
60-day	**12.2** (11.7-12.8)	**14.3** (13.7-15.0)	**16.4** (15.6-17.2)	**17.9** (17.1-18.8)	**19.8** (18.9-20.8)	**21.2** (20.2-22.2)	**22.5** (21.4-23.6)	**23.7** (22.5-24.9)	**25.2** (23.8-26.5)	**26.2** (24.7-27.7)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

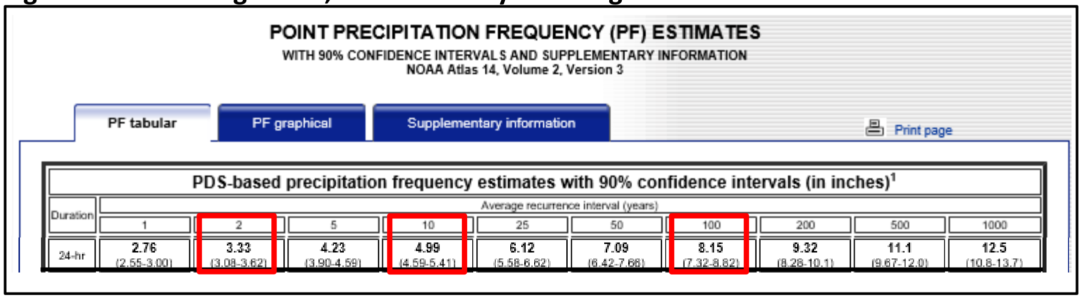
Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

Estimates from the table in CSV format: Precipitation frequency estimates | Submit

The data needed is found in the row labeled "24-hr." The values in the columns labeled "2," "10" and "100" correspond to the rainfall depths generated by the 2-, 10- and 100-year design storms, respectively, for this weather station location, as outlined in red in Figure 5-8 on the following page.

Figure 5-8: Locating the 2-, 10- and 100- year Design Storm Rainfall Data



POINT PRECIPITATION FREQUENCY (PF) ESTIMATES
WITH 90% CONFIDENCE INTERVALS AND SUPPLEMENTARY INFORMATION
NOAA Atlas 14, Volume 2, Version 3

PF tabular | PF graphical | Supplementary information | Print page

PDS-based precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
24-hr	2.76 (2.55-3.00)	3.33 (3.08-3.62)	4.23 (3.90-4.59)	4.99 (4.59-5.41)	6.12 (5.58-6.62)	7.09 (6.42-7.68)	8.15 (7.32-8.82)	9.32 (8.28-10.1)	11.1 (9.67-12.0)	12.5 (10.8-13.7)

5. **Rainfall distribution for the stormwater runoff quantity control design storms:** In addition to the rainfall depth, knowing how rain falls during a storm event is important in calculating the peak flow rate of the stormwater runoff generated. Keep in mind that, generally, a precipitation event typically begins with a lighter intensity of rain falling, followed by a period during which rain falls at a higher intensity before gently tapering off. To achieve the goal of estimating rainfall events for design and planning purposes, between 1961 and 1977, NRCS developed synthetic rainfall distributions from historical records from the different regions of the country. These rainfall distributions were based upon the assumption that the rain distribution is bell-shaped, meaning it has less rainfall in the beginning and at the end of the rain event. The NRCS rainfall distributions were grouped into four types according to the applicable regions or geographic situations. Types I and IA represented the Pacific maritime climate with wet winters and dry summers. Type III represented the Gulf of Mexico and Atlantic coastal areas, including New Jersey, where tropical storms produced large 24-hour rainfall events. Type II represented the rest of the country. These NRCS rainfall distributions had durations of 24-, 18-, 12- or 6-hours.

   On September 10, 2012, NCRS issued a note, NEW JERSEY BULLETIN NO. NJ210-12-1, stating that:

   > Based on updated rainfall data from NOAA, NRCS has developed new storm distributions for use with EFH-2 and WinTR-55. New Jersey has two new rainfall distribution regions: Region C covering the counties of Sussex, Warren, Hunterdon, Somerset, Mercer, Burlington, Camden, Gloucester, Atlantic, Salem, Cumberland, and Cape May; and Region D covering Bergen, Hudson, Essex, Passaic, Morris, Union, Middlesex, Monmouth and Ocean. The new rainfall distributions replace use of the TYPE III distribution in New Jersey. The 24-hour rainfall-frequency data has been updated as well with only minor variations for some of the counties.

   As also stated in Bulletin No. NJ210-12-1, when designing BMPs to meet the stormwater runoff quantity control standards, NOAA_C and NOAA_D rainfall distributions must be applied to Region C and Region D, respectively.

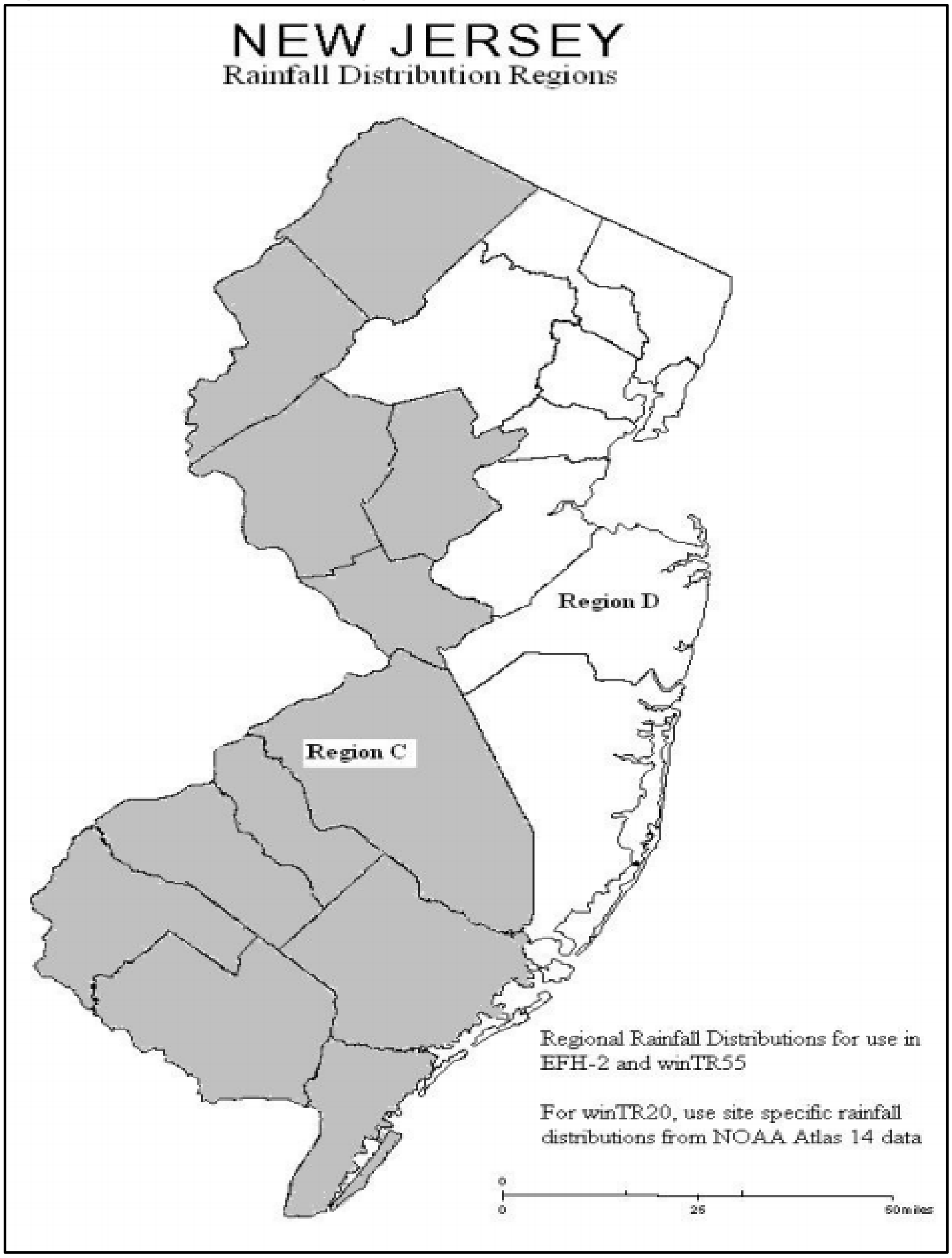
   The location of Regions C and D are shown on the following page in Figure 5-9. NOAA_C and NOAA_D rainfall distributions, in text format, are available online at:

   https://www.nrcs.usda.gov/wps/portal/nrcs/main/nj/technical/engineering/.

   NOAA_C and NOAA_D rainfall precipitation distributions and rainfall intensity are also available in Excel format from the Department's website, under the heading for *Chapter 5*, via the following link:

https://www.njstormwater.org/bmp_manual2.htm

Figure 5-9: NJ Locations of Regions C and D



6. **Rainfall Depth for the Stormwater Runoff Water Quality Design Storm**: For stormwater runoff quality control, N.J.A.C. 7:8-5.5 requires using 1.25 inches of rain falling nonuniformly in a 2-hour storm event, which is also known as the Water Quality Design Storm (WQDS).

7. **Rainfall Distribution for the NJDEP Water Quality Design Storm:** During its duration, precipitation falls in a nonlinear pattern as depicted in N.J.A.C. 7:8-5.5(a) and in Table 5-2 on the following page. This rainfall pattern or distribution is based on Trenton, New Jersey, rainfall data collected between 1913 and 1975 and contains intermediate rainfall intensities that have the same probability or recurrence interval as the storm's total rainfall and duration. As such, for times of concentration up to two hours, the NJDEP WQDS can be used to compute runoff volumes, peak rates and hydrographs of equal probability. This ensures that all stormwater runoff water quality BMPs, whether they are based on total runoff volume or peak runoff rate, will provide the same level of stormwater pollution control. An Excel file providing the rainfall distribution and rainfall intensity of the WQDS, in 1 minute intervals, is also available on the Department's website, under the heading for *Chapter 5*, via the following link:

   https://www.njstormwater.org/bmp_manual2.htm

Table 5-2: NJDEP 1.25-Inch/2-Hour Stormwater Runoff Water Quality Design Storm Rainfall Distribution

Time (Minutes)	Cumulative Rainfall (Inches)	Time (Minutes)	Cumulative Rainfall (Inches)	Time (Minutes)	Cumulative Rainfall (Inches)
1	0.00166	41	0.1728	81	1.0906
2	0.00332	42	0.1796	82	1.0972
3	0.00498	43	0.1864	83	1.1038
4	0.00664	44	0.1932	84	1.1104
5	0.00830	45	0.2000	85	1.1170
6	0.00996	46	0.2117	86	1.1236
7	0.01162	47	0.2233	87	1.1302
8	0.01328	48	0.2350	88	1.1368
9	0.01494	49	0.2466	89	1.1434
10	0.01660	50	0.2583	90	1.1500
11	0.01828	51	0.2783	91	1.1550
12	0.01996	52	0.2983	92	1.1600
13	0.02164	53	0.3183	93	1.1650
14	0.02332	54	0.3383	94	1.1700
15	0.02500	55	0.3583	95	1.1750
16	0.03000	56	0.4116	96	1.1800
17	0.03500	57	0.4650	97	1.1850
18	0.04000	58	0.5183	98	1.1900
19	0.04500	59	0.5717	99	1.1950
20	0.05000	60	0.6250	100	1.2000
21	0.05500	61	0.6783	101	1.2050
22	0.06000	62	0.7317	102	1.2100
23	0.06500	63	0.7850	103	1.2150
24	0.07000	64	0.8384	104	1.2200
25	0.07500	65	0.8917	105	1.2250
26	0.08000	66	0.9117	106	1.2267
27	0.08500	67	0.9317	107	1.2284
28	0.09000	68	0.9517	108	1.2300
29	0.09500	69	0.9717	109	1.2317
30	0.10000	70	0.9917	110	1.2334
31	0.10660	71	1.0034	111	1.2351
32	0.11320	72	1.0150	112	1.2367
33	0.11980	73	1.0267	113	1.2384
34	0.12640	74	1.0383	114	1.2400
35	0.13300	75	1.0500	115	1.2417
36	0.13960	76	1.0568	116	1.2434
37	0.14620	77	1.0636	117	1.2450
38	0.15280	78	1.0704	118	1.2467
39	0.15940	79	1.0772	119	1.2483
40	0.16600	80	1.0840	120	1.2500

The accumulative distribution curve for rainfall depth, shown below in Figure 5-10, is a graphical representation of 1.25 inches of rainfall falling in the 2-hour NJDEP WQDS.

Figure 5-10: Stormwater Runoff Water Quality Design Storm Rainfall Cumulative Distribution Curve

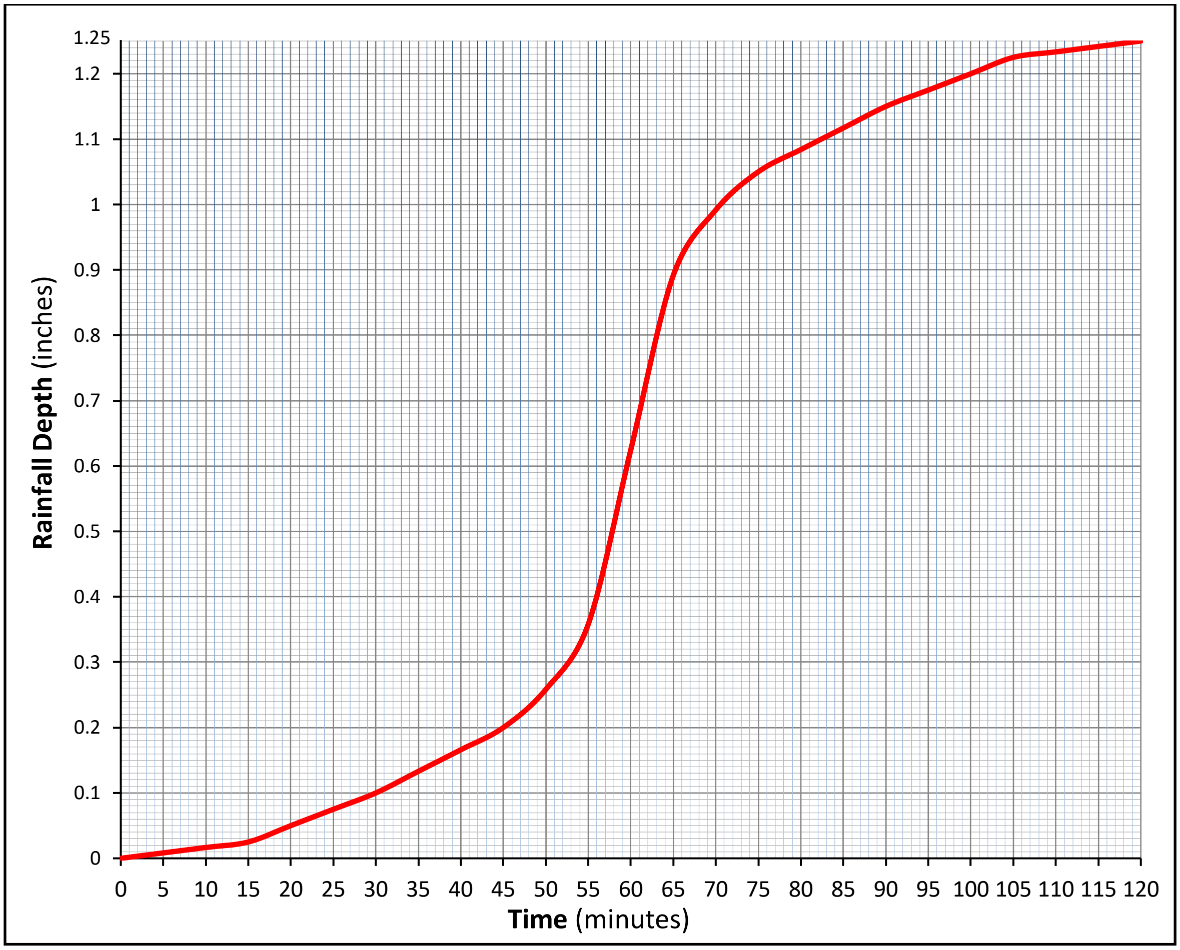
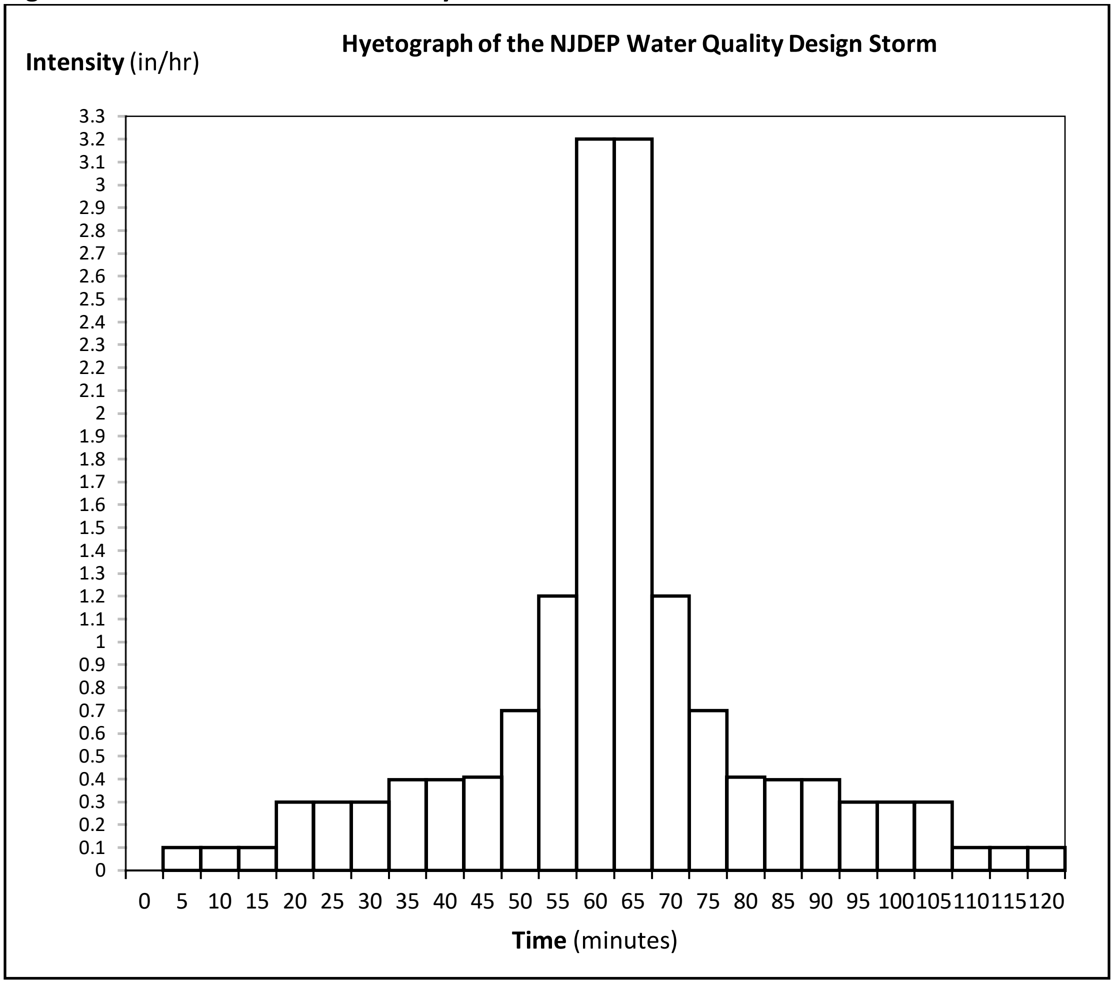


Figure 5-11, shown on the following page, is the intensity of the rainfall distribution derived from Table 5-2.

Figure 5-11: WQDS Rainfall Intensity Distribution



8. **The time of travel and the time of concentration:** One of the methods identified in the NRCS methodology for calculating time of concentration (T_c) is the velocity method, which assumes the time of concentration is "*the sum of travel times for segments along the hydraulically most distant flow path,*" as stated in Chapter 15, in Part 630 of the *NEH*. Flow in a segment may occur as sheet, shallow concentrated or open channel flow, which describe the nature of the flow. Sheet flow is lowest in energy of the three and typically occurs at depths less than or equal to 0.1 ft, before the flow transitions to shallow concentrated flow.

   In performing T_c calculations, designers must apply the following:

   - **Maximum sheet flow roughness coefficient:** According to the NRCS, the maximum Manning's Roughness Coefficient (n) to be used in Equation 15-8, which is for sheet flow, is 0.80 for woods with dense underbrush; however, **in New Jersey, the maximum Manning's coefficient for sheet flow that may be used is 0.40**. For impervious pavement such as a driveway, street, concrete sidewalk, cement finished walkway, stone, paver blocks, porous paving or rooftop, $n = 0.011$.

Table 15-1 in *NEH, Part 630*, *Chapter 15* lists additional values for Manning's roughness coefficient for sheet flow.

- **Maximum sheet flow length:**

  - ☐ **For the pre-construction condition,** the maximum distance which can be used as the length of sheet flow in the time of concentration calculation is 100 ft , unless there is something physically in contact with the flow of stormwater runoff, such as a swale, curb or inlet, to prevent sheet flow from occurring, i.e., by increasing the depth of flow in excess of 0.1 ft, regardless of whether the surface is impervious or pervious.
  - ☐ **For the post-construction condition**, the maximum distance for which flow occurs as sheet flow is 100 ft, and the distance over which sheet flow occurs, L, must be calculated using the McCuen-Spiess limitation, as follows:

$$\mathrm{L} = \frac{100\sqrt{S}}{n}$$

where S is the slope, in ft/ft, and n is the Manning's roughness coefficient for sheet flow**. If the sheet flow length calculated by the McCuen-Spiess limitation criteria exceeds 100 ft, the sheet flow length must be limited to 100 ft.** For an undisturbed area, the sheet flow length will remain same as in the pre-construction condition.

- **Calculating the travel time for a segment in which sheet flow occurs:** According to the *NEH*, a simplified form of Manning's kinematic solution, Equation 15-8, is used to compute travel time for sheet flow, as follows:

$$\mathrm{T}_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5} S^{0.4}}$$

- **Calculating the travel time for a segment in which shallow concentrated flow occurs:** Shallow concentrated flow occurs after sheet flow and the depths range from 0.1 to 0.5 ft. For this type of flow, the average velocity of the flow in the segment must be derived from Figure 15-4 in NEH and then input into Equation 15-1 to calculate the travel time:

$$\mathrm{T}_t = \frac{Shallow\ Concentrated\ Flow\ Length}{V\ x\ 3600}$$

where T_t is the travel time (hr) and V is the average flow velocity (ft/s). These steps are presented in Example 5-1, which begins on Page 30.

- **Calculating the travel time for a segment in which open channel flow occurs:** Open channel flow is assumed to occur after shallow concentrated flow and where "*either surveyed cross-sectional information has been obtained, where channels are visible on aerial photographs or where blueline (indicated streams) occur on U.S. Geological Survey (USGS) quadrangle sheets*," per the *Chapter 15, Part 630* of the *NEH*, which also includes Equation 15-10, which is to be used for open channel flow, along with information regarding its application and limitations.

