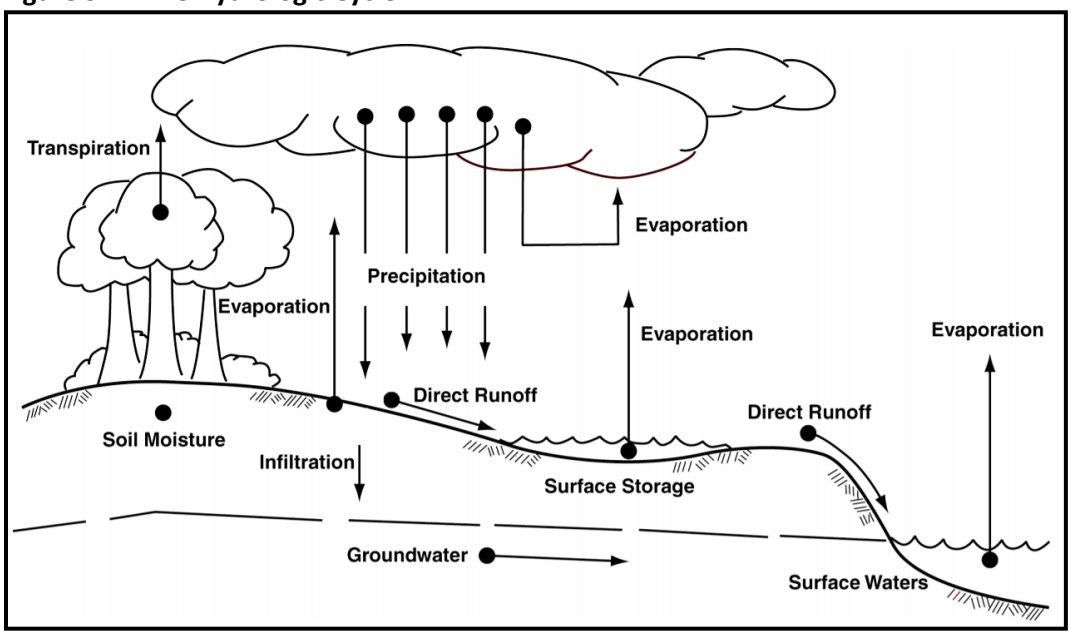
5. Stormwater Management Quantity and Quality Standards and Computations

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) methodology, which is 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 hydrologic 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 methodology listed below, which will be further discussed in later section of this chapter:

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/landingpage/30411 or

at the United States Department of Agriculture Natural Resources Conservation Service.

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 organizations that collect and publish hydrological data, such as National Oceanic and Atmospheric Administration's (NOAA) National Weather Service (NWS), which 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, NOAA 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 NOAA data is found beginning on Page 12.

Regulatory Requirements of the Stormwater Management Rules

The Stormwater Management rules set forth stormwater runoff quantity control, 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. Major development projects must demonstrate compliance with those standards. Take note that throughout this chapter, terms such as "*current storm event*" and "*projected storm event*" are used to identify the design storms for the calculations of runoff from the 2-, 10-, and 100-year storms that have been updated to the rainfall data up to 2019 and projected for climate change.

The studies of the rainfall updates and projections by the Cornell University and peer-reviewed by the Department's Science Advisory Board are listed in the references. These terms are further explained below and in later subsections of this chapter.

Current Storm Event– term used where the precipitation data published by the NOAA Atlas 14 Precipitation-Frequency Atlas of the United States, Volume 2, Version 3, last revised in 2006 (NOAA Atlas 14 Rainfall Depth) is modified by multiplication with a current precipitation adjustment factor listed in Table 5-5 at N.J.A.C. 7:8-5(c)2, which updates the rainfall data from 1999 to 2019.

Projected Storm Event – term used where the NOAA Atlas 14 Rainfall Depth is modified by multiplication with a future precipitation change factor listed in Table 5-6 at N.J.A.C. 7:8-5(d), which projects the precipitation depth for the time period from 2050 to 2099 due to the continued impacts of climate change.