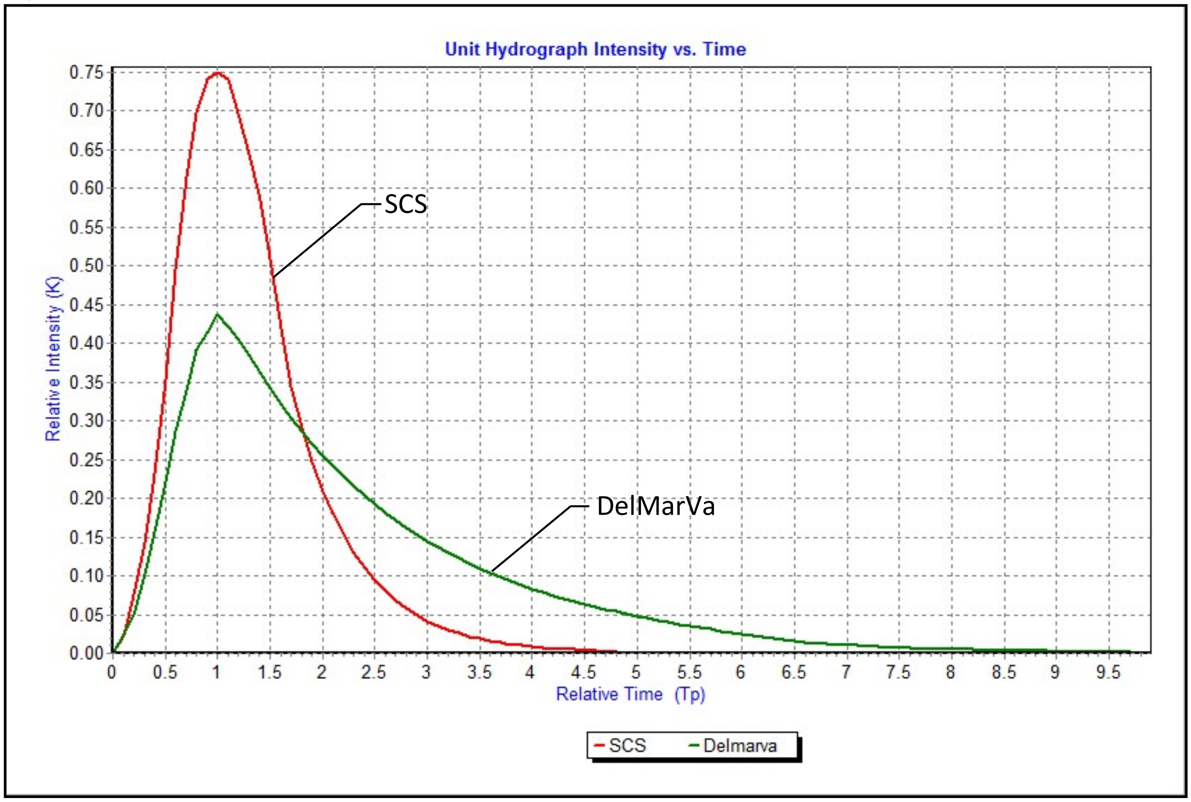
- **T_c routes:** Consideration must be given to the hydraulic conditions that exist along a selected T_c route, particularly in pre-developed drainage areas. T_c routes should not cross through significant flow constrictions and ponding areas without considering the peak flow and time attenuation effects of such areas, meaning the flow must be routed as a pond. As noted in the NJDEP Stormwater Management rules, such areas can occur at hedgerows, undersized culverts, fill areas, sinkholes and isolated ponding areas. In general, a separate subarea tributary to such areas should be created and its runoff routed through the area before combining with downstream runoff.

  There is no longer a minimum or default value that may be used for the time of concentration. T_c for pre- and post-construction conditions must be calculated based on the aforementioned requirements.

9. **Runoff Hydrographs:** The NRCS method uses a Unit Hydrograph for runoff incorporated with the NRCS rainfall distributions (NOAA_C and NOAA_D for New Jersey) to develop a Dimensionless Unit Hydrograph. Runoff is transformed into a hydrograph by using unit hydrograph theory and routing procedures that depend on runoff travel time through segments of the watershed. In development of the runoff hydrograph, the runoff discharge is nonlinear in relation to the time of the rain event in accordance with NRCS observations from many natural unit hydrographs developed from watersheds varying widely in size and geographical locations. A dimensionless unit hydrograph was developed which has a peak rate factor of 484, which means that 48.4% of the total runoff volume is discharged before the peak time and 51.6% of the total runoff volume is discharged after the peak time. The dimensionless unit hydrograph having a 484 peak rate factor is normally called the "SCS Standard Dimensionless Unit Hydrograph (DUH)."

   NRCS also developed an alternative DUH for the DelMarVa region (which corresponds to the **De**laware, **Mar**yland and **V**irgini**a** peninsula), where coastal, flat areas that have an average watershed slope less than 5 percent, with low topographic relief and significant surface storage in swales and depressions are found. NRCS call it the "DelMarVa DUH," which as a peak rate factor of 284. Under the DelMarVa DUH, the amount of runoff volume discharged before the peak time is smaller, i.e., 28.45% of the total volume ; additionally, the length of time under the runoff curve is prolonged. Therefore, by using the DelMarVa DUH, the peak flow rate of runoff will be smaller and the entire runoff routing time will be longer. The graph in Figure 5-12, found on the following page, illustrates the differences between the 484 DUH and the DelMarVa DUH.

Figure 5-12: NRCS Standard DUH (484 DUH) versus the DelMarVa DUH



The DelMarVa DUH must be used in calculating pre-construction peak flowrates for the 2-, 10- and 100-year storms in the Coastal Plain Region of New Jersey, unless the design engineer proves, to the satisfaction of the review engineer, that the conditions for applicability are not present anywhere in the watershed. The physiographic provinces of New Jersey are depicted in Figure 5-13, which may be found on the next page, or are available online from NJDEP's Bureau of Geologic Information Systems at:

https://www.nj.gov/dep/gis/digidownload/metadata/html/Geol_province.html.

Also note that the same type of DUH must be used in the pre- and post-development hydrographs. Projects which lie on or near the boundary between the Standard and Delmarva regions identified by NRCS should be modeled with the DelMarVa Unit Hydrograph, except as noted above.

Take note that the DelMarVa DUH cannot be used in sizing Manufactured Treatment Devices, even if the site is located in the geographical area where the NRCS recommends the application of the DelMarVa DUH.

Figure 5-13: Physiographic Provinces of NJ

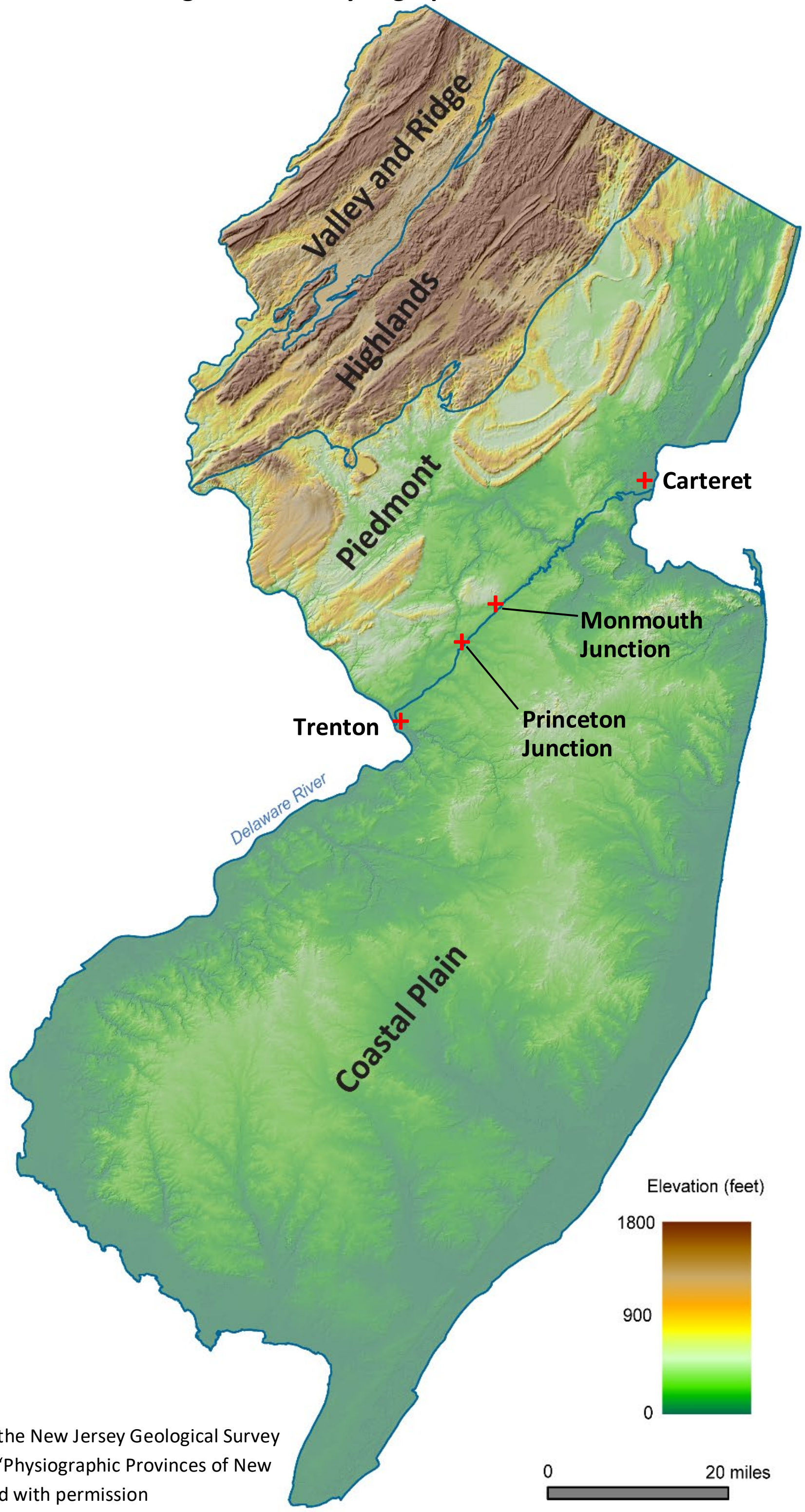


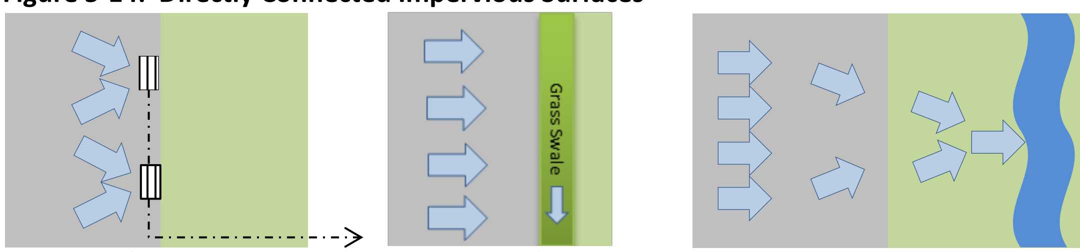
Image modified from the New Jersey Geological Survey Information Circular, "Physiographic Provinces of New Jersey, 2006" and used with permission

10. **Directly Connected Impervious Cover:** Impervious surfaces are considered directly connected if the impervious surface meets one of the conditions listed below:

    a. Runoff from the impervious surface flows directly into the drainage system, water bodies and riparian zones or wetlands.

    b. Runoff is shallow concentrated flow that runs over a pervious area and then into the drainage system, water bodies and riparian zones or wetlands.

Figure 5-14: Directly Connected Impervious Surfaces



Connected to Stormwater Collection System | Connected to Down-gradient BMP | Concentrated Flow Directed to Down-gradient Stream

Shown above are examples of directly connected impervious surfaces, which include, but are not limited to, runoff from an impervious surface

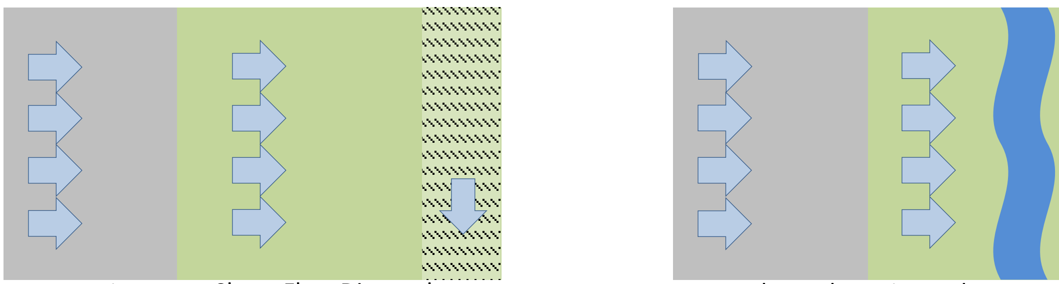
- collected by a storm drain, which then connects to a conduit or channel to a downstream BMP, stormwater collection system or stream or
- flowing over a pervious surface by shallow concentrated flow or channelized flow and then into a channel to a down-gradient stream or other flowing water body.

The Stormwater Management rules at N.J.A.C. 7:8-5.7 requires that *the design engineer shall consider the relative stormwater runoff rates and/or volumes of pervious and impervious surfaces separately to accurately compute the rates and volume of stormwater runoff from the site in computing stormwater runoff from all design storms. Therefore, when the site has directly connected impervious surface, the runoff volume and peak flow rate from impervious surface and pervious surface shall be modelled individually*.

If the runoff from an impervious surface and from a pervious surface will converge into one point of analysis, such as stormwater BMP or stormwater conveyance system, the runoff volumes from impervious surface and pervious surface, each calculated separately, can be added together to obtain the total runoff volume. For peak flow modeling, since the time of the peak flow for runoff from impervious surface may not be at the same time as that from the pervious surface within a sub-drainage area, the two peak flow rates must not be simply added together. Instead, a composite hydrograph must be created by adding the separate runoff hydrographs from the impervious surface and the pervious surface, from which the overall peak flow rate can be determined.

11. **Unconnected Impervious Cover:** As described in detail in *Chapter 2: Low Impact Development Techniques*, an important nonstructural BMP is new impervious cover that is not directly connected to a site's drainage system. Instead, runoff from these impervious areas must undergo sheet flow onto adjacent pervious areas, where a portion of the impervious area runoff is given an opportunity to infiltrate into the soil. Under certain conditions described on the following page, this can help provide both groundwater recharge and stormwater quality treatment for small rainfall events as well as reduce the overall runoff volume that must be treated and/or controlled in a down-gradient BMP.

Figure 5-15: Unconnected Impervious Surfaces



An impervious area can be considered to be an unconnected impervious surface only when meeting all of the following conditions:

a. Upon entering the down-gradient pervious area, all runoff must remain as sheet flow.

b. Flow from the impervious surface must enter the down-gradient pervious area as sheet flow or, in the case of roofs, from one or more downspouts, each equipped with a splash pad, level spreader or dispersion trench that reduces flow velocity and induces sheet flow in the down-gradient pervious area.

c. All discharges onto the down-gradient pervious surfaces must be stable and non-erosive.

d. The shape, slope and vegetated cover in the down-gradient pervious area must be sufficient to maintain sheet flow throughout its length.

e. The maximum slope of the down-gradient pervious area is 8 percent.

Computation of the resultant runoff from unconnected impervious areas can be performed using two different methods: the NRCS composite CN with unconnected impervious area method published in *NEH, Part 630, Chapter 9*, or the Two-Step Method. Both methods require the following conditions to be met:

a. Only the portions of the impervious surface and the down-gradient pervious surface on which sheet flow occurs can be considered as an unconnected surface in the calculation. The area beyond the maximum sheet flow path length cannot be considered in the calculation.

b. The maximum sheet flow path length across the unconnected impervious surface is 100 ft.

c. The minimum sheet flow length across the down-gradient pervious surface is 25 ft in order to maintain the required sheet flow state of the runoff.

d. **The NRCS composite CN with unconnected impervious area method published in *NEH, Part 630, Chapter 9*, can be used only when the total impervious surface is less than 30 percent of the receiving down-gradient pervious surface because absorptive capacity of the pervious surface will not be sufficient to affect the overall runoff significantly.**

Example 5-2 uses the unconnected impervious area method in *NEH, Part 630, Chapter 9*. See Page 34.

12. **Reduced Curve Number:** The runoff volume retained or infiltrated by a stormwater BMP may provide a reduction of the runoff flow rate of the runoff passing through the stormwater BMP. For example, runoff managed with a green roof or a pervious paving system may have a portion of the runoff retained in the filtration medium of the green roof or the pervious paving system. The runoff flow rate discharged from the green roof or the pervious paving system will be reduced due to the retained runoff volume. The reduced runoff flow rate will be equivalent to the runoff flow rate calculated by a smaller curve number. Therefore, a reduced curve number method may be used to calculate the peak flow rate of 2-, 10- and 100-year design storms from a stormwater BMP. The reduced curve number method is illustrated in Example 1 of Chapter 9.6: Pervious Paving Systems and the example in Chapter 9.4: Green Roofs of the BMP Manual.

NRCS Methodology Examples

The examples listed in the table on the following page illustrate how to use the NRCS Methodology to calculate the time of concentration and the stormwater runoff volume generated by an unconnected impervious surface using the CN Method and the NJDEP Two-Step Method for calculating the stormwater runoff volume generated by an unconnected impervious surface flowing onto a pervious surface. **The method used in Example 5-4 must not be used** and is provided to illustrate why composite hydrographs are not permitted. Example 5-5 compares the pre- and post-condition hydrographs produced by a project in which impervious cover is reduced. Take note Examples 5-6 and 7, which begin on Page 44, illustrate designing a site with two points of discharge and then comparing the results to a similar site with a single converged discharge. These examples include both exfiltration in the routing calculations as a means of discharge and the use of the *Hantush Spreadsheet* to demonstrate the redesign process when groundwater mounding negatively impacts a BMP. Details on using the *Hantush Spreadsheet*, along with additional examples and a discussion of the acceptable range for input parameters, are found in *Chapter 13: Groundwater Table Hydraulic Impact Assessments for Infiltration BMPs*.

Example No.	Scenario Description	Page No.
5-1	Calculate of Time of Concentration	30
5-2	Use the NRCS CN Method for an Unconnected Impervious Surface to Calculate the Runoff Volume for a Site	34
5-3	Use the NJDEP Two-Step Method for an Unconnected Impervious Surface to Calculate the Runoff Volume for a Site	36
5-4	Demonstration of Why a Composite CN Generates an Incorrect Runoff Volume	38
5-5	A Comparison of Pre- and Post-condition Hydrographs for Compliance Under N.J.A.C. 7:8-5.6(b)2 When Impervious Cover is Reduced	40
5-6	A Re-development Project with Two Drainage Areas, Each Discharging to Separate Points,	44
5-7	The Same Re-development Project with Two Drainage Areas, having One Combined Discharge Point	66

Example 5-1: Calculate Time of Concentration

For the post-construction condition, stormwater runoff flows through a wooded drainage area along a flow path, measuring 1,000 ft in length, consisting of sheet flow over an area with a 0.5% slope and shallow concentrated flow over an area of 1% slope. Calculate the time of concentration for the post-construction condition.

Step 1: In this example, there are only 2 different segments of flow. Travel time under sheet flow is calculated as follows:

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5} s^{0.4}}$$

where:

T_t = travel time, hr
n = Manning's roughness coefficient for sheet flow
L = sheet flow length, ft
P_2 = 2-year, 24-hour rainfall, in
s = slope of land surface, ft/ft

The sheet flow length is calculated by using the formula from the McCuen-Spiess limitation criterion:

$$L = \frac{100\sqrt{S}}{n}$$

The values for the Manning's roughness coefficient can also be found in Table 15-1 in Chapter 15 of *NEH, Part 630*, which is shown to the right. Values for Manning's roughness coefficient must be selected in accordance with the land surface condition. The maximum value that can be used for woods, in New Jersey, is 0.40.

Table 15–1 Manning's roughness coefficients for sheet flow (flow depth generally ≤ 0.1 ft)

Surface description	n [1]
Smooth surface (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤ 20%	0.06
Residue cover > 20%	0.17
Grass:	
Short-grass prairie	0.15
Dense grasses [2]	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods: [3]	
Light underbrush	0.40
Dense underbrush	0.80

1 The Manning's n values are a composite of information compiled by Engman (1986).
2 Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.
3 When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

The 2-year 24-hour rainfall depth, outlined in red in the table to the right, is obtained from the NOAA Precipitation Frequency Server website, as shown on Page 16, in "Step 4" of the example that begins on Page 14.

PDS-based precipitation frequency

Duration	1	2	5	10
5-min	0.342 (0.311-0.377)	0.408 (0.371-0.450)	0.486 (0.440-0.535)	0.542 (0.490-0.596)
10-min	0.547 (0.497-0.602)	0.653 (0.594-0.720)	0.778 (0.704-0.856)	0.866 (0.783-0.953)
15-min	0.683 (0.621-0.753)	0.821 (0.747-0.905)	0.984 (0.890-1.08)	1.10 (0.991-1.21)
30-min	0.937 (0.852-1.03)	1.13 (1.03-1.25)	1.40 (1.26-1.54)	1.59 (1.44-1.75)
60-min	1.17 (1.06-1.29)	1.42 (1.29-1.57)	1.79 (1.62-1.97)	2.07 (1.87-2.28)
2-hr	1.41 (1.28-1.56)	1.72 (1.56-1.90)	2.18 (1.97-2.40)	2.53 (2.28-2.78)
3-hr	1.55 (1.40-1.72)	1.89 (1.71-2.10)	2.39 (2.16-2.66)	2.79 (2.51-3.09)
6-hr	1.96 (1.77-2.19)	2.38 (2.15-2.65)	3.01 (2.71-3.35)	3.52 (3.15-3.91)
12-hr	2.38 (2.15-2.66)	2.88 (2.60-3.23)	3.66 (3.30-4.10)	4.33 (3.88-4.83)
24-hr	2.76 (2.55-3.00)	3.33 (3.08-3.62)	4.23 (3.90-4.59)	4.99 (4.59-5.41)
2-day	3.19 (2.94-3.48)	3.86 (3.56-4.21)	4.91 (4.51-5.36)	5.78 (5.29-6.30)

Using the McCuen-Spiess limitation, the length over which sheet flow occurs is calculated to be:

$$L = \frac{100\sqrt{0.005}}{0.4} = 17.68 \text{ ft}$$

The travel time is then calculated entering the appropriate values into the equation:

$$T_t = \frac{0.007[(0.40)(17.68)]^{0.8}}{(3.33)^{0.5}(0.005)^{0.4}}$$

$$= 0.153 \text{ hr} = 9.18 \text{ min}$$

Step 2: Travel time under shallow concentrated flow is calculated as follows:

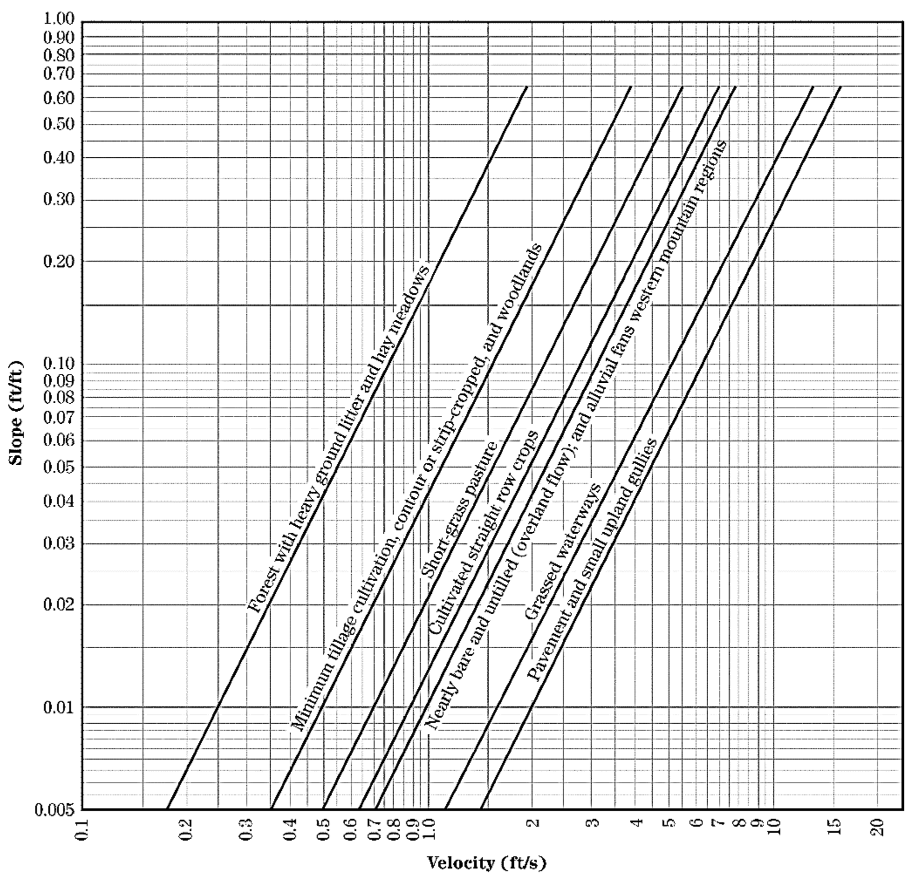
$$T_t = \frac{Shallow\ Concentrated\ Flow\ Length}{V\ x\ 3600}$$

where T_t is the travel time (hr) and V is the flow velocity (ft/s).

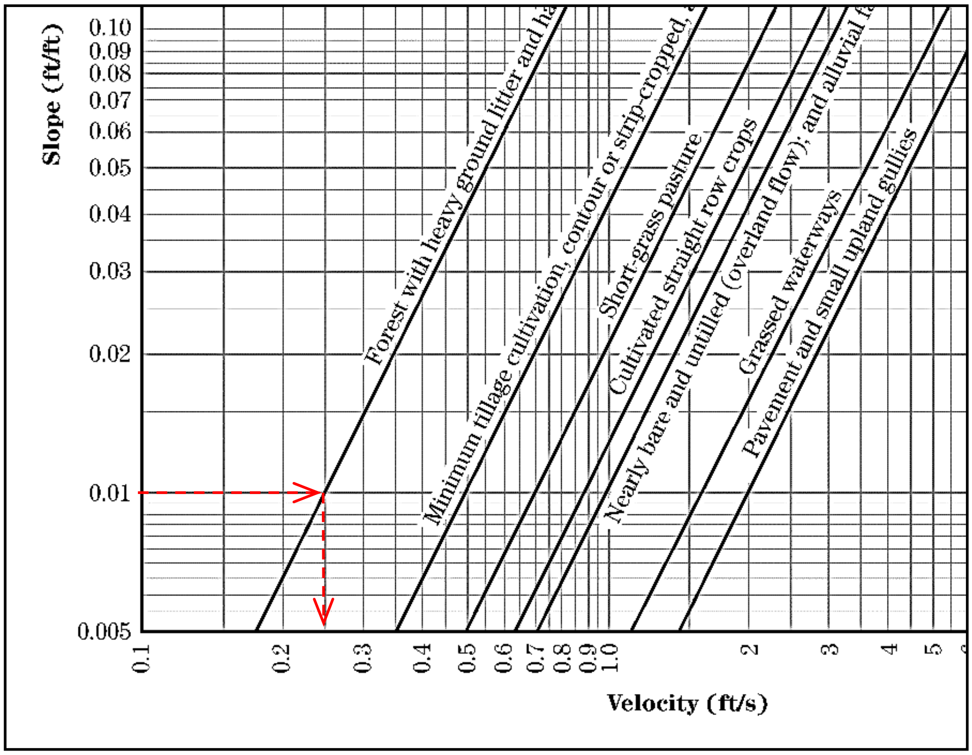
The total flow path length is 1,000 ft. Since the sheet flow segment length is 17.68 ft, the length of the shallow concentrated flow segment must be 982.32 ft. The value for the flow velocity can be determined from the graphical source from NEH. The velocities plotted in each are average values and are a function of watercourse slope and the cover condition of the channel.

The graphical source, reprinted below, is Figure 15-4 in *NEH, Part 630, Chapter* 15. This source was derived by solving Manning's equation for a wide variety of land covers.

Figure 15–4 Velocity versus slope for shallow concentrated flow



For this example, a horizontal line is projected across from the y-axis at the tic mark denoting the 1% slope to the curved representing forested areas.



The corresponding velocity is 0.25 ft/s. This value is then entered into the equation for the travel time, as follows:

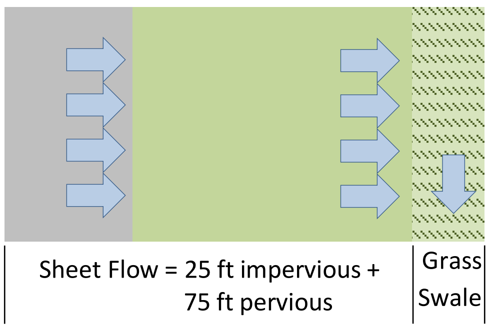
$$T_t = \frac{982.32}{0.25\ x\ 3600} = 1.09 \text{ hr} = 65.5 \text{ min}$$

Step 3: Since no channel flow is specified in the example, the time of concentration for the post-construction condition is the sum of the travel times under sheet flow and shallow concentrated flow, as follows:

$$T_c = 9.18 + 65.5 = 74.7 \text{ min, using Figure 15-4}$$

Example 5-2: Use the NRCS Composite CN Method for an Unconnected Impervious Surface to calculate the runoff volume for a site

A portion of a major development consists of a 200 ft wide, 25 ft long impervious surface and a 200 ft wide, 75 ft long grass lawn adjacent to the impervious surface. The stormwater runoff generated by the impervious surface will flow through the lawn area before it drains into the grass swale. The soils present are identified as HSG 'A.' The design storm event of concern is the 2-year storm, in which 3.5 inches of rain falls during a period of 24 hours. The slope of the impervious surface and the grass lawn area are each at 1%. From Table 9-5, in *NEH Part 630, Chapter 9* , a lawn area in HSG 'A' soil has a Curve Number of 39, under good condition.



Step 1: Calculate the Percentage of Total Impervious Surface

To use the NRCS composite CN with unconnected impervious area method , one must first know the percentage of the total impervious area to the total area. The percentage of the total impervious surface to the total area is

$$= (200\ ft\ x\ 25\ ft)/[(200\ ft\ x\ 25\ ft) + (200\ ft\ x\ 75\ ft)] = 0.25 = 25\%$$

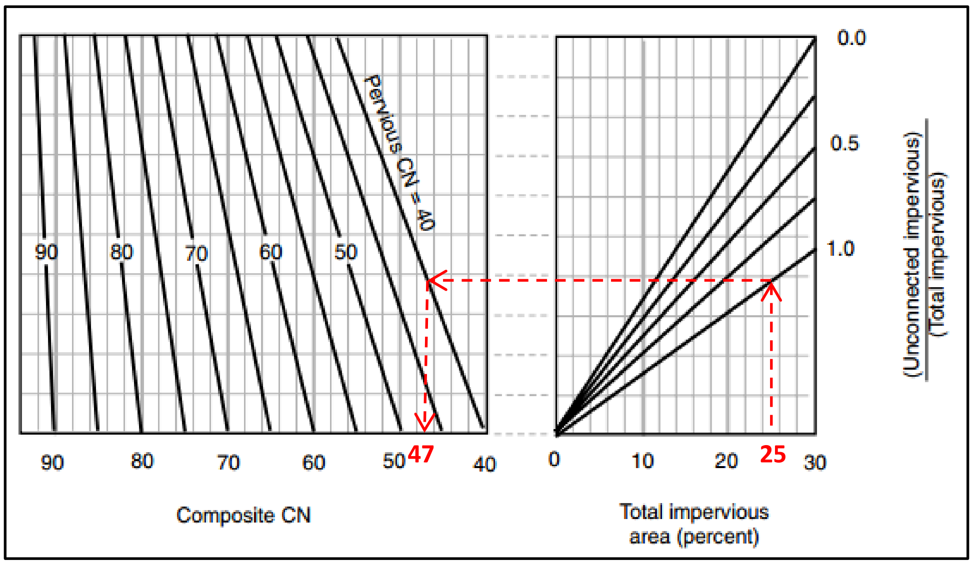
Since this percentage is less than the 30% maximum allowed (see the text at the top of Page 29), the NRCS composite CN with unconnected impervious area method is applicable.

Step 2: Ratio of Unconnected Impervious Surface to Total Impervious Surface

Secondly, one must determine the ratio of unconnected impervious surface to total impervious surface. In this case, all of the impervious surface present is the unconnected impervious surface under consideration; therefore, the ratio of unconnected impervious surface to total impervious surface is 1.

Step 3: Determine the Composite CN Representing Both the Unconnected Impervious and the Down-gradient Pervious Areas from the Pervious Area CN using NEH, Part 630, Chapter 9, Figure 9-4

Starting with the right side of Figure 9-4, reprinted below, find the intersection of the total impervious area with the line representing the ratio of unconnected impervious to total impervious. Draw a horizontal line across to intersect with the appropriate line representing the CN value of the site's pervious area. In this example, the lawn has a CN = 39, so the line for CN = 40 is used. A vertical line is next drawn down to connect with the x-axis to establish the composite CN value for the site, which is approximately 47. Take care reading the x-axis as the values increase from right to left. Therefore, a Curve Number = 47 can be used to represent the entire area measuring 200 ft wide and 100 ft long.



Source: Figure 9-4, *NEH, Part 630, Chapter 9*

Step 4: Use the Composite CN from Step 3 in the Runoff Depth Calculation

The runoff will be calculated by Equation 10-11 in *Chapter 10* of *NEH, Part 630*, as follows,

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)}$$

where:

Q = runoff, in

P = rainfall, in = 3.5 in

$S = \frac{1000}{CN} - 10 = \frac{1000}{47} - 10 = 11.3$, using the CN value determined in "Step 3"

Therefore,

$$Q = \frac{(3.5 - 0.2 \ x \ 11.3)^2}{(3.5 + 0.8 \ x \ 11.3)}$$

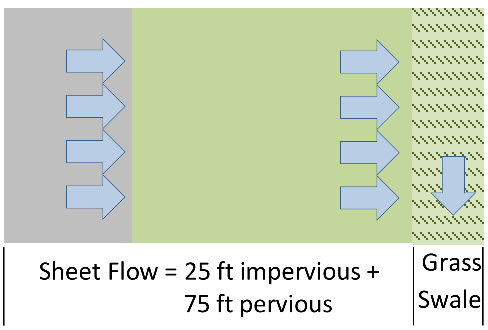
$$= \frac{(1.24)^2}{(12.5)} = 0.123 \text{ in}$$

Step 5: Calculate the Total Runoff Volume Generated by the Entire Area

The total runoff volume generated by the impervious surface and the lawn area is

$$= 0.123\ in\ \ x\ \ \frac{1\ ft}{12\ in}\ \ x\ \ 200\ ft\ \ x\ \ (25\ ft + 75\ ft) = 205 \text{ cf}$$

Example 5-3: Use the NJDEP Two-Step Method for an Unconnected Impervious Surface to calculate the runoff volume for a site



As can be surmised from the name, this method requires a two-step technique using the initial abstraction provided by NRCS runoff equation. First the volume of runoff generated by just the impervious area is calculated and then this volume is considered as if it were additional rain falling on the pervious area.

Step 1: Calculate Runoff Volume from Impervious Area

Use the NRCS runoff equation in a manner similar to the technique described in the previous example for impervious surfaces. For Curve Number 98:

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)}$$

where:

P = rainfall, in = 3.5 in

$$S = \frac{1000}{CN} - 10 = \frac{1000}{98} - 10 = 0.20$$

Therefore,

$$Q = \frac{(3.5 - 0.2\ x\ 0.20)^2}{(3.5 + 0.8\ x\ 0.20)} = 3.27 \text{ in}$$

The runoff volume generated by the impervious surface is calculated as was done in "Step 5" of Example 5-2:

$$= 3.27\ in\ x\ \frac{1\ ft}{12\ in}\ x\ 200\ ft\ \ x\ \ 25\ ft = 1{,}362.5\text{ cf}$$

Step 2: Convert the Runoff from the Impervious Surface to a Hypothetical Rainfall on the Pervious Area

Assume the entire runoff volume from "Step 1," i.e., 1,362.5 cf, is evenly distributed as rain falling on the adjacent pervious surface. The converted rainfall depth is calculated as follows:

$$= \frac{(1{,}362.5\ cf\ \ x\ \ (12\ in)/(1\ ft))}{(200\ ft\ x\ \ 75\ ft)} = 1.09\text{ in}$$

Note that only the sheet flow area (the area within the maximum 100 ft of flow path on the pervious surface) can be used to receive runoff from the impervious surface.

The total effective rainfall on the pervious surface is equal to the direct rainfall plus the unconnected impervious area runoff that was converted above to a hypothetical rainfall depth. This means 1.09 in is added to the design rainfall depth (3.5 in), resulting in a total rainfall depth of 4.59 in. The runoff generated by the grass lawn is then calculated using the runoff equation with this new value substituted for P, as follows:

$$S = \frac{1000}{CN} - 10 = \frac{1000}{39} - 10 = 15.64$$

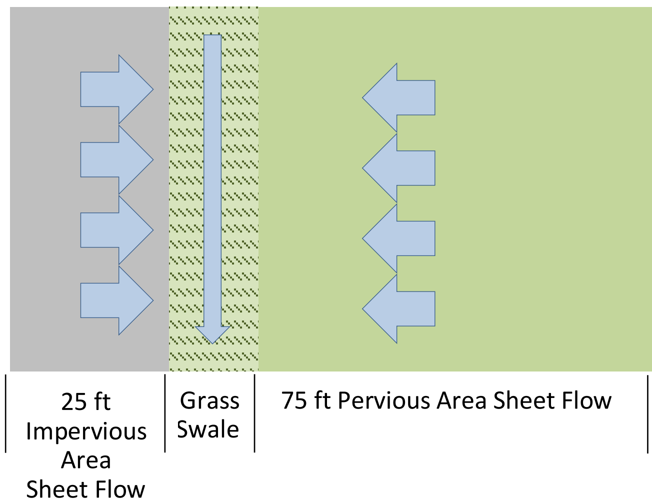
$$Q = \frac{(4.59 - 0.2\ \ x\ \ 15.64)^2}{(4.59 + 0.8\ \ x\ \ 15.64)} = 0.125\text{ in}$$

The total effective runoff volume generated is calculated as follows:

$$= 0.125\ in\ x\ \frac{1\ ft}{12\ in}\ x\ \ 200\ ft\ x\ 75\ ft = 156\text{ cf}$$

Example 5-4: Demonstration of Why a Composite CN Generates an Incorrect Runoff Volume

This example demonstrates the incorrect calculation of runoff volume by weighted CNs when the impervious surface is directly connected to the stormwater conveyance system. A portion of a major development consists of a 200 ft wide, 25 ft long impervious surface and a 200 ft wide, 75 ft long grass lawn area that are separated by a grass swale. In other words, the runoff from the impervious surface will flow directly into the grass swale. The soil is identified as belonging to HSG 'A.' The storm event of concern is the 2-year storm, in which 3.5 in of rain falls over a period of 24 hours. The slopes of the impervious surface and the grass lawn are each 1%. From Table 9-5 in *Chapter 9* of *NEH, Part 630*, the grass lawn area specified has a Curve Number of 39.



A value of 98 is used as the CN value for impervious surfaces. If a weighted composite CN were applied in this situation, the weighted composite CN would be calculated as follows:

$$CN = \frac{98\, x\, (200\, ft\ \ x\, 25\, ft) + 39\, x\, (200\, ft\, x\, 75\, ft)}{(200\, ft\ \ x\, 25\, ft) +\ (200\, ft\ \ x\ \ 75\, ft)} = 53.75$$

$$S = \frac{1000}{CN} - 10 = \frac{1000}{53.75} - 10 = 8.60$$

$$Q = \frac{(3.5 - 0.2\ \ x\, 8.60)^2}{(3.5 + 0.8\, x\, 8.60)} = 0.305 \text{ in}$$

The total runoff volume would then be calculated as follows:

$$= 0.305\, in\ \ x\ \ \frac{1\, ft}{12\, in}\, x\ \ 200\, ft\ \ x\ \ 100\, ft = 508 \text{ cf.}$$

To demonstrate why this is incorrect, the total runoff volume for each area is calculated separately and then added.

For the impervious area,

$S = 0.204$ and

$Q = 3.27$ in, as calculated previously in "Step 1" of Example 5-3.

The runoff volume generated by the impervious area was previously calculated to be 1,362.5 cf (see the top of Page 37).

For the pervious surface,

$$S = \frac{1000}{CN} - 10 = \frac{1000}{39} - 10 = 15.64$$

$$Q = \frac{(3.5 - 0.2\ x\ 15.64)^2}{(3.5 + 0.8\ x\ 15.64)} = 0.009 \text{ in}$$

which results in a runoff volume generated by the pervious area as follows:

$$= 0.009\ in\ x\ \frac{1\ ft}{12\ in}\ x\ 200\ ft\ x\ 75\ ft = 10.8 \text{ cf}$$

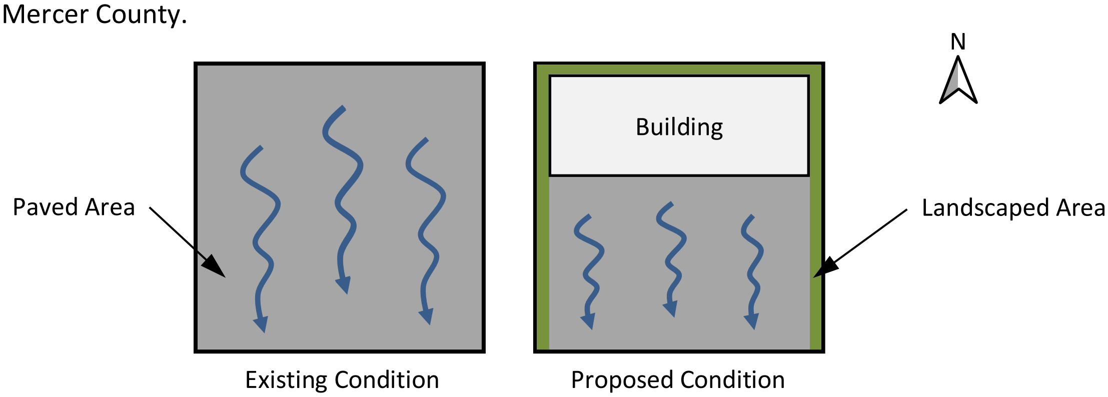
Adding these separately calculated volumes together yields the total runoff volume entering the grass swale equal to 1,373.3 cf. The previous, i.e. composite, calculation is only 37% of this volume.

The results show that the use of a weighted, or composite, CN in which pervious and impervious CN values are averaged will underestimate the runoff volume. Therefore, the use of weighted or composite CN values must not be used.

Example 5-5: A Comparison of Pre- and Post-condition Hydrographs for Compliance Under N.J.A.C. 7:8-5.4(a)3.i When Impervious Cover is Reduced

N.J.A.C. 7:8-5.4(a)3.i requires the design engineer choosing this option to demonstrate compliance with the quantity control requirements "*through hydrologic and hydraulic analysis that for stormwater leaving the site, post-construction runoff hydrographs for the two-, 10- and 100-year storm events do not exceed, at any point in time, the pre-construction runoff hydrographs for the same storm events*." This example provides a scenario showing noncompliance with the requirements when the proposed development reduces the regulated motor vehicle impervious surface and increases the slope of this surface.

An approximately 2 acre paved parking lot is to be redeveloped as an office complex consisting of a 0.5 acre new building, a 1.25 acre parking lot and landscaped areas totaling 0.25 acres. The existing lot is 300 ft x 300 ft with a slope of 1% from the north edge of the lot to the south edge of the lot. The runoff under existing conditions is as overland flow from the north side to the south side. The runoff generated by the proposed building is to be collected by a roof drainage system and directed via a downspout to the proposed parking lot where it will spread out as overland flow. The parking lot runoff is to remain as overland flow, but it will re-graded to be 5% slope for better drainage. The landscaped area is located on the north, east and west sides of the proposed development. The landscaped area will not receive runoff from the impervious surfaces. The precipitation depth in this example uses the county average rainfall depth for Mercer County.



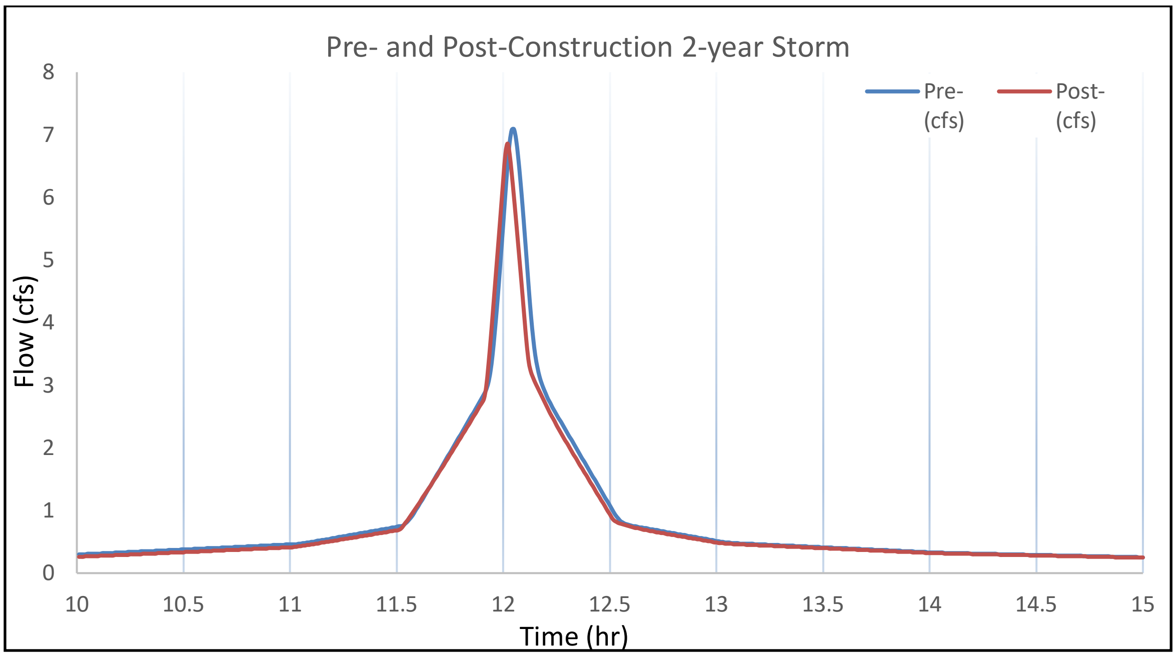
The pre-construction drainage pattern consists of sheet flow for the first 100 ft, followed by shallow concentrated flow for 200 ft. For pavement, the value for Manning's roughness coefficient is 0.011, as shown in Table 15-1, in *NEH, Part 630, Chapter 15*, and reprinted on Page 31. Rainfall depths for the 2-, 10- and 100-year storms are 3.31, 5.01 and 8.33 in, respectively.