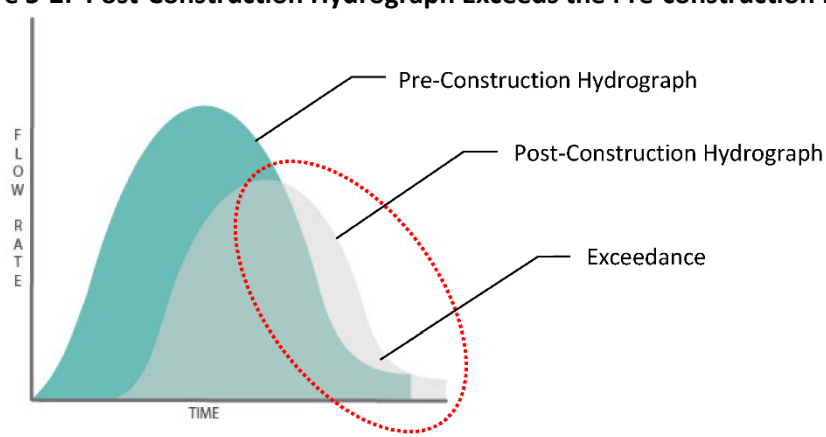
Stormwater Runoff Quantity Control Design and Performance Standards

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. Unless the project is granted a variance pursuant to N.J.A.C. 7:8-4.6(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:

a. *Demonstrate through hydrologic and hydraulic analysis that for stormwater leaving the site, post-construction runoff hydrographs for the* ***current and projected*** *two-, 10- and 100-year storm events, as defined and determined pursuant to N.J.A.C. 7:8-5.7(c) and (d), respectively, do not exceed, at any point in time, the pre-construction runoff hydrographs for the same storm events.*

   Below is Figure 5-2, which is an illustration demonstrating noncompliance with the requirement under N.J.A.C. 7:8-5.6(b)1, followed by Figure 5-3 which demonstrates compliance:

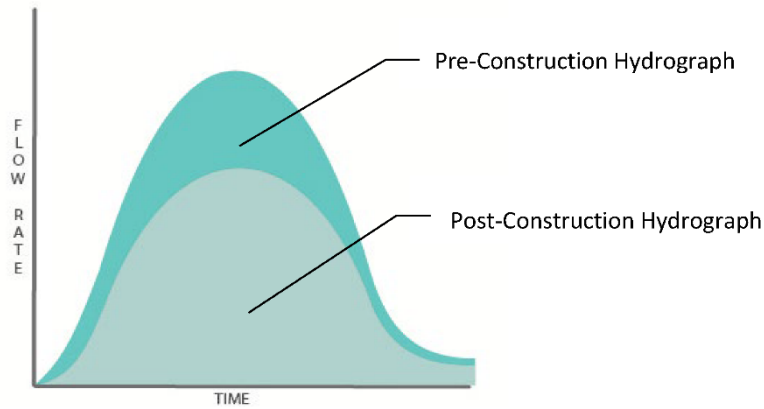
Figure 5-2: Post-Construction Hydrograph Exceeds the Pre-construction Hydrograph



   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 Figure 5-3, 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 control 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.

b. *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* ***current and projected*** *two-, 10- and 100-year storm events, as defined and determined pursuant to N.J.A.C. 7:8-5.7(c) and (d), respectively, 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 current and projected 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 current and projected 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.

c. *Design stormwater management measures so that the post-construction peak runoff rates for the* ***current and projected*** *two-, 10- and 100-year storm events, as defined and determined pursuant to N.J.A.C. 7:8-5.7(c) and (d), respectively, 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 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 the 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. 5:21-7.1(a), allow municipalities to require stormwater runoff controls for development falling below the major development threshold to address the control of the runoff rate and routing from any site that is the subject of a site plan or subdivision application.

- 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. 7:8-5.2(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** current and/or projected 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 may 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 Control 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 47, illustrate the methodology to be used.

Calculation of Stormwater Runoff

NRCS Methodology

As stated above, for the purposes of managing potential flooding, stormwater runoff quantity and quality, it is essential to calculate the volume and peak flow of the stormwater runoff produced by a storm event. In accordance with N.J.A.C 7:8-5.7, the U.S. Department of Agriculture NRCS methodology is the only method acceptable for use in the computation of stormwater runoff.

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 method 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 online at:

https://directives.sc.egov.usda.gov/landingpage/30411.