The post-construction drainage pattern remains the same as the existing condition, i.e., flowing from the north to the south. The slope, however, is increased from 1% to 5%. The sheet flow length calculated by McCuen-Spiess limiting criteria exceeds 100 ft. Therefore, the sheet flow length must be limited to the maximum of 100 ft and therefore, the shallow concentrated flow length is 200 ft. However, the time of concentration is shorter due to the increased slope.

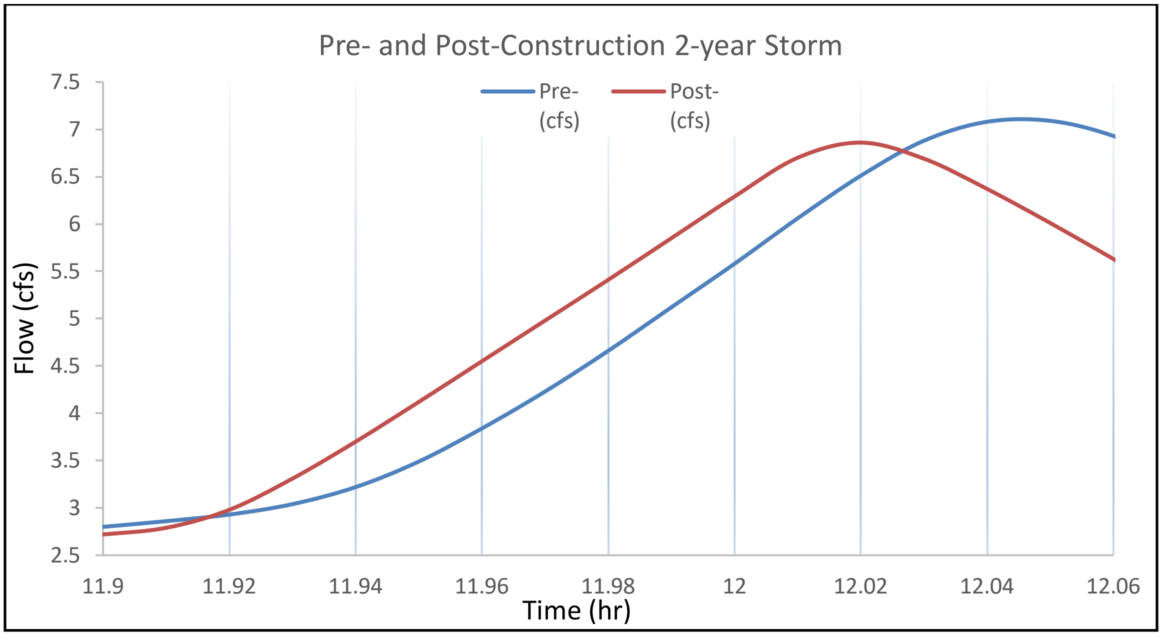
A summary of the results is shown in the table below:

2-year Design Storm		
Parameter	**Existing Condition**	**Proposed Condition**
Peak Flow Rate & Time of Peak =	7.11 cfs @ 12.05 hr	6.79 cfs @ 12.02 hr
Runoff Volume=	22,340 cf	20,719 cf

Pre- and post-condition hydrographs for the 2-year storm, calculated using the NRCS methodology, are depicted below as a reprint from a hydrologic modelling software package.



At first glance, one might assume the difference is negligible. However, the rules do not permit any exceedance. If one were to zoom in on the previous hydrograph, starting at 11.91 hours, one would see the post-construction hydrograph has a higher flow rate than the pre-construction hydrograph, as shown on the following page. This information is also listed in the table below the close-up of the hydrographs.



Time (hr)	Pre-construction (cfs)	Post-construction (cfs)	Difference in Flow Rate, Post - Pre, (cfs)
11.91	2.86	2.79	-0.07
11.92	2.93	2.98	0.05
11.93	3.04	3.31	0.27
11.94	3.22	3.70	0.48
11.95	3.49	4.12	0.63
11.96	3.84	4.55	0.71
11.97	4.23	4.98	0.75
11.98	4.66	5.41	0.75
11.99	5.12	5.85	0.73
12.00	5.58	6.29	0.71
12.01	6.06	6.70	0.64
12.02	6.51	6.79	0.28
12.03	6.88	6.69	-0.19

NRCS Methodology *(cont'd.)*

Although the reduction of impervious surface reduces the total volume of runoff and peak flow rate produced by the proposed construction, the design is not in compliance with N.J.A.C. 7:8-5.6(b)2, which requires that the post-construction runoff hydrographs do not exceed, at any point in time, the pre-construction runoff hydrographs for the same storm events, if the design engineer chooses to demonstrate the quantity control using this option. Since the hydrographs for the 2-year storm have already shown noncompliance, this example does not continue further to calculate hydrographs for the 10- and 100-year storms.

If the design engineer chooses to demonstrate compliance with the quantity control requirements under N.J.A.C. 7:8-5.4(a)3.iii, e.g., that the post-construction peak runoff rates for the 2-, 10- and 100-year storm events are 50, 75 and 80 percent, respectively, of the pre-construction peak runoff rates, the calculation of the 2-year storm will be as follows:

	2-year Design Storm		
	Existing Peak Flow Rate	**Allowable Post-Construction (50% Reduction)**	**Proposed Peak Flow Rate**
Peak Flow Rate =	7.11 cfs	3.56 cfs	6.79 cfs

The 2-year post-construction peak flow rate, 6.79 cfs, exceeds the allowable flow rate, 3.56 cfs. Therefore, the project is still not in compliance with the stormwater runoff quantity control requirement under the option found in N.J.A.C. 7:8-5.4(a)3.iii. A stormwater runoff quantity control or green infrastructure BMP will be required to reduce the post-construction peak flow to 50%, or lower, of the pre-construction peak flow rate.

This example dispels the common misconception that the reduction of impervious surface will automatically meet the quantity control requirements. Municipal review engineers must require that the design report include hydrologic modelling and hydrographs even when the design engineer claims there is reduction of impervious surface by the proposed development. To reiterate, the rules do not allow a "de minimus" exception.

Examples 5-6 and 5-7: A Re-development Project with Two Drainage Areas, Each Discharging to Separate Points, Compared to the Same Development having One Combined Discharge Point

In this example, a proposed development in Ocean County consists of two drainage areas, each discharging to a separate point, is compared to a scenario in which the stormwater runoff produced by the two drainage areas converges to a single discharge point before leaving the development site. **This example combines a groundwater mounding analysis and stormwater routing calculations, which includes exfiltration as an allowable discharge, and also illustrates how the stormwater runoff quantity control design standards may and <u>may not</u> be applied.**

Example 5-6: Two Discharge Points

Drainage area A is 1.0 acre in area. Under existing conditions, stormwater runoff from drainage area A is discharged to a riparian zone toward a small creek. The existing cover in drainage area A is a pasture on HSG 'B' soil. The proposed development for drainage area A consists of a 0.25 acre gravel parking lot, with the remainder to be undisturbed. A small-scale infiltration basin (52 ft long by 52 ft wide) is proposed to provide water-quality treatment for the stormwater runoff generated by the proposed gravel parking lot, as well as provide stormwater runoff quantity control through infiltration of the runoff produced by the 2-, 10- and 100-year design storms. The small-scale infiltration basin has an emergency spillway discharging to the same riparian zone toward to the creek.

Drainage area B is 0.75 acres in area. Under existing conditions, stormwater runoff from drainage area B is discharged to a street that has a roadside catch basin connected to a municipal stormwater sewer system. Drainage area B consists of HSG 'C' soil. The concrete foundation of an abandoned warehouse covers 0.5 acres of drainage area B. Although there was an asphalt parking area adjacent to the warehouse, the parking area was removed and vegetation, grass and woods, have re-established on the site. The proposed development includes demolishing the warehouse to construct a 0.16 ac building and pedestrian walkway. Under proposed conditions, the rest of the drainage area B is to be vegetated as follows: 0.59 ac of turf grass and landscaping. No green infrastructure is proposed in drainage area B.

The tested soil permeability rate for the most restrictive soil layer within the proposed small-scale infiltration basin is 3 in/hr. The Seasonal High Water Table (SHWT) is 8 feet below the existing ground elevation. The stormwater management report for the proposed development claims that the proposed stormwater management measures will meet the stormwater runoff quantity requirements in N.J.A.C. 7:8-5.6. Determine the validity of this claim.

Step 1: Determine Whether the Project is a Major Development

The proposed development will have 0.25 acres of gravel parking lot in drainage area A and a new 0.16 ac building and walkway plus 0.59 acres of grass lawn in drainage area B. The total disturbance is 1.00 acre and the project creates one-quarter acre of regulated motor vehicle surface. Therefore, the proposed development is a major development.

Step 2: Stormwater Runoff Quantity Standards:

For a major development project, stormwater runoff quantity control is required. The option to demonstrate compliance with N.J.A.C. 7:8-5.6(b)3 is chosen. The peak flow rates for pre-construction condition are calculated as follows:

- CN values for the pre-construction condition are chosen based on the land cover having the least runoff potential and assuming good condition rather than fair or poor. The asphalt parking lot cannot be considered to be impervious surface since it has been removed and vegetation has been re-established.

- The time of concentration is calculated by using a flow path of sheet flow computed by McCuen-Spiess limitation, plus a segment of shallow concentrated flow based the land cover specified above for the drainage area. The 2-, 10- and 100- year design storms produce rainfall depths, in Ocean County, of 3.4, 5.4 and 9.2 inches, respectively. For this example, the existing condition has two points of analysis: POA-A and POA-B. The times of concentration, volumes and peak flow rates from impervious surfaces and pervious surfaces must be calculated separately and the results are shown in the table below. When subdrainage areas have different times of concentration, the combined flow rate cannot be the sum of the peak flow rates for each of the subdrainage areas. In such an instance, the hydrographs for each of the subdrainage areas must be added, and then the peak flow rate is obtained from the results.

Pre-construction Drainage Area Name (cover condition or undisturbed)	**Area (ac)**	**CN**	**T_c (min)**	**Pre-construction Design Storm Flow Rate (cfs)**		
				2-year	**10-year**	**100-year**
Pre-A1 (pasture)	0.25	61	17.4	0.07	0.30	0.88
Pre-A2 (undisturbed)	0.75	61	24.2	0.19	0.78	2.26
Total Pre-A (Hydrograph addition)	**1.00**			**0.26**	**1.05**	**3.07**
Pre-B1 (warehouse)	0.50	98	2.8	1.76	2.81	4.80
Pre-B2 (woods/grass)	0.25	70	18.6	0.17	0.45	1.07
Total Pre-B (Hydrograph addition)	**0.75**			**1.83**	**3.03**	**5.37**

- Allowable post-construction peak flow under N.J.A.C. 7:8-5.6(b)3:

  Under N.J.A.C. 7:8-5.6(b)3, the post-construction condition peak flow rates must be reduced to 50%, 75% and 80% of the pre-construction peak flow rates, respectively. It is also stated in the rules that the percentages apply only to the post-construction stormwater runoff that is attributable to the portion of the site on which the proposed development or project is to be constructed. Therefore, the reduction percentages are not required for the undisturbed drainage area, Pre-A2. The allowable peak flow rates are listed below.

Pre-construction Drainage Area Name (cover condition)	Area (ac)	Allowable Design Storm Peak Flow Rates (cfs)		
		2-year	10-year	100-year
Pre-A1 (pasture)	0.25	0.04	0.23	0.70
Pre-A2 (undisturbed)	0.75	0.19	0.78	2.26
Total Pre-A	**1.00**	**0.23**	**1.01**	**2.96**
Pre-B1 (warehouse)	0.50	0.88	2.11	3.84
Pre-B2 (woods/grass)	0.25	0.09	0.34	0.86
Total Pre-B	**0.75**	**0.92**	**2.27**	**4.30**

- The post-construction peak flow rates for each of the proposed drainage areas (prior to the incorporation of stormwater management measures) are shown below:

Post-construction Drainage Area Name (undisturbed/cover condition)	Area (ac)	CN	T_c (min)	Post-construction Design Storm Flow Rate (cfs)		
				2-year	10-year	100-year
Post-A1 (parking/ gravel)	0.25	96	3.5	0.83	1.35	2.33
Post-A2 (undisturbed/ pasture)	0.75	61	24.2	0.19	0.78	2.26
Total Post-A (Hydrograph addition)	**1.00**			0.85	1.62	3.31
Post-B1 (building or walkway)	0.16	98	1.6	0.57	0.90	1.54
Post-B2 (open space/ grass 50% to 75%)	0.59	79	8.9	0.90	1.91	3.93
Total Post-B (Hydrograph addition)	**0.75**			**1.24**	**2.41**	**4.74**

- □ The uncontrolled post-construction peak flow rates generated by drainage area Post-A exceed the allowable design storm peak flow rates of stormwater runoff generated by drainage area Pre-A. Therefore, green infrastructure BMPs listed in Table 5-1 or 5-2, as required by N.J.A.C. 7:8-5.3(c), must be used to meet the stormwater runoff quantity control requirements. As stated on Page 44, a small-scale infiltration basin is proposed to provide the stormwater runoff quantity control.
- □ Similarly, for drainage area B, the uncontrolled post-construction peak flow rates of stormwater runoff generated by drainage area Post-B exceed the allowable design storm peak flow rates of stormwater runoff generated by drainage area Pre-B. Therefore, green infrastructure BMPs listed in Table 5-1 or 5-2 must also be used to meet the stormwater runoff quantity control requirements.

Step 3: Calculate the Post-Construction Design Storm Peak Flow Rates for Drainage Area Post-A-1

The proposed small-scale infiltration basin has a 2,700 sf bottom footprint and a basin depth of 2 ft, plus 1 ft of freeboard. The 4-sided concrete outlet structure includes a 2.5 in orifice located 0.15 ft above the basin bottom and a top grate, which acts as a broad-crested weir, located 1 ft above the basin bottom. The weir opening is 20 ft long = 4 sides x 5 ft per side, and the depth of the weir, i.e., the outlet structure wall thickness, is 0.5 ft. Exfiltration is included in the routing calculation, using the design permeability rate of 1.5 in/hr, which is one-half of the tested permeability rate of 3 in/hr. Take note that the exfiltration can be credited only on the infiltration area, which is the footprint of the BMP. The side slopes of the BMP cannot be used for exfiltration. Therefore, the routing, with exfiltration, must only use the design permeability rate of the most restrictive soil layer under the BMP and the footprint, i.e., the exfiltration area, of the BMP to determine the maximum exfiltration flow rate (cfs) in the BMP. In the current case, however, the BMP has the same surface area 2,700 sf from the bottom to the top. A constant exfiltration flow rate, 0.09375 cfs (2,700 sf x 1.5 in/hr x 1/12 in/ft x 1/3600 second/hr), in the routing or a constant exfiltration rate 1.5 in/hr is applied to 2,700 sf for all water elevations in the routing. The results obtained from a hydraulic and hydrologic modeling software program are each shown on the following pages for the 2-, 10- and 100-year design storms.

2-year Design Storm Post-Construction Condition Summary Report

```
Inflow Area =       10,890 sf,   0.00% Impervious,  Inflow Depth =  2.95"   for 2-Year event
Inflow      =        0.83 cfs @  12.09 hrs,  Volume=           2,674 cf
Outflow     =        0.12 cfs @  12.57 hrs,  Volume=           2,674 cf,  Atten= 85%,  Lag= 28.8 min
Discarded   =        0.09 cfs @  11.60 hrs,  Volume=           2,532 cf
Primary     =        0.03 cfs @  12.57 hrs,  Volume=             142 cf

Routing by Stor-Ind method, Time Span= 0.00-48.00 hrs, dt= 0.05 hrs
Peak Elev= 0.29' @ 12.57 hrs   Surf.Area= 2,700 sf   Storage= 777 cf

Plug-Flow detention time= 51.7 min calculated for 2,674 cf (100% of inflow)
Center-of-Mass det. time= 51.6 min ( 825.1 - 773.5 )
```

Volume	Invert	Avail.Storage	Storage Description
#1	0.00'	8,100 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
0.00	2,700	0	0
1.00	2,700	2,700	2,700
2.00	2,700	2,700	5,400
3.00	2,700	2,700	8,100

Device	Routing	Invert	Outlet Devices
#1	Primary	1.00'	**20.0' long x 0.5' breadth Broad-Crested Rectangular Weir** Head (feet) 0.20 0.40 0.60 0.80 1.00 Coef. (English) 2.80 2.92 3.08 3.30 3.32
#2	Primary	0.15'	**2.5" Vert. Orifice/Grate** C= 0.600
#3	Discarded	0.00'	**1.50 in/hr Exfiltration over Surface area**

Discarded OutFlow Max=0.09 cfs @ 11.60 hrs HW=0.03' (Free Discharge)
└3=**Exfiltration** (Exfiltration Controls 0.09 cfs)

Primary OutFlow Max=0.03 cfs @ 12.57 hrs HW=0.29' (Free Discharge)
├1=**Broad-Crested Rectangular Weir** (Controls 0.00 cfs)
└2=**Orifice/Grate** (Orifice Controls 0.03 cfs @ 1.26 fps)

Source: HydroCAD® Summary Report; HydroCAD is a registered trademark of HydroCAD Software Solutions LLC. Used with permission.

10-year Design Storm Post-Construction Condition Summary Report

Inflow Area = 10,890 sf, 0.00% Impervious, Inflow Depth = 4.93" for 10-Year event
Inflow = 1.35 cfs @ 12.09 hrs, Volume= 4,474 cf
Outflow = 0.18 cfs @ 12.62 hrs, Volume= 4,474 cf, Atten= 87%, Lag= 31.7 min
Discarded = 0.09 cfs @ 11.10 hrs, Volume= 3,672 cf
Primary = 0.09 cfs @ 12.62 hrs, Volume= 802 cf

Routing by Stor-Ind method, Time Span= 0.00-48.00 hrs, dt= 0.05 hrs
Peak Elev= 0.54' @ 12.62 hrs Surf.Area= 2,700 sf Storage= 1,451 cf

Plug-Flow detention time= 72.0 min calculated for 4,470 cf (100% of inflow)
Center-of-Mass det. time= 72.0 min (832.6 - 760.6)

Volume	Invert	Avail.Storage	Storage Description
#1	0.00'	8,100 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
0.00	2,700	0	0
1.00	2,700	2,700	2,700
2.00	2,700	2,700	5,400
3.00	2,700	2,700	8,100

Device	Routing	Invert	Outlet Devices
#1	Primary	1.00'	**20.0' long x 0.5' breadth Broad-Crested Rectangular Weir** Head (feet) 0.20 0.40 0.60 0.80 1.00 Coef. (English) 2.80 2.92 3.08 3.30 3.32
#2	Primary	0.15'	**2.5" Vert. Orifice/Grate** C= 0.600
#3	Discarded	0.00'	**1.50 in/hr Exfiltration over Surface area**

Discarded OutFlow Max=0.09 cfs @ 11.10 hrs HW=0.03' (Free Discharge)
└─**3=Exfiltration** (Exfiltration Controls 0.09 cfs)

Primary OutFlow Max=0.09 cfs @ 12.62 hrs HW=0.54' (Free Discharge)
├─**1=Broad-Crested Rectangular Weir** (Controls 0.00 cfs)
└─**2=Orifice/Grate** (Orifice Controls 0.09 cfs @ 2.56 fps)

Source: HydroCAD® Summary Report; HydroCAD is a registered trademark of HydroCAD Software Solutions LLC. Used with permission.

100-year Design Storm Post-Construction Condition Summary Report

Inflow Area = 10,890 sf, 0.00% Impervious, Inflow Depth = 8.72" for 100-Year event
Inflow = 2.33 cfs @ 12.09 hrs, Volume= 7,912 cf
Outflow = 0.45 cfs @ 12.42 hrs, Volume= 7,912 cf, Atten= 81%, Lag= 19.7 min
Discarded = 0.09 cfs @ 10.00 hrs, Volume= 5,296 cf
Primary = 0.36 cfs @ 12.42 hrs, Volume= 2,616 cf

Routing by Stor-Ind method, Time Span= 0.00-48.00 hrs, dt= 0.05 hrs
Peak Elev= 1.02' @ 12.40 hrs Surf.Area= 2,700 sf Storage= 2,762 cf

Plug-Flow detention time= 103.3 min calculated for 7,904 cf (100% of inflow)
Center-of-Mass det. time= 103.2 min (852.1 - 748.9)

Volume	Invert	Avail.Storage	Storage Description
#1	0.00'	8,100 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
0.00	2,700	0	0
1.00	2,700	2,700	2,700
2.00	2,700	2,700	5,400
3.00	2,700	2,700	8,100

Device	Routing	Invert	Outlet Devices
#1	Primary	1.00'	**20.0' long x 0.5' breadth Broad-Crested Rectangular Weir** Head (feet) 0.20 0.40 0.60 0.80 1.00 Coef. (English) 2.80 2.92 3.08 3.30 3.32
#2	Primary	0.15'	**2.5" Vert. Orifice/Grate** C= 0.600
#3	Discarded	0.00'	**1.50 in/hr Exfiltration over Surface area**

Discarded OutFlow Max=0.09 cfs @ 10.00 hrs HW=0.03' (Free Discharge)
└3=**Exfiltration** (Exfiltration Controls 0.09 cfs)

Primary OutFlow Max=0.34 cfs @ 12.42 hrs HW=1.02' (Free Discharge)
├1=**Broad-Crested Rectangular Weir** (Weir Controls 0.20 cfs @ 0.43 fps)
└2=**Orifice/Grate** (Orifice Controls 0.14 cfs @ 4.22 fps)

Source: HydroCAD® Summary Report; HydroCAD is a registered trademark of HydroCAD Software Solutions LLC. Used with permission.

The post-construction peak flow discharges from the proposed small-scale infiltration basin, which are labeled "Primary" flow rates in the above listed reports, were calculated to be 0.03, 0.09 and 0.36 cfs for the 2-, 10- and 100-year design storms, respectively. Adding the hydrograph to the corresponding flow rates generated by the undisturbed drainage area Post-A2, yields the total calculated peak flow rates of Post-A drainage area - 0.19 cfs, 0.78 and 2.26 cfs - for the 2-, 10- and 100-year design storms, respectively. The table on the following page compares the allowable design storm peak flow rates for the post-construction condition for Post-A drainage area at point of analysis A to those calculated for the post-construction condition to check whether the design meets the requirement to reduce the peak flow rates, respectively, to 50, 75 and 80% of the pre-construction peak flow rates.

The post-construction peak flow rates for each of the proposed drainage areas are shown below (after the incorporation of stormwater management measures).

Post-construction Drainage Area Designation	Allowable Design Storm Peak Flow Rate (cfs)			Design Storm Peak Flow Rates with a Small-Scale Infiltration Basin (cfs)		
	2-year	**10-year**	**100-year**	**2-year**	**10-year**	**100-year**
Post-A1 (parking lot/ gravel)	0.04	0.23	0.70	0.03	0.09	0.36
Post-A2 (undisturbed area/ pasture)	0.19	0.78	2.26	0.19	0.78	2.26
Post-A	**0.23**	**1.01**	**2.96**	**0.22**	**0.86**	**2.57**

Step 4: Perform the Required Groundwater Mounding Analysis

The design of the small-scale infiltration basin must include a groundwater mounding analysis to verify the drain time within 72 hours and whether there is an adverse hydraulic impact to the groundwater level due to the infiltration practice, in accordance with N.J.A.C. 7:8-5.2(h).

The NJDEP *Hantush Spreadsheet* is used for the groundwater mounding analysis for which guidance is included in *Chapter 13* of this manual. For this example, the small-scale infiltration basin measures 52 ft by 52 ft. The recharge rate, R, is the design permeability rate, which is one half the tested permeability rate, and equals 0.5 x 3.0 in/hr = 1.5 in/hr. The parameters for specific yield, S_y, and the horizontal hydraulic conductivity, Kh, is will be set at the default values specified in *Chapter 13*. The horizontal hydraulic conductivity rate is 5 times the recharge rate since the site, located in Ocean County, is in the Coastal plain. The x and y values are equal to half of the respective basin dimensions.

For the calculation of the duration of the infiltration period - *when exfiltration is used in the basin routing calculation* - the time calculated must be determined from the volume of runoff exfiltrated, or discarded, typically found in the modeling software results. As shown in the preceding summary reports, the exfiltration volumes or discarded volumes during the 2-, 10- and 100-year design storms are 2,532, 3,672 and 5,296 cf, respectively. Since the 100-year design storm produced the largest exfiltration volume, this design storm is most likely to yield the greatest mounding height. The exfiltration volume during the 100-year storm is used to calculate the duration of the infiltration period.

From Page 4 of *Chapter 13*, the duration of infiltration period, t, is calculated as follows for the 100-year design storm:

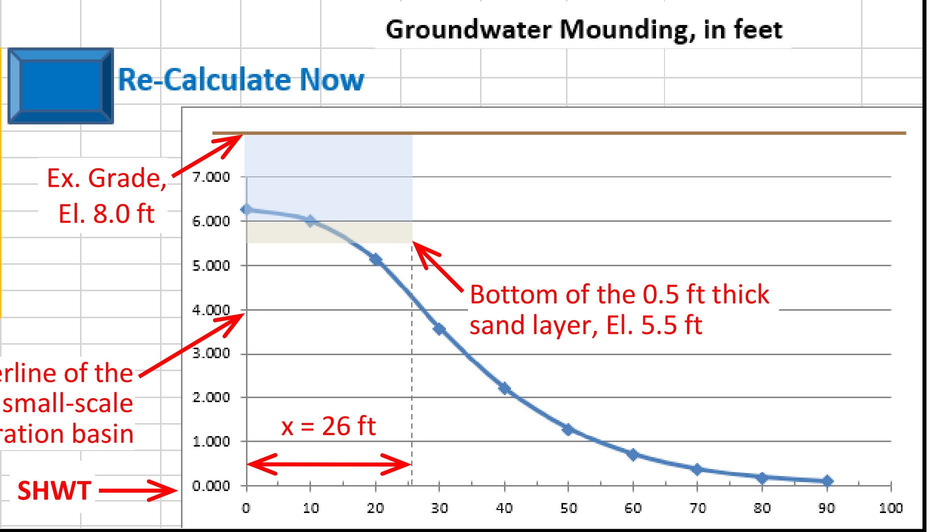
$$\text{Duration of infiltration period, t } (hr) = \frac{Discarded\ Volume\ via\ Exfiltration\ (cf)\ x\ 12\ in/ft}{Infiltration\ area\ (sf)\ x\ Exfiltration\ rate\ (in/hr)}$$

$$= \frac{5{,}296\ cf\ \ x\ 12\ in/ft}{2{,}700\ sf\ x\ 1.5\ in/hr} = 15.69 \text{ hr}$$

The initial thickness of the saturated zone, hi(0), is set at the default value. Clicking on the blue button labeled "Re-Calculate Now" produced the results depicted below.

Input Values		
1.50	R	Recharge rate (permeability rate) (in/hr)
0.150	Sy	Specific yield, Sy (dimensionless) default value is 0.15; max value is 0.2 provided that a lab test data is submitted
7.50	Kh	Horizontal hydraulic conductivity (in/hr) Kh = 5xRecharge Rate (R) in the costal plan; Kh=R outside the coastal plan
26.000	x	1/2 length of basin (x direction, in feet)
26.000	y	1/2 width of basin (y direction, in feet)
15.69	t	Duration of infiltration period (hours)
10.000	hi(0)	Initial thickness of saturated zone (feet)
16.26	h(max)	Maximum thickness of saturated zone (beneath center of basin at end of infiltration period)
6.26	Δh(max)	Maximum groundwater mounding (beneath center of basin at end of infiltration period)

Ground-water Mounding, in feet	Distance from center of basin in x direction, in feet
6.259	0
5.991	10
5.123	20
3.558	30
2.206	40
1.287	50
0.710	60
0.372	70
0.186	80
0.090	90



The results show that the maximum height of the groundwater mounding, Δh(max), is 6.26 ft. Since the groundwater table is 8 ft below the existing ground elevation, the elevated groundwater table will be 1.74 ft below the existing ground level at the center of the small-scale infiltration basin at the moment all of the stormwater runoff has infiltrated into the sand layer. However, the proposed small-scale basin provides 2 ft of temporary storage for stormwater, as depicted above by the light blue rectangle. The basin also includes a sand layer that is 0.5 ft deep, shown in tan. Therefore, the lowest point of the proposed basin is 2.5 ft below the existing ground elevation. Since the elevated groundwater table will be only 1.74 ft below the existing ground elevation, the elevated groundwater level will have an adverse impact on the drainage of the basin.

Since there is an adverse impact on the drainage of the basin, the rate of exfiltration will also be impacted, meaning the ability of the basin to drain will be reduced. A smaller recharge rate must be used to run the *Hantush Spreadsheet* again in order to further evaluate the mounding height. **A trial and error approach must be undertaken because the extent to which the soil permeability rate will be reduced is unknown.** The approach is outlined below.

Steps to Follow When an Adverse Impact is Encountered:

a. Further reduce the value for the recharge rate, R.

b. Use the reduced recharge rate from "Step a" as the exfiltration rate in the BMP routing calculation.

   i. Keep in mind that reducing the exfiltration rate will result in a greater volume to be discharged via the outlet structure and a lower volume of stormwater runoff will be discarded via exfiltration.
   ii. If the increased flow rate through the outlet structure exceeds the allowable design storm peak flow rate, the infiltration BMP will require a larger detention volume and the outlet structure may also need to be adjusted to meet the allowable design storm peak flow rate.
   iii. Although the BMP can be enlarged by increasing the depth, doing so will reduce the distance between the bottom of the BMP and the groundwater table, meaning the new design will be more likely to be negatively impacted by the elevated groundwater table during infiltration.
   iv. Therefore, enlarging the footprint of the BMP will generally produce more favorable results than increasing the depth.

c. After adjusting the footprint size and the outlet structure to meet the allowable design storm peak flow rates for all storms with the reduced exfiltration rate, use the exfiltration (discarded) volume from the routing calculation as the infiltration volume to calculate a new value for the duration of infiltration period. **Note that the new duration of the infiltration period must be less than or equal to 72 hours.**

d. Run the *Hantush Spreadsheet* with the new values from "Step c."

   i. The recharge rate will be equal to the reduced exfiltration rate.
   ii. However, the horizontal conductivity must remain unchanged.
   iii. **If the BMP, using the reduced exfiltration rate, needs more than 72 hours to infiltrate the exfiltration volume, the BMP is considered unsuitable.**

e. Use the mounding height from the results in "Step d" to assess the impact of the groundwater mounding on the BMP.

   i. If the new height of the groundwater mounding is below the bottom of the BMP, the infiltration practice will not be impacted by the groundwater mounding, and the trial and error process is concluded at this point.

ii. However, if the new height of the groundwater mounding is still above the bottom of the BMP, a new iteration using a further reduced exfiltration rate will be needed.

iii. **Note that an exfiltration rate less than 0.5 in/hr may be used as long as the duration of infiltration period does not exceed 72 hours.**

- Based on the methodology discussed above, the trial and error approach for the current example is as follows:

a. Reduce the recharge rate from 1.5 in/hr to 1.1 in/hr.

b. The exfiltration rate used in the new basin routing calculation equals the recharge rate from "Step a." The results are shown in the image below:

Revised 100-year Design Storm Post-Construction Condition Summary Report
Exfiltration = 1.1 in/hr

```
Inflow Area =      10,890 sf,   0.00% Impervious,  Inflow Depth =  8.72"   for  100-Year event
Inflow      =     2.31 cfs @  12.10 hrs,  Volume=          7,912 cf
Outflow     =     0.65 cfs @  12.30 hrs,  Volume=          7,912 cf,  Atten= 72%,  Lag= 11.8 min
Discarded   =     0.07 cfs @   9.28 hrs,  Volume=          4,603 cf
Primary     =     0.58 cfs @  12.30 hrs,  Volume=          3,309 cf

Routing by Stor-Ind method, Time Span= 0.00-48.00 hrs, dt= 0.04 hrs
Peak Elev= 1.04' @ 12.30 hrs   Surf.Area= 2,700 sf   Storage= 2,802 cf

Plug-Flow detention time= 124.3 min calculated for 7,912 cf (100% of inflow)
Center-of-Mass det. time= 124.1 min ( 873.0 - 748.9 )

Volume        Invert     Avail.Storage   Storage Description
  #1          0.00'         8,100 cf     Custom Stage Data (Prismatic) Listed below (Recalc)

 Elevation        Surf.Area        Inc.Store        Cum.Store
   (feet)           (sq-ft)      (cubic-feet)     (cubic-feet)
     0.00            2,700               0                0
     1.00            2,700           2,700            2,700
     2.00            2,700           2,700            5,400
     3.00            2,700           2,700            8,100

Device    Routing          Invert    Outlet Devices
  #1      Primary           1.00'    20.0' long  x 0.5' breadth Broad-Crested Rectangular Weir
                                     Head (feet)  0.20  0.40  0.60  0.80  1.00
                                     Coef. (English)  2.80  2.92  3.08  3.30  3.32
  #2      Primary           0.15'    2.5" Vert. Orifice/Grate    C= 0.600
  #3      Discarded         0.00'    1.10 in/hr Exfiltration over Surface area

Discarded OutFlow  Max=0.07 cfs @ 9.28 hrs  HW=0.03'   (Free Discharge)
  3=Exfiltration  (Exfiltration Controls 0.07 cfs)

Primary OutFlow  Max=0.54 cfs @ 12.30 hrs  HW=1.04'   (Free Discharge)
  1=Broad-Crested Rectangular Weir  (Weir Controls 0.40 cfs @ 0.54 fps)
  2=Orifice/Grate  (Orifice Controls 0.15 cfs @ 4.26 fps)
```

Source: HydroCAD® Summary Report; HydroCAD is a registered trademark of HydroCAD Software Solutions LLC. Used with permission

i. The 100-year storm routing calculation shows the exfiltration (discarded) volume is reduced from 5,296 cf at an exfiltration rate of 1.5 in/hr to 4,603 cf at 1.1 in/hr.

ii. The peak flow rate (primary) from the proposed small-scale infiltration basin (Post-A1) is increased from 0.36 cfs to 0.58 cfs and must be added to the flow from Post-A2 (undisturbed area/ pasture). The new value for the combined peak flow rate by hydrograph addition is

2.81 cfs, which does not exceed the allowable design storm peak flow rate, 2.96 cfs, for the Post-A drainage area.

iii. The peak flow rates for 2- and 10-year design storms from the small-scale infiltration basin are calculated and the results are shown below:

Revised 2-year Design Storm Post-Construction Condition Summary Report
Exfiltration = 1.1 in/hr and Basin Footprint Remains = 2,700 sf

```
Inflow Area =        10,890 sf,   0.00% Impervious,  Inflow Depth =  2.95"   for  2-Year event
Inflow       =      0.84 cfs @   12.10 hrs,  Volume=            2,674 cf
Outflow      =      0.11 cfs @   12.62 hrs,  Volume=            2,674 cf,  Atten= 87%,  Lag= 31.4 min
Discarded =         0.07 cfs @   11.36 hrs,  Volume=            2,395 cf
Primary      =      0.04 cfs @   12.62 hrs,  Volume=              279 cf

Routing by Stor-Ind method, Time Span= 0.00-48.00 hrs, dt= 0.04 hrs
Peak Elev= 0.32' @ 12.62 hrs   Surf.Area= 2,700 sf   Storage= 877 cf

Plug-Flow detention time= 75.8 min calculated for 2,672 cf (100% of inflow)
Center-of-Mass det. time= 75.8 min ( 849.3 - 773.5 )

Volume          Invert       Avail.Storage    Storage Description
  #1              0.00'          8,100 cf     Custom Stage Data (Prismatic) Listed below (Recalc)

  Elevation            Surf.Area         Inc.Store        Cum.Store
     (feet)               (sq-ft)        (cubic-feet)     (cubic-feet)
       0.00                2,700                  0                0
       1.00                2,700              2,700            2,700
       2.00                2,700              2,700            5,400
       3.00                2,700              2,700            8,100

Device   Routing           Invert    Outlet Devices
  #1     Primary            1.00'    20.0' long  x 0.5' breadth Broad-Crested Rectangular Weir
                                     Head (feet) 0.20  0.40  0.60  0.80  1.00
                                     Coef. (English)  2.80  2.92  3.08  3.30  3.32
  #2     Primary            0.15'    2.5" Vert. Orifice/Grate     C= 0.600
  #3     Discarded          0.00'    1.10 in/hr Exfiltration over Surface area

Discarded OutFlow  Max=0.07 cfs @ 11.36 hrs  HW=0.03'   (Free Discharge)
 └3=Exfiltration  (Exfiltration Controls 0.07 cfs)

Primary OutFlow  Max=0.04 cfs @ 12.62 hrs  HW=0.32'   (Free Discharge)
 ├1=Broad-Crested Rectangular Weir  ( Controls 0.00 cfs)
 └2=Orifice/Grate  (Orifice Controls 0.04 cfs @ 1.42 fps)
```

Source: HydroCAD® Summary Report; HydroCAD is a registered trademark of HydroCAD Software Solutions LLC. Used with permission

Revised 10-year Design Storm Post-Construction Condition Summary Report
Exfiltration = 1.1 in/hr and Basin Footprint Remains = 2,700 sf

```
Inflow Area =        10,890 sf,   0.00% Impervious,  Inflow Depth =   4.93"   for  10-Year event
Inflow      =       1.34 cfs @   12.10 hrs,  Volume=            4,474 cf
Outflow     =       0.16 cfs @   12.73 hrs,  Volume=            4,474 cf,  Atten= 88%,  Lag= 37.6 min
Discarded =         0.07 cfs @   10.72 hrs,  Volume=            3,351 cf
Primary     =       0.10 cfs @   12.73 hrs,  Volume=            1,123 cf

Routing by Stor-Ind method, Time Span= 0.00-48.00 hrs, dt= 0.04 hrs
Peak Elev= 0.60' @ 12.73 hrs   Surf.Area= 2,700 sf   Storage= 1,607 cf

Plug-Flow detention time= 98.7 min calculated for 4,471 cf (100% of inflow)
Center-of-Mass det. time= 98.6 min ( 859.2 - 760.6 )

Volume         Invert       Avail.Storage   Storage Description
  #1            0.00'          8,100 cf    Custom Stage Data (Prismatic) Listed below (Recalc)

 Elevation           Surf.Area          Inc.Store          Cum.Store
   (feet)             (sq-ft)        (cubic-feet)       (cubic-feet)
    0.00               2,700                  0                  0
    1.00               2,700              2,700              2,700
    2.00               2,700              2,700              5,400
    3.00               2,700              2,700              8,100

Device   Routing            Invert    Outlet Devices
   #1    Primary             1.00'    20.0' long  x 0.5' breadth Broad-Crested Rectangular Weir
                                      Head (feet)  0.20  0.40  0.60  0.80  1.00
                                      Coef. (English)  2.80  2.92  3.08  3.30  3.32
   #2    Primary             0.15'    2.5" Vert. Orifice/Grate    C= 0.600
   #3    Discarded           0.00'    1.10 in/hr Exfiltration over Surface area

Discarded OutFlow  Max=0.07 cfs @ 10.72 hrs  HW=0.03'   (Free Discharge)
 └3=Exfiltration  (Exfiltration Controls 0.07 cfs)

Primary OutFlow  Max=0.10 cfs @ 12.73 hrs  HW=0.60'   (Free Discharge)
 ├1=Broad-Crested Rectangular Weir  ( Controls 0.00 cfs)
 └2=Orifice/Grate  (Orifice Controls 0.10 cfs @ 2.81 fps)
```

Source: HydroCAD® Summary Report; HydroCAD is a registered trademark of HydroCAD Software Solutions LLC. Used with permission

iv. A summary of design storm peak flow rates for two-, 10- and 100-year design storms are shown below:

Post-construction Peak Flow Rates of Drainage Area Post-A, with the Incorporation of a Green Infrastructure BMP (1.1 in/hr Exfiltration Rate)

Post-construction Drainage Area Designation	**Allowable Design Storm Peak Flow Rate (cfs)**			**Design Storm Peak Flow Rates with a Small-Scale Infiltration Basin (cfs)**		
	2-year	**10-year**	**100-year**	**2-year**	**10-year**	**100-year**
Post-A1 (parking lot/ gravel)	0.04	0.23	0.70	0.04	0.10	0.58
Post-A2 (undisturbed area/ pasture)	0.19	0.78	2.26	0.19	0.78	2.26
Post-A	**0.23**	**1.01**	**2.96**	**0.23**	**0.87**	**2.80**

c. From the revised summary for the 100-year design storm for a 2,700 sf footprint, the exfiltration (discarded) volume is 4,603 cf. The duration of infiltration for 1.1 in/hr exfiltration rate is calculated to be 18.60 hours, as shown below. The 72 hour maximum has not been exceeded, meaning the procedure may be continued.

$$\text{Duration of infiltration period, t }(hr) = \frac{Discarded\ Volume\ via\ Exfiltration\ (cf)\ x\ 12\ in/ft}{Infiltration\ area\ (sf)\ x\ Exfiltration\ rate\ (in/hr)}$$

$$= \frac{4{,}603\ cf\ \ x\ 12\ in/ft}{2{,}700\ sf\ x\ 1.1\ in/hr} = 18.60\text{ hr}$$

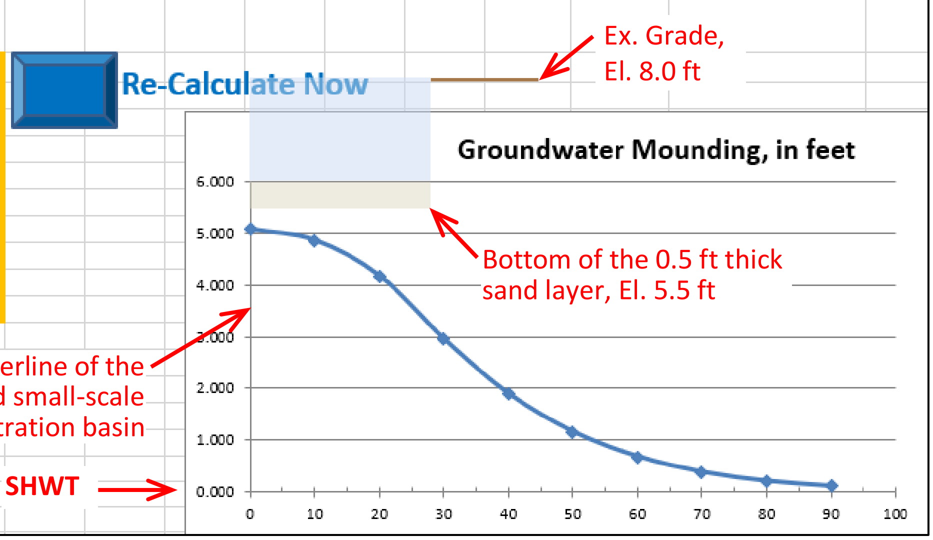
d. The *Hantush Spreadsheet* must be run again.

i. The recharge rate is the exfiltration rate, 1.1 in/hr.

ii. The horizontal conductivity remains as 7.5 in/hr (five times the original design soil permeability rate, 1.5 in/hr).

Input Values		
1.10	R	Recharge rate (permeability rate) (in/hr)
0.150	Sy	Specific yield, Sy (dimensionless) default value is 0.15; max value is 0.2 provided that a lab test data is submitted
7.50	Kh	Horizontal hydraulic conductivity (in/hr) Kh = 5xRecharge Rate (R) in the costal plan; Kh=R outside the coastal plan
26.000	x	1/2 length of basin (x direction, in feet)
26.000	y	1/2 width of basin (y direction, in feet)
18.60	t	Duration of infiltration period (hours)
10.000	hi(0)	Initial thickness of saturated zone (feet)
15.08	h(max)	Maximum thickness of saturated zone (beneath center of basin at end of infiltration period)
5.08	Δh(max)	Maximum groundwater mounding (beneath center of basin at end of infiltration period)

Ground-water Mounding, in feet	Distance from center of basin in x direction, in feet
5.082	0
4.868	10
4.178	20
2.953	30
1.893	40
1.158	50
0.677	60
0.380	70
0.205	80
0.107	90



e. From the results in "Step d," the mounding height is 5.08 feet. The groundwater table is 8 feet below the ground level. The elevated groundwater table will be 2.92 feet below the ground level. The basin sand bottom is 2.5 feet below the ground level, which is above the elevated groundwater table. The elevated groundwater table will not have an adverse impact on the infiltration of the proposed infiltration basin. The trial and error process can therefore end.