Basics of the NRCS Methodology

- The NRCS methodology can provide total stormwater runoff volume, the peak flow rate and produce hydrographs.
  - Under the NRCS methodology, 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 27.
- Limitations on the size and/or cover condition of the drainage area must also be taken into consideration.
  - *Chapter 10* of the *NEH*, *Part 630*, titled "Estimation of Direct Runoff from Storm Rainfall" does not indicate any drainage area size limitation when apply the NRCS methodology, but *Chapter 16* of the *NEH*, *Part 630*, titled "Hydrograph," recommends that "no subarea exceeds 20 square miles in area" when applying the unit hydrograph in the NRCS methodology. Practically speaking, this limitation has no significant impact on the application of the NRCS methodology in a single major development, which is normally far less than 20 square miles.
  - The drainage area is 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 unless the drainage area meets the criteria of unconnected impervious cover, which is discussed later in this chapter.

- The contributory drainage area limitations imposed by N.J.A.C. 7:8-5.3(b) for the individual stormwater management best management practices (BMPs) are discussed in the respective subchapters of *Chapter 9: Green Infrastructure BMPS*.

A table is provided on Page 84 which summarizes the applicability of the NRCS methodology discussed in this chapter and how the methodology is to be used.

Information Required to Use the NRCS Methodology

Table 5-1 lists the information required in order to use the NRCS methodology of computing stormwater runoff. Examples are provided and begin on Page 32.

Table 5-1: NRCS Methodology Index of Information Required

Topic No.	Information Required to use the NRCS Methodology	Page No.
1	Hydrologic Soil Group of the drainage area soil	11
2	Sub-drainage areas	11
3	Land cover	11
4	Rainfall depth for the stormwater runoff quantity control design storms	12
5	Rainfall distribution for the stormwater runoff quantity control design storms	19
6	Rainfall depth for the stormwater runoff water quality design storm	20
7	Rainfall distribution for the stormwater runoff water quality design storm	20
8	Time of travel and time of concentration	23
8	Maximum sheet flow roughness coefficient	23
8	Sheet flow length	24
8	Shallow concentrated flow	25
8	Open channel flow	26
8	T_C routes	26
9	Runoff Hydrographs	27
10	Directly Connected Impervious Cover	30
11	Unconnected Impervious Cover	31
12	Reduced Curve Numbers	32
13	Calculating the Peak Flow Rate of Runoff for the WQDS	32

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. Although some hydrologic modelling software packages allow the user to separately calculate the runoff from impervious and pervious areas by modelling them as one sub-drainage area, **this modelling setting may only be used if the impervious area time of concentration is the same as the pervious area time of concentration**. The calculation of the time of concentration begins on Page 23.

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. N.J.A.C. 7:8-5.6 requires the use of current and projected storm events in the demonstration of meeting the stormwater runoff standards. The current and projected storm events are defined and determined in N.J.A.C. 7:8-5.7(c)2 and N.J.A.C. 7:8-5.7(d)2. The steps to obtain the rainfall precipitation depths of the current and projected storm events are illustrated below.

   a. First, the designer must obtain the rainfall data for the site location. Rainfall data is available, as follows:

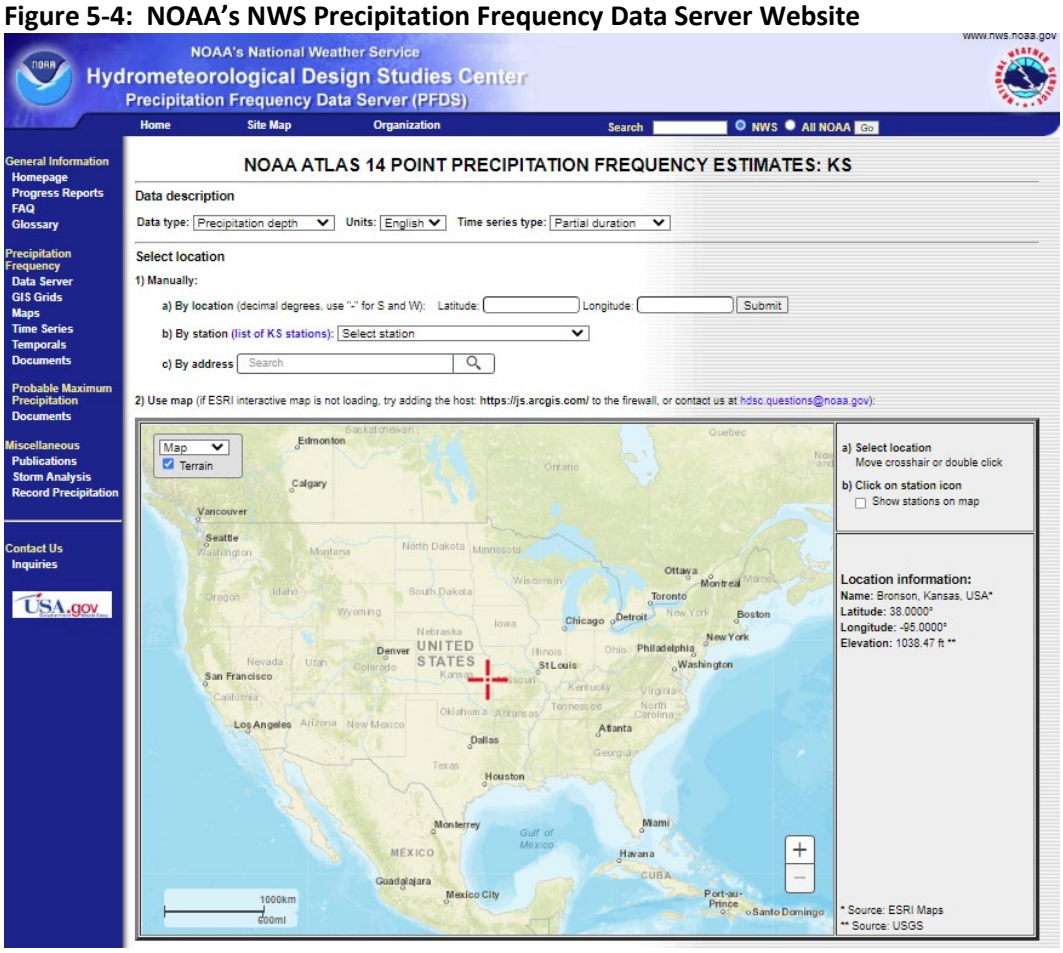
      i. NOAA Atlas 14 rainfall data is provided by NOAA's NWS Precipitation Frequency Data Server (PFDS), which is available online at:

      https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=nj.

Example A-1: Obtain rainfall depth data for a location in Trenton, NJ, using the above link.

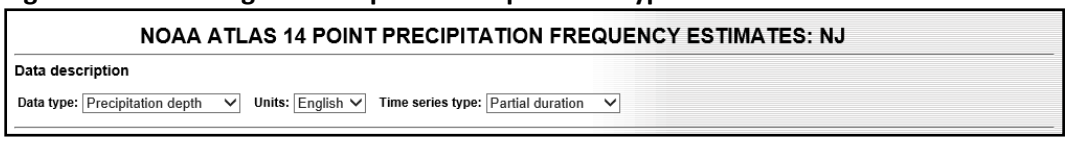
Step 1: Under the Select location option, depicted in Figure 5-4 below, manually enter the location using one of the options offered on the screen or use the map to locate the site, making sure to zoom in.