Step 5: Since the option to meet the Stormwater Runoff Quantity Standards of N.J.A.C. 7:8-5.6(b)3 was selected in "Step 2," Determine Whether the Post-B Drainage Area Meets the Same Standard

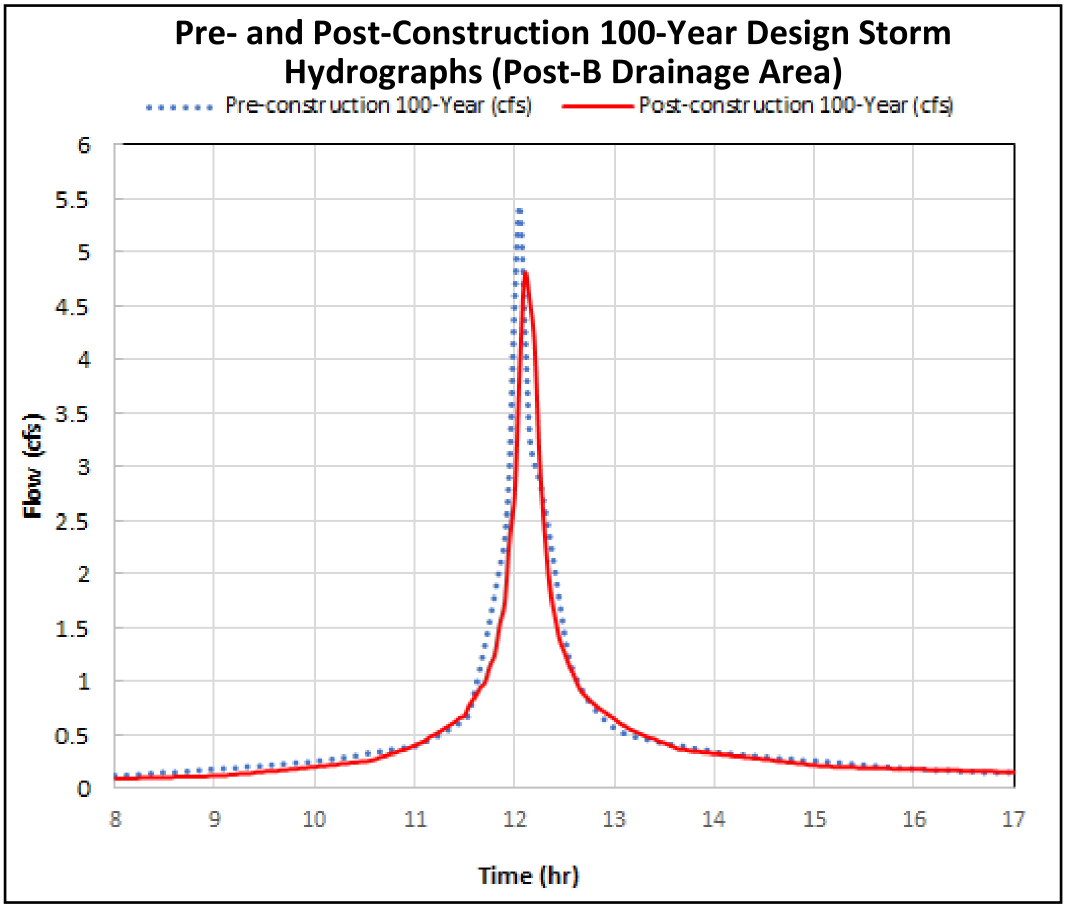
From Page 46, the 2-, 10- and 100-year design storm total peak flow rates from Post-B drainage area are 1.24, 2.41 and 4.74 cfs, respectively. The corresponding allowable design storm peak flow rates for 2-, 10- and 100-year design storms are 0.92, 2.27 and 4.30 cfs. Therefore, Post-B drainage area does not meet the Stormwater Runoff Quantity Requirements of N.J.A.C. 7:8-5.6(b)3.

Step 6: Determine Whether the Post-B Drainage Area Meets the Stormwater Runoff Quantity Standards of N.J.A.C. 7:8-5.6(b)1

As demonstrated above, the Post-B drainage area does not meet the Stormwater Runoff Quantity Control Requirements of N.J.A.C. 7:8-5.6(b)3. A comparison of the pre- and post-construction design storm peak flow rates is made to determine whether Post-B drainage area meets the Stormwater Runoff Quantity Control Requirements of N.J.A.C. 7:8-5.6(b)1.

	Pre-construction Design Storm Flow Rate (cfs)			**Post-construction Design Storm Flow Rate (cfs)**		
Drainage Area Name	**2-year**	**10-year**	**100-**	**2-year**	**10-year**	**100-year**
Post-B1 (building or walkway)	1.76	2.81	4.80	0.57	0.90	1.54
Post-B2 (open space/ grass > 75% and woods)	0.17	0.45	1.07	0.90	1.91	3.93
Total Post-B (Hydrograph addition)	**1.83**	**3.03**	**5.37**	**1.24**	**2.41**	**4.74**

The comparisons between the pre- and post-construction design storm peak flow rates show that the peak flow rates of the post-construction condition are less than the peak flow rates of the pre-construction. However, N.J.A.C. 7:8-5.6(b)1 does not require a comparison of the pre- and post-construction condition peak flow rates. This standard requires the demonstration that the post-construction runoff hydrographs for the 2-, 10- and 100-year design storm events do not exceed, *at any point in time*, the pre-construction runoff hydrographs for the same storm events. Therefore, from the definition of a hydrograph on Page 2, this standard means that the flow rates of the post-construction hydrograph at every point in time must be less than the flow rates of the pre-construction hydrograph for the same instance of time. A comparison of the 100-year design storm hydrographs for the pre-construction and post-construction conditions for the Post-B drainage area is shown below.



A detailed look at the flow rates generated between 12 and 13 hours is provided in a table found on the next page. Note that the flow rates in the table are from a combined hydrograph of drainage area B that is obtained by superposing runoff hydrographs of subdrainage areas B1 and B2 at the same time scale. Therefore, the peak flow rate may be different from the arithmetic sum of the peak flow rates of subdrainage areas B1 and B2.

In the table at the top of the next page, yellow shaded cells denote a time increment at which the Post-Construction Peak Flow Rate exceeds the Pre-Construction Peak Flow Rate. The hydrographs shown above, as well as the information in the aforementioned table, show that the post-construction 100-year design storm flow rate exceeds the pre-construction 100-year design storm flow rate from 12.15 to 12.20 hours and from 12.75 to 13 hours. Therefore, drainage area Post-B has failed to demonstrate compliance with N.J.A.C. 7:8-5.6(b)1.

Time (hours)	Pre-construction flow (cfs)	Post-construction flow (cfs)	Difference (Post-Pre) (cfs)	Time (hours)	Pre-construction flow (cfs)	Post-construction flow (cfs)	Difference (Post-Pre) (cfs)
12.00	4.57	2.79	-1.78	12.55	1.19	1.13	-0.06
12.05	5.44	3.94	-1.5	12.60	1.07	1.04	-0.03
12.10	4.35	4.78	0.43	12.65	0.97	0.93	-0.04
12.15	3.28	4.54	1.26	12.70	0.88	0.87	-0.01
12.20	3.02	4.12	1.1	12.75	0.8	0.82	0.02
12.25	2.87	3.06	0.19	12.80	0.74	0.78	0.04
12.30	2.63	2.39	-0.24	12.85	0.68	0.74	0.06
12.35	2.34	1.89	-0.45	12.90	0.64	0.71	0.07
12.40	2.03	1.6	-0.43	12.95	0.6	0.68	0.08
12.45	1.73	1.38	-0.35	13.00	0.56	0.65	0.09
12.50	1.43	1.26	-0.17				

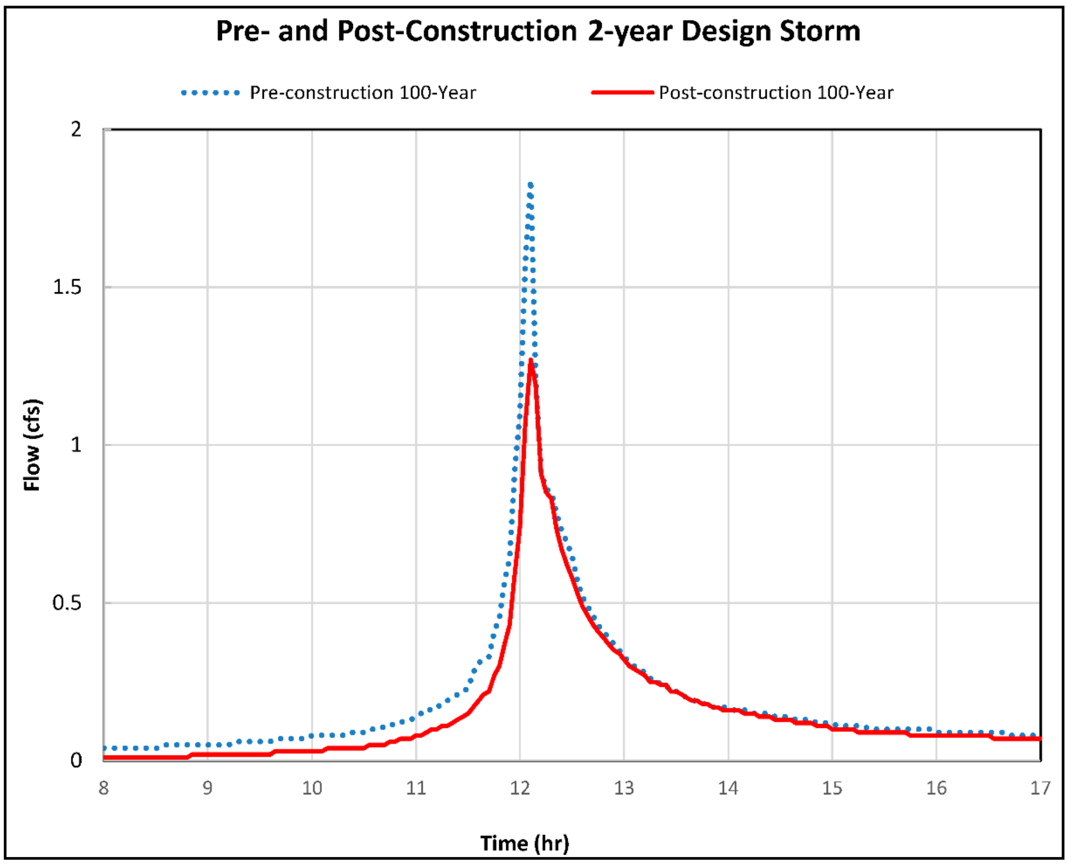
Step 7: Since Post-B Drainage Area Failed to Meet the Stormwater Runoff Quantity Control Standards of N.J.A.C. 7:8-5.6(b), on its own, Determine Whether the Site as a Whole Meets the Stormwater Runoff Quantity Control Standards of N.J.A.C. 7:8-5.6(b)

The question here is whether stormwater runoff from drainage areas Post-A and Post-B can be combined together to meet the Stormwater Runoff Quantity Control requirements either under N.J.A.C. 7:8-5.6(b)3 or N.J.A.C. 7:8-5.6(b)1. The table below illustrates combining post-construction flow rates from drainage areas A and B for each of the regulatory design storms.

Post-construction Drainage Area Designation	Allowable Design Storm Peak Flow Rate (cfs) under N.J.A.C. 7:8-5.6(b)3			Design Storm Peak Flow Rate with a Small-Scale Infiltration Basin in drainage area Post-A		
	2-year	10-year	100-year	2-year	10-year	100-year
Post-A	0.23	1.01	2.96	0.23	0.88	2.81
Post-B	0.92	2.27	4.30	1.33	2.56	4.05
Post A and B Combined	**1.15**	**3.28**	**7.26**	**1.28**	**2.79**	**5.95**

The combined post-construction 10- and 100-year storm peak flow rates for the whole site are 2.79 and 5.95 cfs, respectively, which are less than the allowable 10- and 100-year design storm peak flow rates, 3.28 cfs and 7.26 cfs, respectively. However, the combined post-construction 2-year peak flow rate 1.28 cfs is greater than the allowable 2-year design storm peak flow rate of 1.15 cfs.

If the hydrographs of the 2-year storm for the pre- and post-construction conditions are compared, it seems that the stormwater runoff quantity control requirements, when analyzing the site as a whole, for the 2-year design storm have been met under N.J.A.C. 7:8-5.6(b)1 since the post-construction hydrograph for the entire site exceed, some points in time, the pre-construction hydrograph.



At this point, **it would be incorrect** to state that areas Post-A and Post-B – combined - meet the Stormwater Runoff Quantity Control Standards of N.J.A.C. 7:8-5.6(b)1 for the 2-year design storm and those of N.J.A.C. 7:8-5.6(b)3 for the 10- and 100-year design storms. N.J.A.C. 7:8-5.2(l) requires

> *If there is more than one drainage area, the groundwater recharge, stormwater runoff quality, and stormwater runoff quantity standards at N.J.A.C. 7:8-5.4, 5.5, and 5.6 shall be met in each drainage area, unless the runoff from the drainage areas converge onsite and no adverse environmental impact would occur as a result of compliance with any one or more of the individual standards being determined utilizing a weighted average of the results achieved for that individual standard across the affected drainage areas.*

Specifically, N.J.A.C. 7:8-5.6(b) and N.J.A.C. 7:8-5.6(c) both require the standards be applied to the stormwater leaving the site or at the boundary to each abutting lot, roadway, watercourse or receiving storm sewer system. Recall that the runoff from drainage area Post-B discharges at a point on one side of the property and drains to a municipal storm sewer system, while the runoff from drainage area Post-A is discharged on the other side of the property and drains to the riparian zone of a creek. The flows do not converge into one point of discharge before leaving the site boundaries. Therefore, the hydrographs for drainage area Post-A cannot be combined with the hydrographs of drainage area Post-B, and in other words, the above combined hydrograph or combined flow rates cannot be used to demonstrate the compliance with the requirements under N.J.A.C. 7:8-5.6(b)1. Furthermore, the requirements under N.J.A.C. 7:8-5.6(b)1, 2 and 3 are three separate options that cannot be mixed. In each option, all three design storms are stated, which means **one cannot choose to use one of the options from N.J.A.C. 7:8-5.6(b) for a single design storm and pick another option for a different design storm** and so forth when in the same drainage area.

Therefore, in this example, the stormwater management design for drainage area Post-B alone will need to demonstrate compliance with either N.J.A.C. 7:8-5.6(b)1, 2 or 3. Moreover, the required demonstration of N.J.A.C. 7:8-5.6(b)2 is an analysis of the whole watershed, which will be a more difficult task than the demonstration of the requirement specified in N.J.A.C. 7:8-5.6(b)1 or 3.

In conclusion, the Post-B drainage area, as shown above, has failed to meet the requirements under N.J.A.C. 7:8-5.6(b)3. In the next step, green infrastructure BMPs will be evaluated to provide the required stormwater runoff quantity controls for the Post-B drainage area and to bring this portion of the site into compliance with the design and performance standards.

Step 8: Design Small-Scale Bioretention Basins to Address Roof Runoff from Drainage Area Post-B

Two small-scale bioretention basins are proposed to provide stormwater runoff quantity controls for the Post-B1 drainage area. Each of the basins has a footprint measuring 550 sf in area. A 2.5 in orifice elevated 2 ft above the basin bottom is used as the outlet structure for each of the proposed small scale-bioretention basins during the 2-, 10- and 100-year design storms. Soil permeability tests were conducted at the most restrictive soil layer within the proposed small-scale bioretention basins. The tested soil permeability rate is 1 in/hr. The design permeability rate, equal to one-half of the tested permeability rate, is 0.5 in/hr, and it is used as the exfiltration rate in the 2-, 10- and 100- year design storm routings. The summary report from the modeling software is shown on the next page for the 100-year design storm.

100-year Design Storm Summary Report

```
Inflow Area =        3,485 sf,100.00% Impervious,  Inflow Depth =  8.96"   for  100-Year event
Inflow      =     0.77 cfs @  12.07 hrs,  Volume=          2,602 cf
Outflow     =     0.17 cfs @  12.31 hrs,  Volume=          2,602 cf,  Atten= 77%,  Lag= 14.4 min
Discarded =       0.01 cfs @   2.90 hrs,  Volume=            967 cf
Primary     =     0.17 cfs @  12.31 hrs,  Volume=          1,635 cf

Routing by Stor-Ind method, Time Span= 0.00-48.00 hrs, dt= 0.05 hrs
Peak Elev= 1.96' @ 12.31 hrs   Surf.Area= 550 sf   Storage= 1,078 cf

Plug-Flow detention time= 295.7 min calculated for 2,602 cf (100% of inflow)
Center-of-Mass det. time= 295.6 min ( 1,031.8 - 736.2 )

Volume      Invert     Avail.Storage   Storage Description
  #1         0.00'         2,200 cf    Custom Stage Data (Prismatic) Listed below

 Elevation          Surf.Area        Inc.Store       Cum.Store
    (feet)            (sq-ft)       (cubic-feet)    (cubic-feet)
      0.00               550               0               0
      1.00               550             550             550
      2.00               550             550           1,100
      3.00               550             550           1,650
      4.00               550             550           2,200

Device   Routing          Invert   Outlet Devices
  #1     Discarded         0.00'   0.50 in/hr Exfiltration over Surface area below 2.00'
  #2     Primary           0.80'   2.5" Vert. Orifice/Grate    C= 0.600

Discarded OutFlow  Max=0.01 cfs @ 2.90 hrs  HW=0.04'   (Free Discharge)
└─1=Exfiltration  (Exfiltration Controls 0.01 cfs)

Primary OutFlow  Max=0.17 cfs @ 12.31 hrs  HW=1.96'   (Free Discharge)
└─2=Orifice/Grate  (Orifice Controls 0.17 cfs @ 4.94 fps)
```

Source: HydroCAD® Summary Report; HydroCAD is a registered trademark of HydroCAD Software Solutions LLC. Used with permission

The peak flow rates resulting from the proposed design are as follows:

Post-construction Drainage Area Designation	**Allowable Design Storm Peak Flow Rate (cfs)**			**Design Storm Peak Flow Rates with a Small-Scale Bioretention Basin (cfs)**		
	2-year	**10-year**	**100-year**	**2-year**	**10-year**	**100-year**
Post-B1 (building/walkway) after small-scale bioretention basins	0.88	2.11	3.84	0.1	0.09	0.17
Post-B2 (open space, grass > 75% and woods)	0.09	0.34	0.86	0.90	1.91	3.93
Post-B	**0.92**	**2.27**	**4.30**	**0.90**	**1.91**	**4.08**

Note that the peak flow rate from each small-scale bioretention basin is 0.1 cfs for the 100-year design storms. Since there are two small-scale bioretention basins in the Post-B1 drainage area, the total post-construction design peak flow rates are twice the peak flow rates from each small-scale bioretention basin. For Drainage area Post-B, the total peak flow rates are 0.90, 1.91 and 4.08 cfs for the 2-, 10- and 100-year design storms, respectively, for which each of the corresponding design storm values are less than the allowable design storm peak flow rates, i.e., 0.92, 2.27 and 4.30 cfs. By constructing the two small-scale bioretention basins (rain gardens), the stormwater runoff peak flow rates from drainage area Post-B meet the design standard under N.J.A.C. 7:8-5.6(b)3.

Step 9: Groundwater Mounding Analysis

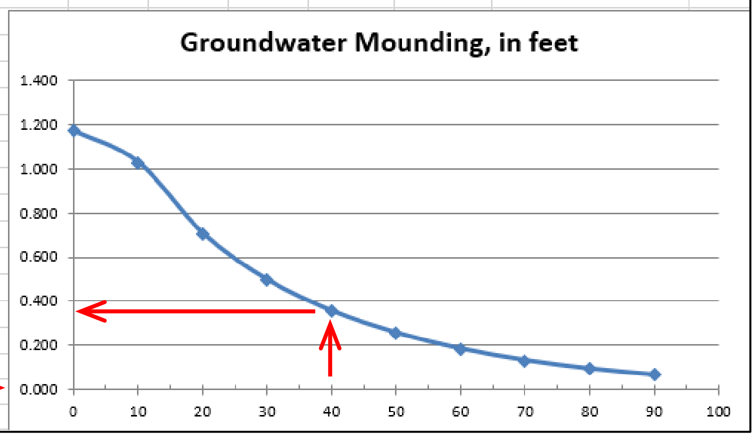
NJDEP's *Hantush Spreadsheet* is used for the groundwater mounding analysis. Assume each small-scale bioretention basins is 25 ft by 22 ft. The recharge rate, R, is the design permeability rate, 0.5 in/hr. The parameters for specific yield, S_y, and the horizontal hydraulic conductivity, K_h, are set at the default values specified in *Chapter 13*. The horizontal hydraulic conductivity rate is 5 times the recharge rate since the site, located in Ocean County, is in the coastal plain. The x and y values are equal to half of the respective basin dimensions.

The duration of infiltration period of each basin during the 100-year storm is calculated by using the exfiltration (discarded) volume, 967 cf, the footprint of the basin, 550 sf, and the exfiltration rate, 0.5 in/hr. The result is 42.20 hr.

The results, depicted on the following page, show that the maximum height of the groundwater mounding, $\Delta h(max)$, is 1.17 ft. Since the groundwater table is 8 ft below the existing ground elevation, the elevated groundwater table will be 6.83 ft below the existing ground level. The proposed small-scale bioretention basin has a 100-year storm outlet structure at 1 ft. The basin also has 1 ft of freeboard and a soil bed 1.5 ft in depth. Therefore, the lowest point of the proposed basin is 3.5 ft below the existing ground elevation. Since the elevated groundwater table will be 6.83 feet below the existing ground elevation, the elevated groundwater level will not have adverse impact on the drainage of the basin.

Input Values		
0.50	*R*	Recharge rate (permeability rate) (in/hr)
0.150	Sy	Specific yield, Sy (dimensionless) default value is 0.15; max value is 0.2 provided that a lab test data is submitted
7.50	Kh	Horizontal hydraulic conductivity (in/hr) Kh = 5xRecharge Rate (R) in the costal plan; Kh=R outside the coastal plan
12.500	x	1/2 length of basin (x direction, in feet)
11.000	y	1/2 width of basin (y direction, in feet)
46.20	t	Duration of infiltration period (hours)
10.000	hi(0)	Initial thickness of saturated zone (feet)
11.17	h(max)	Maximum thickness of saturated zone (beneath center of basin at end of infiltration period)
1.17	Δh(max)	Maximum groundwater mounding (beneath center of basin at end of infiltration period)
Ground-water Mounding, in feet	Distance from center of basin in x direction, in feet	
1.171	0	Re-Calculate Now
1.028	10	
0.706	20	
0.495	30	
0.355	40	
0.255	50	
0.183	60	
0.130	70	
0.092	80	
0.065	90	

SHWT



The groundwater mounding curve shows that within 40 ft from the center of the proposed small-scale bioretention (27.5 ft from the edge of the basin), the groundwater level will be elevated by approximately 0.36 ft, or roughly 7.64 ft below the existing ground elevation when the mounding under the center of the basin reaches maximum height. Note that the curve shows the mounding height at the time when it is at its maximum in the center of the basin. Mounding height away from the center of the basin may become higher as the mound decays and the infiltrated runoff spreads out from the center to the mound to the tails of the mound. If there is a basement within 40 ft of one of the small-scale bioretention basins and the slab of the basement is 8 ft below the existing ground elevation, the basement may sometimes experience inundation by the temporary increase in groundwater level during the 100-year storm. Therefore, the small-scale bioretention basins may need to be located away from the building to consider the possibility that the lowest point of the basement may sometimes be below the elevated groundwater table. Note that the mounding height away from the center of the basin may become higher as the mounding dissipates and the infiltrated runoff spreads out from the center of the mound to the farthest extents of the mounding.

Example 5-7:

The proposed development is the same as that of Example 5-6, but a swale is proposed to run through drainage area B and directs the runoff from drainage area B to the discharge point of drainage area A. Determine whether the proposed development meets the stormwater runoff quantity standards under N.J.A.C. 7:8-5.6(b).

Step 1: Design the Grass Swale

Grass swales are one of the green infrastructure BMPs listed in Table 5-1 of N.J.A.C. 7:8-5.2, but as stated in this table, they cannot be used to meet the groundwater recharge or stormwater runoff quantity requirements of N.J.A.C. 7:8-5.4 and 5.6. However, grass swales can be designed to provide conveyance of stormwater runoff from one drainage area to another drainage area. A grass swale also can be designed to convey flows at a lower flow velocity, which results in a slower time of concentration, than that of a stormwater drainage pipe, which typically has a smooth surface.

Assuming the grass swale has a bottom width of 6 ft, a depth of 0.5 ft, a slope of 0.46%, a length of 650 ft and a Manning's roughness coefficient 0.03, the grass swale will be able to convey the peak flow of stormwater runoff produced by the 100-year design storm with a runoff depth of 0.36 ft and a maximum flow velocity of 1.51 fps, which is under the maximum allowable velocity of 2.5 fps, for sandy clay loam soil texture as suggested in the *Standards for Soil Erosion and Sediment Control in New Jersey*.

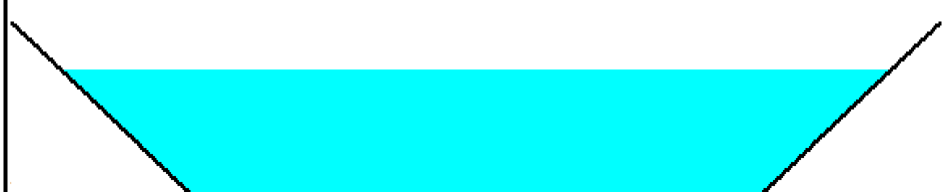
Grass Swale Summary Report for the 100-year Design Storm

Inflow Area = 32,670 sf, 21.33% Impervious, Inflow Depth = 7.13" for 100-Year event
Inflow = 4.74 cfs @ 12.12 hrs, Volume= 19,412 cf
Outflow = 4.05 cfs @ 12.32 hrs, Volume= 19,412 cf, Atten= 14%, Lag= 11.9 min

Routing by Stor-Ind+Trans method, Time Span= 0.00-48.00 hrs, dt= 0.05 hrs
Max. Velocity= 1.51 fps, Min. Travel Time= 7.2 min
Avg. Velocity = 0.32 fps, Avg. Travel Time= 34.1 min

Peak Storage= 1,764 cf @ 12.20 hrs
Average Depth at Peak Storage= 0.36'
Bank-Full Depth= 0.50' Flow Area= 4.0 sf, Capacity= 7.25 cfs

6.00' x 0.50' deep channel, n= 0.030 Earth, grassed & winding
Side Slope Z-value= 4.0 '/' Top Width= 10.00'
Length= 650.0' Slope= 0.0046 '/'
Inlet Invert= 0.00', Outlet Invert= -3.00'



Source: HydroCAD® Summary Report; HydroCAD is a registered trademark of HydroCAD Software Solutions LLC. Used with permission

Grass Swale Summary Report for the 10-year Design Storm

Inflow Area = 32,670 sf, 21.33% Impervious, Inflow Depth = 3.58" for 10-Year event
Inflow = 2.41 cfs @ 12.11 hrs, Volume= 9,743 cf
Outflow = 1.98 cfs @ 12.36 hrs, Volume= 9,743 cf, Atten= 18%, Lag= 14.8 min

Routing by Stor-Ind+Trans method, Time Span= 0.00-48.00 hrs, dt= 0.05 hrs
Max. Velocity= 1.19 fps, Min. Travel Time= 9.1 min
Avg. Velocity = 0.25 fps, Avg. Travel Time= 43.0 min

Peak Storage= 1,085 cf @ 12.21 hrs
Average Depth at Peak Storage= 0.24'
Bank-Full Depth= 0.50' Flow Area= 4.0 sf, Capacity= 7.25 cfs

6.00' x 0.50' deep channel, n= 0.030 Earth, grassed & winding
Side Slope Z-value= 4.0 '/' Top Width= 10.00'
Length= 650.0' Slope= 0.0046 '/'
Inlet Invert= 0.00', Outlet Invert= -3.00'

Source: HydroCAD® Summary Report; HydroCAD is a registered trademark of HydroCAD Software Solutions LLC. Used with permission

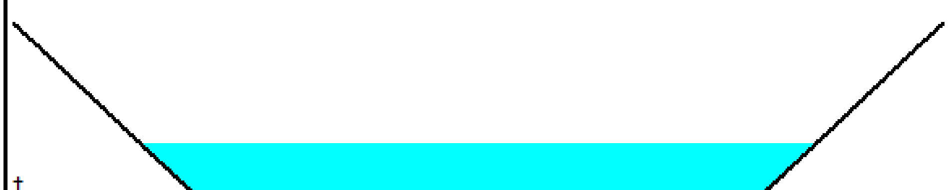
Grass Swale Summary Report for the 2-year Design Storm

Inflow Area = 32,670 sf, 21.33% Impervious, Inflow Depth = 1.85" for 2-Year event
Inflow = 1.24 cfs @ 12.11 hrs, Volume= 5,028 cf
Outflow = 0.94 cfs @ 12.42 hrs, Volume= 5,028 cf, Atten= 25%, Lag= 18.8 min

Routing by Stor-Ind+Trans method, Time Span= 0.00-48.00 hrs, dt= 0.05 hrs
Max. Velocity= 0.91 fps, Min. Travel Time= 11.9 min
Avg. Velocity = 0.21 fps, Avg. Travel Time= 52.7 min

Peak Storage= 669 cf @ 12.22 hrs
Average Depth at Peak Storage= 0.16'
Bank-Full Depth= 0.50' Flow Area= 4.0 sf, Capacity= 7.25 cfs

6.00' x 0.50' deep channel, n= 0.030 Earth, grassed & winding
Side Slope Z-value= 4.0 '/' Top Width= 10.00'
Length= 650.0' Slope= 0.0046 '/'
Inlet Invert= 0.00', Outlet Invert= -3.00'



Source: HydroCAD® Summary Report; HydroCAD is a registered trademark of HydroCAD Software Solutions LLC. Used with permission

To summarize the reports listed above, the stormwater runoff peak flow rates for the 2-, 10- and 100-year design storms, generated by the Post-B drainage area and conveyed by the grass swale to the discharge point of convergence in Post-A drainage area are listed in the table below.

Post-construction Drainage Area Designation	Design Storm Peak Flow Rates with a Grass Swale (cfs)		
	2-year	10-year	100-year
Post-B	0.94	1.98	4.05

Step 2: Address Groundwater Recharge

Regarding groundwater recharge, both drainage area Post-B and drainage area Pre-B contain 0.5 acres of impervious surface, meaning there is no change in impervious cover area from the pre-development condition to the post-development condition. However, the project also proposes to change the land cover from woods to grass lawn, which will reduce the amount of groundwater recharge provided. Therefore, an evaluation of the groundwater recharge deficit is needed. See *Chapter 6* of this manual for guidance on performing a groundwater recharge analysis.

Step 3: Address Stormwater Runoff Quantity Control

Although not mentioned in the initial description, the stormwater runoff generated by the proposed building and walkway will flow as overland flow across the proposed lawn area. The lawn area is graded to direct this runoff to the same grass swale and discharge point for the Post-A drainage area. The stormwater runoff generated by both the roof and the non-vehicular walkway do not require water quality treatment. Drainage area Post-A1 will still require a small-scale infiltration basin to provide the required stormwater runoff quantity and quality controls, in addition to the groundwater recharge requirement for the stormwater runoff generated by the proposed gravel parking lot. Drainage area Post-A2 will not be disturbed and is therefore not subject to the stormwater runoff quality and quantity requirements, nor those for groundwater recharge.

In this example, the initial description states that the stormwater runoff from Drainage Areas A and B converge into one discharge point (i.e., the point of analysis A) before leaving the site. Therefore, the peak flow rates for 2-, 10- and 100-year design storms of the two drainage areas can be added to calculate the peak flow rates at the discharge location.

Post-construction Drainage Area Designation	Allowable Design Storm Peak Flow Rate (cfs) per the N.J.A.C. 7:8-5.6(b)3 Standard			Design Storm Peak Flow Rate (cfs)		
	2-year	**10-year**	**100-year**	**2-year**	**10-year**	**100-year**
Post-A	0.23	1.01	2.96	0.23	0.84	2.56
Post-B	0.92	2.27	4.30	0.94	1.98	4.05
Post A and B Combined	**1.15**	**3.28**	**7.26**	**1.17**	**2.85**	**6.89**

As the results show in the table above, each of the combined post-construction peak flow rates for the 2-, 10- and 100-year design storms are less than the respective allowable design storm peak flow rates. Therefore, the site as a whole meets the stormwater runoff quantity standards of N.J.A.C. 7:8-5.6(b)3.

Rational Method

The Rational method is a tool for estimating peak runoff flowrate from small drainage area not greater than 20 acres. The equation for the rational method is expressed below:

$$Q_p = \mathrm{C}\ i\ \mathrm{A}$$

where:

Q_p = peak flow rate (cfs)

C = runoff coefficient (dimensionless), which describes the level of imperviousness of the drainage area and reflect the ability of a soil to infiltrate precipitation before excess precipitation becomes stormwater runoff and sometimes the slope of the drainage area

i = uniform rate of rainfall intensity for a rainfall duration longer or equal to the time of concentration T_c (in/hr)

A = drainage area (ac)

Note that the rational equation may not appear to be dimensionally correct in that although i is specified in inches per hour, 1 in/hr equals 1.008 cfs per acre, and in using this equation, the two are taken to be numerically equal, meaning no conversion factor is required.

Runoff Coefficients

Pervious drainage areas typically have low values for their respective runoff coefficients, and impervious areas have runoff coefficients much closer to a value of 1. Unlike Curve Numbers published by NRCS, runoff coefficients can be obtained from many field studies or authoritative agencies. One source of a comprehensive table for runoff coefficients is the runoff coefficient table in *Design and Construction of Sanitary and Storm Sewers*, Manuals and Reports on Engineering Practice No. 37, American Society of Civil Engineers (ASCE), 1969. Other authorities such as transportation departments or environmental departments of other states may also publish their reference tables of runoff coefficients. However, some of these publications do not consider the HSG and/or land cover, which are two important factors affecting the amount of infiltration and retention of precipitation before excessive precipitation becomes runoff. Therefore, it is recommended that designers use the runoff coefficients listed on the following page in Table 5-3.

Table 5-3: Runoff Coefficients for the Rational Method

Land Use	Description	Hydrologic Soils Group			
		A	B	C	D
Cultivated Land	without conservation treatment	0.49	0.67	0.81	0.88
	with conservation treatment	0.27	0.43	0.67	0.67
Pasture or Range Land	poor condition	0.38	0.63	0.78	0.84
Meadow	good condition	---	0.25	0.51	0.65
	good condition	---	---	0.41	0.61
Wood or Forest Land	thin stand, poor cover, no mulch	---	0.34	0.59	0.70
	good cover	---	---	0.45	0.59
Open Spaces, Lawns, Parks, Golf Courses, Cemeteries					
Good Condition	grass cover on 75% or more	---	0.25	0.51	0.65
Fair Condition	grass cover on 50% to 75%	---	0.45	0.63	0.74
Commercial and Business Area	85% impervious	0.84	0.90	0.93	0.96
Industrial Districts	72% impervious	0.67	0.81	0.88	0.92
Residential Average Lot Size (acres)	average % impervious				
1/8	65	0.59	0.76	0.86	0.90
1/4	38	0.29	0.55	0.70	0.80
1/3	30	---	0.49	0.67	0.78
1/2	25	---	0.45	0.65	0.76
1	20	---	0.41	0.63	0.74
Paved Areas	parking lots, roofs, driveways, etc.	0.99	0.99	0.99	0.99
Streets and Roads	paved with curbs & storm sewers	0.99	0.99	0.99	0.99
	gravel	0.57	0.76	0.84	0.88
	dirt	0.49	0.69	0.80	0.84

Source: New Jersey Department of Environmental Protection, Land Use Management Program. 1988. Technical Manual for Land Use Regulation Program, Bureau of Inland and Coastal Regulations, NJDEP Flood Hazard Area Permits.

Drainage Area Size and Land Cover Limitations

As previously mentioned on Page 9, there are some basic assumptions when applying the Rational Method. The maximum size limit for a drainage area is 20 ac. The drainage area must also have homogeneous land cover and topography, meaning the same runoff coefficient and slope apply to the entire drainage area. Furthermore, the rainfall distribution on a drainage area is assumed to be uniform over the entire drainage area.

Sites with Pervious and Directly Connected Impervious Cover

Due to the linear character of the Rational Method equation, a representative Rational Runoff Coefficient (C) can be computed for the entire site by standard area weighting techniques.

Sites with Unconnected Impervious Cover

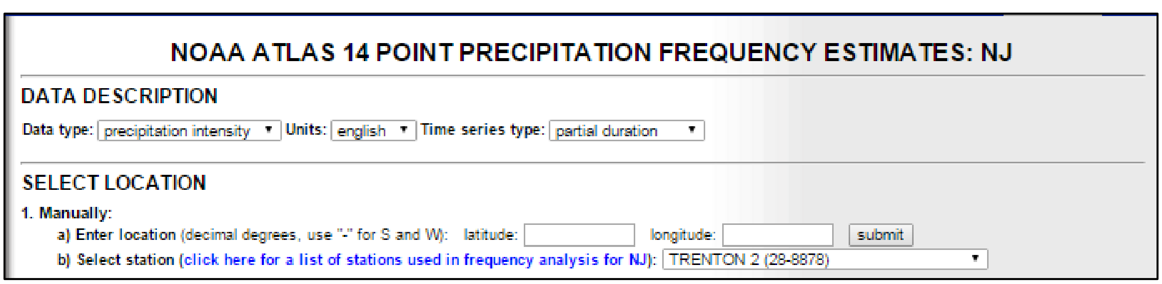
Due to the basic nature of the Rational Method equation, there is currently no technique for addressing the effects of unconnected impervious cover. As such, neither the Rational nor Modified Rational Methods can be recommended at this time for use at sites with unconnected impervious areas. Calculations should be completed using the NRCS method, as previously discussed.

Time of Concentration

Although time of concentration is not an input factor in the equation, as mentioned above, the rainfall intensity in the equation is related to the choice of the time of concentration. Therefore, the time of concentration must be calculated using Manning's equation for sheet flow, based on the limitations for the maximum length of sheet flow established on Page 23. Equation 15-8 in *NEH, Part 630, Chapter 15* can be used for estimating the travel time. When calculating peak flow rates using the Rational Method, the minimum time of concentration is 10 minutes.

Rainfall Intensity for Stormwater Runoff Quantity Calculation

The rainfall intensity used in the equation is the uniform rate of rainfall intensity for durations equal to the time of concentration. In the Rational Method, the intensity of a specific duration for a particular recurrence year of storm can also be obtained from NOAA's Precipitation Frequency (PF) Estimates. On the NOAA Precipitation Frequency Data Server, the Data Type to be selected is precipitation intensity and the Time Series Type is partial duration, as shown in in the image below.



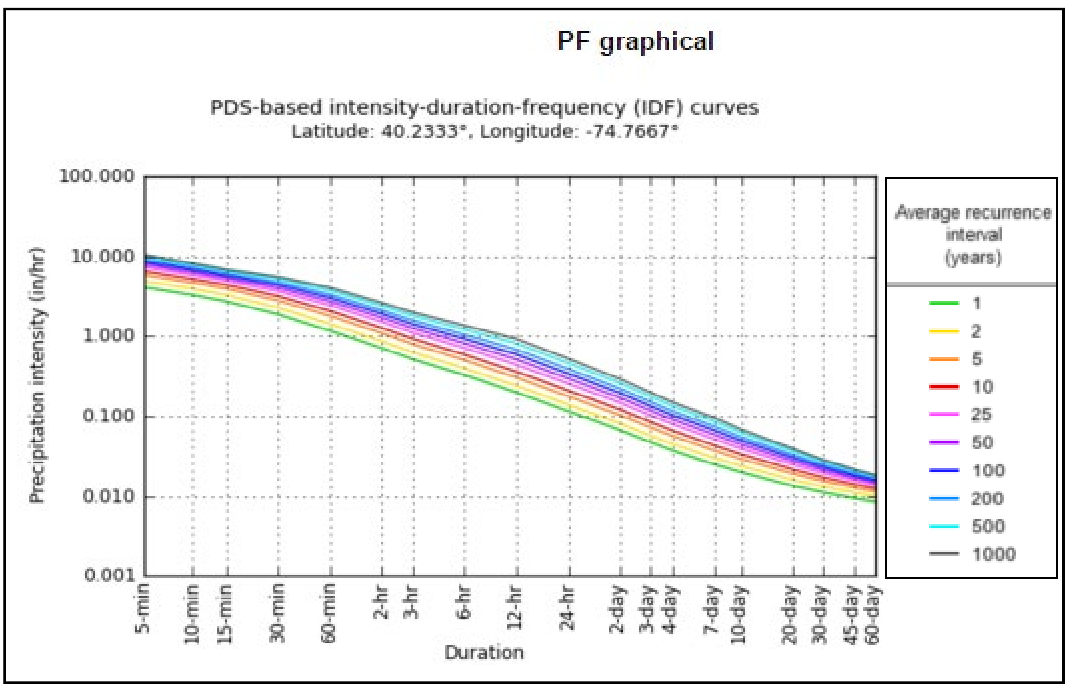
After selecting the location, the Precipitation Frequency Estimates provide a table of the intensity of precipitation in respect to the average recurrence interval of the storm and the duration. The duration referenced in the table is equal to the time of concentration. For example, if the time of concentration is 15 minutes, the intensities for the 2-, 10- and 100 year design storms located in the PF tabular table correspond to the values found in the row for a duration of 15-min, which are outlined in red in the table on the following page and read 3.28, 4.38 and 5.72 in/hr, respectively.

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches/hour)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	4.10 (3.73-4.52)	4.90 (4.45-5.40)	5.83 (5.28-6.42)	6.50 (5.88-7.15)	7.33 (6.59-8.06)	7.94 (7.10-8.75)	8.54 (7.61-9.43)	9.11 (8.06-10.1)	9.83 (8.60-10.9)	10.4 (9.01-11.6)
10-min	3.28 (2.98-3.61)	3.92 (3.56-4.32)	4.67 (4.22-5.14)	5.20 (4.70-5.72)	5.84 (5.25-6.43)	6.32 (5.66-6.97)	6.79 (6.05-7.49)	7.22 (6.40-8.00)	7.77 (6.81-8.65)	8.17 (7.09-9.16)
15-min	2.73 (2.48-3.01)	3.28 (2.99-3.62)	3.94 (3.56-4.33)	4.38 (3.96-4.82)	4.94 (4.44-5.43)	5.34 (4.78-5.88)	5.72 (5.10-6.32)	6.08 (5.38-6.73)	6.52 (5.71-7.26)	6.84 (5.94-7.66)
30-min	1.87 (1.70-2.06)	2.27 (2.06-2.50)	2.80 (2.53-3.08)	3.17 (2.87-3.49)	3.66 (3.29-4.02)	4.02 (3.60-4.43)	4.38 (3.90-4.84)	4.73 (4.19-5.24)	5.19 (4.55-5.78)	5.53 (4.81-6.20)
60-min	1.17 (1.06-1.29)	1.42 (1.29-1.57)	1.79 (1.62-1.97)	2.07 (1.87-2.27)	2.44 (2.19-2.68)	2.72 (2.44-3.00)	3.02 (2.69-3.33)	3.32 (2.94-3.67)	3.72 (3.26-4.14)	4.04 (3.51-4.53)
2-hr	0.706 (0.640-0.780)	0.860 (0.781-0.950)	1.09 (0.986-1.20)	1.26 (1.14-1.39)	1.51 (1.35-1.66)	1.70 (1.52-1.87)	1.90 (1.68-2.10)	2.10 (1.85-2.33)	2.39 (2.08-2.66)	2.62 (2.26-2.93)
3-hr	0.516 (0.467-0.573)	0.629 (0.569-0.699)	0.797 (0.719-0.885)	0.928 (0.834-1.03)	1.11 (0.994-1.23)	1.26 (1.12-1.40)	1.42 (1.25-1.57)	1.58 (1.38-1.76)	1.80 (1.55-2.02)	1.99 (1.69-2.24)
6-hr	0.327 (0.296-0.365)	0.398 (0.359-0.443)	0.502 (0.452-0.559)	0.588 (0.527-0.652)	0.711 (0.631-0.789)	0.814 (0.718-0.903)	0.925 (0.807-1.03)	1.04 (0.901-1.16)	1.22 (1.03-1.36)	1.36 (1.14-1.53)
12-hr	0.197 (0.179-0.221)	0.239 (0.216-0.268)	0.304 (0.274-0.340)	0.359 (0.322-0.401)	0.441 (0.392-0.492)	0.513 (0.451-0.572)	0.591 (0.513-0.659)	0.678 (0.579-0.760)	0.808 (0.676-0.912)	0.920 (0.755-1.04)
24-hr	0.115 (0.106-0.125)	0.139 (0.128-0.151)	0.176 (0.163-0.191)	0.208 (0.191-0.226)	0.255 (0.233-0.276)	0.295 (0.267-0.319)	0.340 (0.305-0.367)	0.388 (0.345-0.420)	0.461 (0.403-0.500)	0.522 (0.450-0.569)

A graphical representation of the intensity (Intensity-duration-frequency, IDF, curves) is shown below:

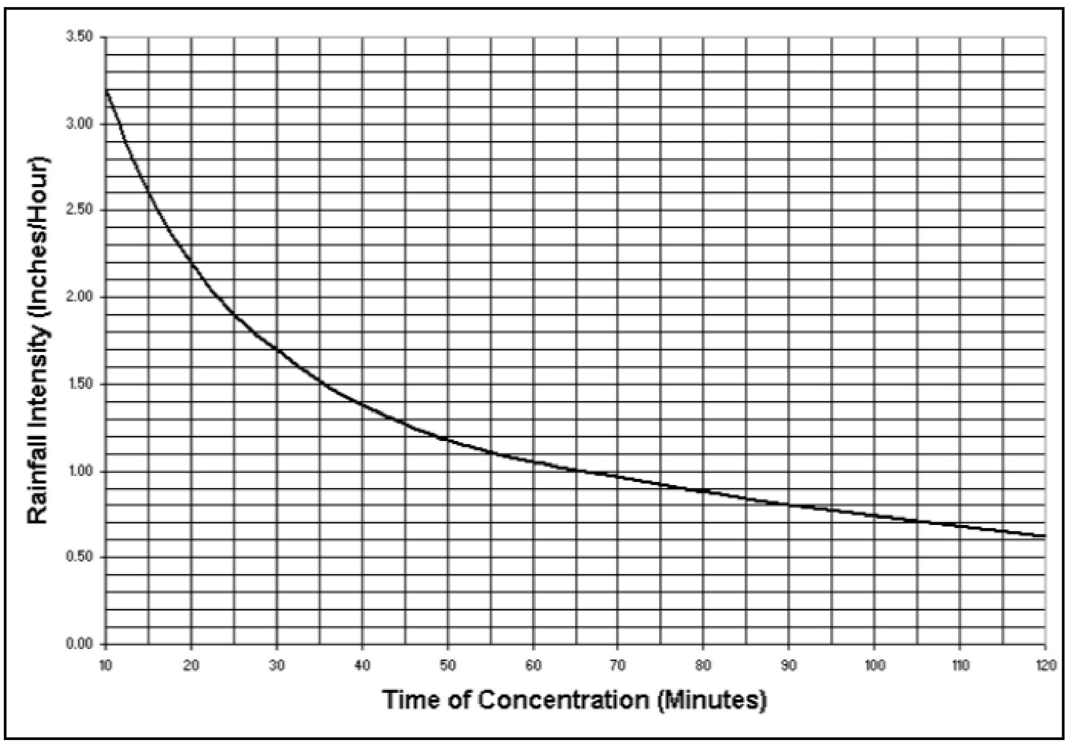


Water Quality Design Storm

The Rational Method calculates only the peak flow rate of a specific storm event. Therefore, it is not appropriate to use the Rational method for sizing BMPs that require knowing the volume of stormwater runoff generated in order to determine the size of the BMP. The Rational method is often used in sizing Manufactured Treatment Devices by a determination of the peak flow rate of inflow.