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



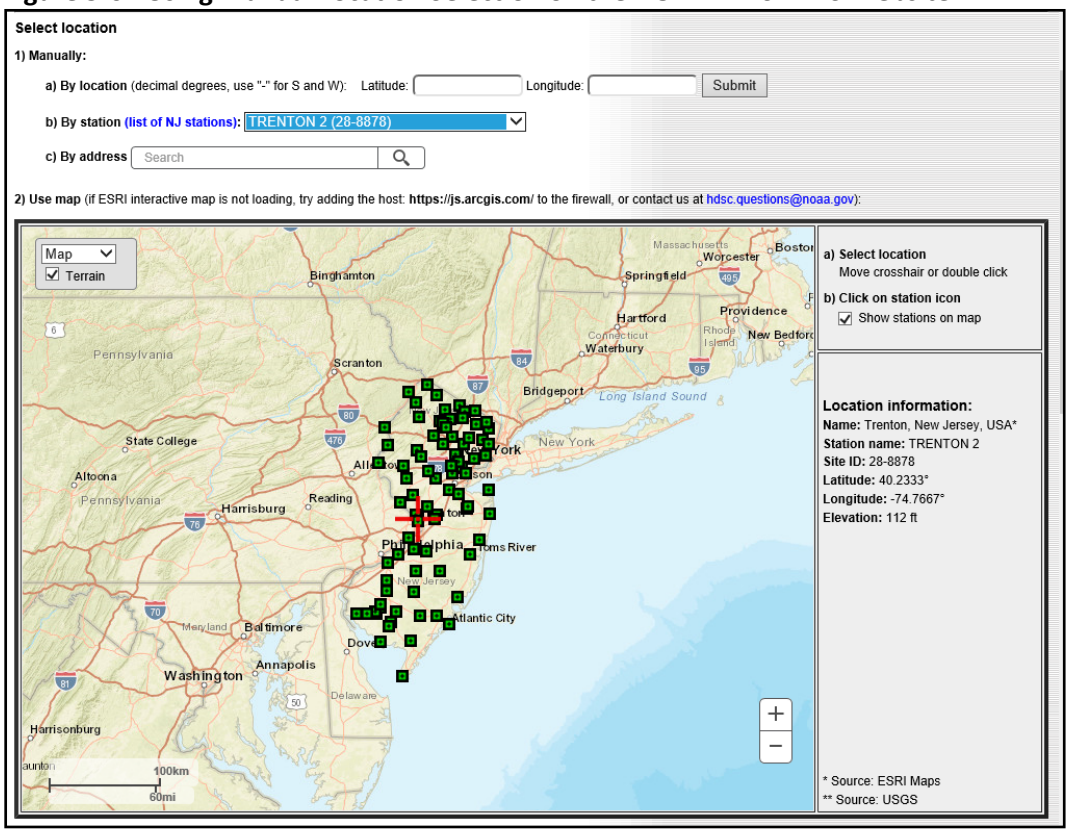
Step 2: In the Data description section of the next window that opens, from the Select Data Type dropdown menu, make sure "Precipitation depth" rather than "Precipitation intensity" is selected. Similarly, for the Time series type, ensure "Partial duration" has been selected from that dropdown menu, as shown in Figure 5-5 below.

Figure 5-5: Selecting the Precipitation Depth Data Type



To illustrate this process, Trenton Station 2 was selected from the dropdown menu under 1.b), as shown in Figure 5-6 below:

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



Step 3: 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 shaded in dark blue, as shown in Figure 5-7.

Figure 5-7: Point Precipitation Frequency (PF) Estimates – Tabular Option for Trenton Station 2

POINT PRECIPITATION FREQUENCY (PF) ESTIMATES

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

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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 below.

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

POINT PRECIPITATION FREQUENCY (PF) ESTIMATES

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

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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 (6.42-7.66)	8.15 (7.32-8.82)	9.32 (8.28-10.1)	11.1 (9.67-12.0)	12.5 (10.8-13.7)

ii. N.J.A.C. 7:8-5.7(c)2 and N.J.A.C. 7:8-5.7(d)2 both allow an alternative to calculating the current and projected rainfall precipitation depths by using separate rainfall totals for each county. The 24-hour county rainfall amount provided by NRCS is duplicated here and can be found online at:

https://www.nrcs.usda.gov/sites/default/files/2022-09/NJ%2024%20Hour%20Rainfall%20Data.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. N.J.A.C.7:8-5.7(c) requires the precipitation depths of the current 2-, 10- and 100-year storm events be determined by multiplying the NOAA rainfall data with the current precipitation adjustment factors in Table 5-5 at N.J.A.C.7:8-5.7(c)2. N.J.A.C.7:8-5.7(d) requires the precipitation depths of the projected 2-, 10- and 100-year storm events be determined by multiplying the NOAA rainfall data with the future precipitation change factors in Table 5-6 at N.J.A.C.7:8-5.7(d). Table 5-5 and Table 5-6 from the Rules are reproduced below.

Current Precipitation Adjustment Factors at N.J.A.C. 7:8-5.7(c) as Table 5-5

County	Current Precipitation Adjustment Factors		
	2-year Design Storm	10-year Design Storm	100-year Design Storm
Atlantic	1.01	1.02	1.03
Bergen	1.01	1.03	1.06
Burlington	0.99	1.01	1.04
Camden	1.03	1.04	1.05
Cape May	1.03	1.03	1.04
Cumberland	1.03	1.03	1.01
Essex	1.01	1.03	1.06
Gloucester	1.05	1.06	1.06
Hudson	1.03	1.05	1.09
Hunterdon	1.02	1.05	1.13
Mercer	1.01	1.02	1.04
Middlesex	1.00	1.01	1.03
Monmouth	1.00	1.01	1.02
Morris	1.01	1.03	1.06
Ocean	1.00	1.01	1.03
Passaic	1.00	1.02	1.05
Salem	1.02	1.03	1.03
Somerset	1.00	1.03	1.09
Sussex	1.03	1.04	1.07
Union	1.01	1.03	1.06
Warren	1.02	1.07	1.15