The Stormwater Management rules define the NJDEP Water Quality Design Storm as 1.25 inches of rainfall in a 2-hour period. However, as explained on Page 19, **the rainfall distribution for this storm event is not evenly distributed over the entire duration**; therefore, **using an average intensity of 1.25 in over 2 hr, or 0.625 in/hr cannot be used as the value for *i*, rainfall intensity, in the Rational Method equation**. Instead, refer to Table 5-2, found on Page 20, or Figure 5-16, located below, for the correct rainfall intensity value corresponding to the calculated time of concentration.

Figure 5-16: NJDEP 1.25-Inch/2-Hour Stormwater Quality Design Storm Rainfall Intensity-Duration Curve



Rational Method Example

The following example illustrates how to correctly use the Rational Method to calculate the peak flow rate of the NJDEP WQDS. It is followed by a discussion of the Modified Rational Method. Example 5-9, illustrating the Modified Rational Methodology to size an extended detention basin, begins on Page 75.

Example 5-8: Determine the Peak Flow Rate of the NJDEP WQDS

A drainage area consists of a 1 ac asphalt pavement parking lot. Determine the peak flow rate for the post-construction condition.

From Figure 5-16, the rainfall intensity corresponding to 10 minutes is 3.2 in/hr. The runoff coefficient for an asphalt pavement parking lot obtained from Table 5-3 is 0.99. The peak flow rate is calculated as follows:

$$Q_p = \mathrm{C}\ i\ \mathrm{A} = 0.99\ x\ \ 3.2\,\frac{in}{hr}\ x\ \ 1\ \mathrm{ac} = 3.17\ \text{cfs}$$

If the average rainfall intensity is <u>incorrectly</u> chosen, i.e., i = 0.625 inches/hour, the calculated peak flow rate would be:

$$Q_p = \mathrm{C}\ i\ \mathrm{A} = 0.99\ x\ 0.625\,\frac{in}{hr}\ x\ 1\ \mathrm{ac} = 0.619\ \text{cfs}$$

which is roughly 1/5th the correct intensity. **If an intensity of 0.625 in/hr were used to size an MTD, it would be drastically undersized.**

Modified Rational Method

The Rational Method was originally developed in 1889 for calculating the peak flow rate of stormwater runoff, not the volume of the runoff. In the 1970s, the Modified Rational Method was developed to size storage facilities, such as a reservoir or detention basin. The Modified Rational Method refers to a procedure for manipulating the basic rational method techniques to reflect the fact that storms with durations greater or less than the normal time of concentration for a basin will result in a larger volume of runoff even though the peak discharge is reduced. Under this concept, hydrographs are developed for various situations with regard to times of concentration and rainfall durations. Take note that the Modified Rational Method cannot be used to calculate the peak flow rate. For the Modified Rational Method, see "Appendix A-9 Modified Rational Method" in the *Standards for Soil Erosion and Sediment Control in New Jersey,* January 2014.

The Stormwater Management rules dictate that Rational Method is used for calculating peak flow rates and the Modified Rational Method is used to develop hydrographs. It is essential to not confuse the intensity used for the peak flow rate calculations under the Rational Method with the intensity used for the Modified Rational Method.

The volumes under the hydrographs developed using the Modified Rational Method are equal to the runoff volumes generated from the total rainfall depth and the corresponding duration of the storm event (2-hours for Water Quality Design Storm and 24-hours for 2-, 10- and 100- year storms), regardless of the time of concentration.

Example 5-9: Sizing a Detention Basin Using the Modified Rational Method

The Modified Rational Method is to be used to size a stormwater management basin for the 100-year storm. The existing condition peak flow rate is 7.024 cfs. The drainage area is 20 acres.

Step 1: Determine the allowable discharge peak flow for 100-year storm.

In accordance with the Stormwater Management rules requirement for stormwater runoff quantity control, the allowable discharge rate for the 100-year storm is 80% of the pre-construction peak flow rate. The pre-construction peak flow rate is 7.024 cfs. The allowable 100-year peak flow rate is therefore 5.62 cfs.

Step 2: Find the storm duration that will produce the maximum difference between the inflow volume and the outflow volume, assuming the maximum allowable peak outflow rate is the rate of discharge from the proposed detention basin.

Different stormwater durations and their rainfall intensities are used to calculate the inflow volumes. In Table 5-4, shown on the following page, the design storm has a 100-year recurrence interval. Since the rainfall intensity decreases as the duration of rainfall increases, compare the rainfall intensity for the 5-min storm with that of the 12-hr storm.

Table 5-4: NOAA Precipitation Frequency Data

POINT PRECIPITATION FREQUENCY (PF) ESTIMATES

WITH 90% CONFIDENCE INTERVALS AND SUPPLEMENTARY INFORMATION
NOAA Atlas 14, Volume 2, Version 3

PF tabular | PF graphical | Supplementary information | Print page

PDS-based precipitation frequency estimates with 90% confidence intervals (in inches/hour)[1]

Duration	Average recurrence interval (years)								
	1	2	5	10	25	50	100	200	500
5-min	**4.12** (3.73-4.54)	**4.91** (4.46-5.41)	**5.83** (5.28-6.42)	**6.50** (5.88-7.16)	**7.34** (6.60-8.08)	**7.96** (7.12-8.75)	**8.56** (7.62-9.44)	**9.13** (8.08-10.1)	**9.84** (8.63-11.0)
10-min	**3.29** (2.98-3.62)	**3.92** (3.57-4.33)	**4.67** (4.23-5.14)	**5.20** (4.70-5.72)	**5.85** (5.26-6.43)	**6.34** (5.67-6.97)	**6.80** (6.06-7.51)	**7.24** (6.41-8.01)	**7.78** (6.82-8.67)
15-min	**2.74** (2.49-3.02)	**3.29** (2.99-3.62)	**3.94** (3.57-4.34)	**4.39** (3.97-4.83)	**4.94** (4.44-5.44)	**5.35** (4.78-5.88)	**5.73** (5.10-6.32)	**6.08** (5.39-6.74)	**6.53** (5.72-7.27)
30-min	**1.88** (1.71-2.07)	**2.27** (2.07-2.50)	**2.80** (2.54-3.08)	**3.18** (2.88-3.50)	**3.66** (3.29-4.03)	**4.03** (3.60-4.43)	**4.39** (3.91-4.84)	**4.74** (4.19-5.24)	**5.20** (4.55-5.79)
60-min	**1.17** (1.06-1.29)	**1.43** (1.30-1.57)	**1.79** (1.63-1.98)	**2.07** (1.87-2.28)	**2.44** (2.19-2.68)	**2.73** (2.44-3.00)	**3.02** (2.69-3.34)	**3.32** (2.94-3.68)	**3.73** (3.27-4.15)
2-hr	**0.707** (0.642-0.781)	**0.862** (0.782-0.950)	**1.09** (0.988-1.20)	**1.27** (1.14-1.39)	**1.51** (1.35-1.66)	**1.70** (1.52-1.87)	**1.90** (1.69-2.10)	**2.11** (1.86-2.33)	**2.40** (2.08-2.67)
3-hr	**0.517** (0.467-0.574)	**0.630** (0.570-0.699)	**0.798** (0.720-0.886)	**0.929** (0.836-1.03)	**1.11** (0.995-1.23)	**1.26** (1.12-1.40)	**1.42** (1.25-1.58)	**1.58** (1.38-1.76)	**1.81** (1.56-2.02)
6-hr	**0.328** (0.296-0.365)	**0.398** (0.359-0.443)	**0.503** (0.453-0.559)	**0.588** (0.527-0.653)	**0.712** (0.632-0.790)	**0.814** (0.718-0.904)	**0.925** (0.808-1.03)	**1.05** (0.901-1.16)	**1.22** (1.03-1.36)
12-hr	**0.197** (0.179-0.221)	**0.239** (0.216-0.268)	**0.304** (0.274-0.340)	**0.359** (0.322-0.401)	**0.442** (0.392-0.492)	**0.513** (0.451-0.573)	**0.592** (0.513-0.660)	**0.679** (0.579-0.761)	**0.809** (0.676-0.913)

The inflow volume in excess of the outflow volume discharging at the allowable outflow rate must be detained in the detention basin to maintain a constant, yet allowable, rate of peak flow out of the detention basin. Construction of a table like the one shown on the following page can facilitate the calculation.

Table 5-5: Modified Rational Method Basin Design Table

Column:	A	B	C	D	E	F	G
Row No.	**Storm Duration (min)**	**Storm Intensity (in/hr)**	**Inflow Rate (cfs)**	**Runoff Volume (cf)**	**Outflow Rate (cfs)**	**Outflow Volume (cf)**	**Storage Volume (cf)**
1	5	8.56					

- The storm duration (Column A) can be set at any interval of time, such as 20 minutes, 1 hour, etc. Smaller intervals of time provide more accurate estimates. For this example, the values of storm duration are initially chosen to match the storm durations in the NOAA website precipitation frequency data as shown in Table 5-4.

- Values for storm intensity (Column B) may be obtained from intensity-duration-frequency curve in accordance with the stormwater recurrence frequency and the storm duration. For the current example, the stormwater recurrence frequency is 100-year storm. The storm duration is the values in column A. From Table 5-4, for example, the storm intensity is 8.56 in/hr for a 5-min duration, 100-year storm. In Row 1, enter "5" in Column A and enter 8.56 in Column B in Table 5-5. Note that these particular values will not be used in the remainder of the example.

- The inflow rate (Column C) is the multiplication of storm intensity (Column B), the size of the contributing inflow drainage area, and the C value, which is assumed to be 0.99 for an impervious surface. As discussed on Page 70, no conversion factor is required to adjust the units of the rational method equation.

- The runoff volume (Column D) is the multiplication of inflow rate (Column C) with the storm duration (Column A), along with the appropriate conversion of units from minutes to seconds.

- The outflow rate (Column E) is the allowable outflow rate from "Step 1."

- The outflow volume (Column F) is the multiplication of the outflow rate (Column E) with the storm duration (Column A). Also include the unit conversion from minutes to seconds in this step.

- The storage volume (Column G) is the subtraction of outflow volume (Column F) from the runoff volume (Column D).

Table 5-5 now looks like this, once the values are calculated as discussed above:

Table 5-5: Modified Rational Method Basin Design Table (revised)

Column:	A	B	C	D	E	F	G
Row No.	**Storm Duration (min)**	**Storm Intensity (in/hr)**	**Inflow Rate (cfs)**	**Runoff Volume (cf)**	**Outflow Rate (cfs)**	**Outflow Volume (cf)**	**Storage Volume (cf)**
1	10	6.80	134.6	80760	5.62	3372	77388
2	15	5.73	113.5	102150	5.62	5058	97092
3	30	4.39	86.9	156420	5.62	10116	146304
4	60	3.02	59.8	215280	5.62	20232	195048
5	120	1.90	37.6	270720	5.62	40464	230255
6	180	1.42	28.1	303480	5.62	60696	242784
7	360	0.925	18.3	395280	5.62	121392	273888
8	720	0.592	11.7	505440	5.62	242784	262656

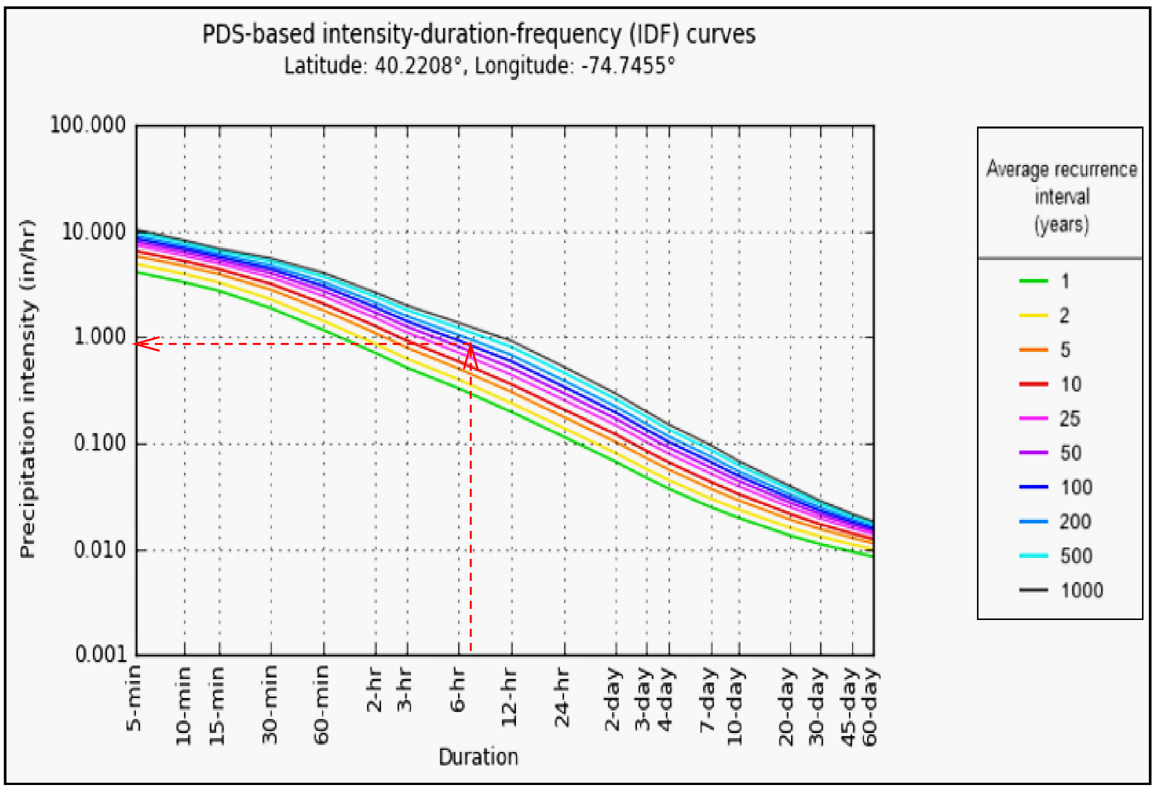
The storage volume shown in Column G increases to 273,888 cf then decrease to 262,656 cf between the durations of 360 and 720 minutes. This means that the maximum storage volume occurs somewhere between these durations. Further analysis is needed as there is a time difference of 360 minutes between the two durations.

Next, an increment of 60 minutes between durations 360 and 720 minutes is used to zero in on the maximum storage volume. The storm intensity is still obtained from the precipitation-duration-frequency ("PDS") data on the NOAA website. However, the tabular data does not provide data at

60-minute intervals between the targeted durations. This means the graphical curve will need to be consulted. NOAA's intensity-duration-frequency ("IDF") curves are shown below. To obtain the value for a particular precipitation intensity, there is a three-step process.

- First, locate the curve for the desired recurrence interval from the color-coded key on the right in the figure reproduced below.
- Second, locate the storm duration on the x-axis and project a line up to the recurrence interval curve.
- Finally, project a horizontal over to the y-axis to find the precipitation intensity.

Take note that the scale for each axis is logarithmic. For example, for a duration of 540 minutes (9 hours), the 100-year curve yields an intensity of 0.757 in/hr, shown below with red arrows.



Rows are then added to Table 5-5 and the method described above is used to complete the additional rows, resulting in a table that now looks, in part, like that depicted below:

Table 5-5: Modified Rational Method Basin Design Table - Expanded

	A	B	C	D	E	F	G
Row No.	**Storm Duration (min)**	**Storm Intensity (in/hr)**	**Inflow Rate (cfs)**	**Runoff Volume (cf)**	**Outflow Rate (cfs)**	**Outflow Volume (cf)**	**Storage Volume (cf)**
7	360	0.925	18.32	395712	5.62	121392	274320
8	420	0.869	17.21	433692	5.62	141624	292068
9	480	0.813	16.10	463680	5.62	161856	301824
10	**540**	**0.757**	**14.99**	**485676**	**5.62**	**182088**	**303588**
11	600	0.701	13.88	499680	5.62	202320	297360
12	660	0.645	12.77	505692	5.62	222552	283140
13	720	0.592	11.72	506304	5.62	242784	263520

Shown as bolded text in the table above, the storage volume increases to reach a maximum of 303,543 cf for a storm duration of 540 min. The expanded table is therefore a more accurate estimate of the maximum storage volume for the allowable discharge rate of 5.62 cfs for the 100-year storm. Of course, further refinements can be made using smaller time increments between durations of 480 and 540 minutes, if so desired.

The detention basin will need further configuration based on the storage volume of 303,543 cf, including sizing of orifices and maximum water depth to be consistent with the allowable peak outflow of 5.62 cfs. The method described above should also be performed for the 2- and 10-year storms to demonstrate the volumes and outflow rates meet the required 50% and 75% reduction rates. Further design and performance standards and other details for extended detention basins are addressed in *Chapter 9.4* of this manual.

Guidance Summary

The following table summarizes how the methods discussed in this chapter may be used to meet the various requirements established in N.J.A.C. 7:8.

Table 5-6: Summary of Modeling Guidance for Various Site Conditions when using the Rational, Modified Rational and NRCS Method

Site Condition or Parameter	Rational Method	Modified Rational Method	NRCS Methodology
Applicability	Peak flow rates	Sizing detention BMPs	Peak flow rate, runoff volume, hydrograph comparison, sizing inflow rate and volume of BMPs
Groundwater recharge	Not applicable	Not applicable	Difference of Runoff volumes of pre- and post-construction 2-year storms
Mixture of pervious and directly connected impervious surfaces	Use standard procedures	Use standard procedures	Calculate the runoff from impervious surface and pervious surface separately
Unconnected impervious surface	Not applicable	Not applicable	*NEH Part 630, Chapter 9* unconnected impervious surface (less than 30%) or NJ DEP Two-Step Technique
Runoff parameters	Runoff coefficients from Table 5-3	Runoff coefficients from Table 5-3	Curve Numbers from *NEH, Part 630, Chapter 9*
Rainfall data	NOAA NWS rainfall intensity-frequency data and NJDEP water quality storm rainfall intensity-duration curve in Figure 5-16		NRCS County rainfall averages, or NOAA NWS rainfall-frequency data NJDEP water quality storm rainfall depth and distribution in Table 5-2
Time of concentration (pre-construction)	Estimated through calculation of time of travel Sheet flow length = 100 feet Maximum sheet flow roughness coefficient n = 0.40		
Time of concentration (post-construction	Estimated through calculation of time of travel and McCuen-Spiess limitation Minimum T_c that may be used is 10 minutes for 2-, 10- and 100-year storms and Water Quality Design Storm		Estimated through calculation of time of travel and McCuen-Spiess limitation. There is no minimum value for T_c An assumed T_c value may not be used Max Sheet flow length = 100 feet Maximum sheet flow roughness coefficient n = 0.40

References

Center for Watershed Protection. March 2003. Impacts of Impervious Cover on Aquatic Systems – Watershed Protection Research Monograph No. 1. Ellicott City, MD.

Claytor, R. and T. Schueler. December. 1996. Design of Stormwater Filtering Systems. The Center for Watershed Protection. Ellicott City, MD.

Desbonnet, A., P. Pogue, V. Lee, and N Wolff. 1994. Vegetated Buffers in the Coastal Zone – A Summary Review and Bibliography, Coastal Resources Center Technical Report No. 2064. University of Rhode Island Graduate School of Oceanography. Narragansett, RI.

Horner, R.R., J.J. Skupien, E.H. Livingston and H.E. Shaver. August 1994. Fundamentals of Urban Runoff Management: Technical and Institutional Issues. In cooperation with U.S. Environmental Protection Agency. Terrene Institute, Washington, D.C.

Langan, T. National Stormwater BMP Database, NSW Statistical Summary 6-18-03 Tables.

Lucas, W.C. October 2002. DURMM: Delaware Urban Runoff Management Model Users Manual. Delaware Department of Natural Resources and Environmental Conservation.

Massachusetts Department of Environmental Protection. 1997. Stormwater Management Volume Two: Stormwater Technical Handbook.

McCuen, Richard H. Development and Evaluation of TR-55 Composite Curve Number for Unconnected Imperviousness.

New Jersey Department of Agriculture. January 2014. Standards for Soil Erosion and Sediment Control in New Jersey. State Soil Conservation Committee. Trenton, NJ.

New Jersey Department of Environmental Protection and Department of Agriculture. December 1994. Stormwater and Nonpoint Source Pollution Control Best Management Practices.

New Jersey Department of Environmental Protection, Land Use Management Program. September 1995. Technical Manual for Stream Encroachment Permits.

New Jersey Department of Environmental Protection, Land Use Management Program. 1988. Technical Manual for Land Use Regulation Program, Bureau of Inland and Coastal Regulations, NJDEP Flood Hazard Area Permits, New Jersey Department of Environmental Protection. Trenton, NJ.

Northeast National Technical Center. June 1986. Hydrology Technical Note No. N4 – Time of Concentration. USDA Natural Resources Conservation Service.

Northeast National Technical Center. 1986. Memorandum: Sheet Flow in the Northeast. USDA Natural Resources Conservation Service.

Pitt, R., J. Lantrip and R. Harrison. December 1999. Infiltration through Disturbed Urban Soils and Compost-Amended Soil Effects on Runoff Quality and Quantity. U.S. Environmental Protection Agency.

Schueler, T.R. July 1987. Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs. Metropolitan Washington Council of Governments. Washington, D.C.

Schueler, T.R. and R.A. Claytor. 2000. Maryland Stormwater Design Manual. Maryland Department of the Environment. Baltimore, MD.

Strecker, E.W., M.M. Quigley, B. Urbonas. Results of Analyses of the Expended EPA/ASCS National BMP Database.

U.S. Army Corps of Engineers Hydrologic Engineering Center. HEC-1 Flood Hydrograph Package Users Manual.

U.S. Army Corps of Engineers Hydrologic Engineering Center. HEC-HMS Hydrologic Modeling System Users Manual.

USDA Natural Resources Conservation Service. National Engineering Handbook, Part 630, Hydrology.

USDA Natural Resources Conservation Service. Technical Release 20 – Project Formulation – Hydrology.

USDA Natural Resources Conservation Service. June 1986. Technical Release 55 – Urban Hydrology for Small Watersheds.

USDA Natural Resources Conservation Service. April 2002. Win-TR55 – User Manual.

USDA Natural Resources Conservation Service, September 10, 2012, Engineering Field Handbook Supplement, Chapter 2, Estimating Runoff and Peak Discharges, NEW JERSEY BULLETIN NO. NJ210-12-1.

USDA Natural Resources Conservation Service, September 27, 2012, WinTR-55 frequently asked precipitation questions.

USDA Natural Resources Conservation Service, December 17, 2001, References on time of concentration with respect to sheet flow, USDA, NRCS, National Water and Climate Center, Beltsville, MD.

USDA Natural Resources Conservation Service, January 2009, Small Watershed Hydrology WinTR–55 User Guide, Washington D.C.

Viessman, W. Jr., J.W. Knapp, G.L. Lewis and T.E. Harbaugh. 1977 Introduction to Hydrology – Second Edition. Harper & Row. New York.

Walesh, S.G. 1989. Urban Surface Water Management. John Wiley & Sons, Inc. New York.

Wanielista, M.P. and Y.A. Yousef. 1993. Stormwater Management. John Wiley & Sons, Inc. New York.

Whipple, W., N.S. Grigg, T. Grizzard, C.W. Randall, R.P. Shubinski and L.S. Tucker. 1983. Stormwater Management in Urbanizing Areas. Prentice-Hall, Inc. Englewood Cliffs, New Jersey.

Winer, R. June 2000. National Pollutant Removal Performance Database for Stormwater Treatment Practices, 2nd Edition, Center for Watershed Protection.

Wong, S.L. and R.H. McCuen. Design of Vegetated Buffer Strips for Runoff and Sediment Control.