Future Precipitation Change Factors at N.J.A.C. 7:8-5.7(d) as Table 5-6

County	Future Precipitation Change Factors		
	2-year Design Storm	10-year Design Storm	100-year Design Storm
Atlantic	1.22	1.24	1.39
Bergen	1.20	1.23	1.37
Burlington	1.17	1.18	1.32
Camden	1.18	1.22	1.39
Cape May	1.21	1.24	1.32
Cumberland	1.20	1.21	1.39
Essex	1.19	1.22	1.33
Gloucester	1.19	1.23	1.41
Hudson	1.19	1.19	1.23
Hunterdon	1.19	1.23	1.42
Mercer	1.16	1.17	1.36
Middlesex	1.19	1.21	1.33
Monmouth	1.19	1.19	1.26
Morris	1.23	1.28	1.46
Ocean	1.18	1.19	1.24
Passaic	1.21	1.27	1.50
Salem	1.20	1.23	1.32
Somerset	1.19	1.24	1.48
Sussex	1.24	1.29	1.50
Union	1.20	1.23	1.35
Warren	1.20	1.25	1.37

Example B-1: Apply the current precipitation adjustment factors for the site depicted in Example A-1.

The table below includes the rainfall depths from Example A-1 and the Mercer County current precipitation adjustment factors from Table 5-5 in the Stormwater Management rules for the site located in Trenton, NJ. To obtain the current precipitation depth for the 2-year design storm, multiply the precipitation depth for the 2-year design storm by the 2-year current precipitation adjustment factor. Repeat the process for the 10- and 100-year design storms using the respective adjustment factor. The results are depicted in the column farthest to the right in the table below.

Frequency of Storms	NOAA NWS PFDS Rainfall Depth (inches)	Current Precipitation Adjustment Factor	Current Rainfall Depth (inches)
2-year	3.33	1.01	3.36
10-year	4.99	1.02	5.09
100-year	8.15	1.04	8.48

Example B-2: Determine the current precipitation adjustment factors for a site with one-third of the area located in Middlesex County and the remainder in Mercer County.

In the event a project spans the boundary line(s) between counties, N.J.A.C. 7:8-5.7(c)2 instructs the user as follows: "*Where the major development lies in more than one county, the precipitation values shall be adjusted according to the percentage of the drainage area in each county*."

Step 1: Obtain the current precipitation for each of the counties.

County	Current Precipitation		
	2-year Design Storm	10-year Design Storm	100-year Design Storm
Middlesex	3.35 * 1.00 = 3.35	5.12 * 1.01 = 5.17	8.63 * 1.03 = 8.89
Mercer	3.31 * 1.01 = 3.34	5.01 * 1.02 = 5.11	8.33 * 1.04 = 8.66

The method is the same as that shown above in Example A-1, but is performed for each of counties using the respective current precipitation adjustment factor.

Step 2: **Multiply the percentage of the project area within each county with the current precipitation for the respective county, and sum the results for each of the design storms, pursuant to N.J.A.C. 7:8-5.7(c)2.**

The results are shown in the following table.

County, Weight Averaged	Current Precipitation		
	2-year Design Storm	10-year Design Storm	100-year Design Storm
Middlesex	0.333 * 3.35 = 1.12	0.333 * 5.17 = 1.72	0.333 * 8.89 = 2.96
Mercer	+ 0.667 * 3.34 = 2.23	+ 0.667 * 5.11 = 3.41	+ 0.667 * 8.66 = 5.78
Total Site Precipitation	**3.35**	**5.13**	**8.74**

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. Both NOAA_C and NOAA_D rainfall distributions, in TBL format for Win TR-55 or other compatible hydrology modeling software, are online, available for downloading, under the heading of Win TR-55 Data (scroll down to find that heading), at:

   https://www.nrcs.usda.gov/conservation-basics/conservation-by-state/new-jersey/new-jersey-engineering

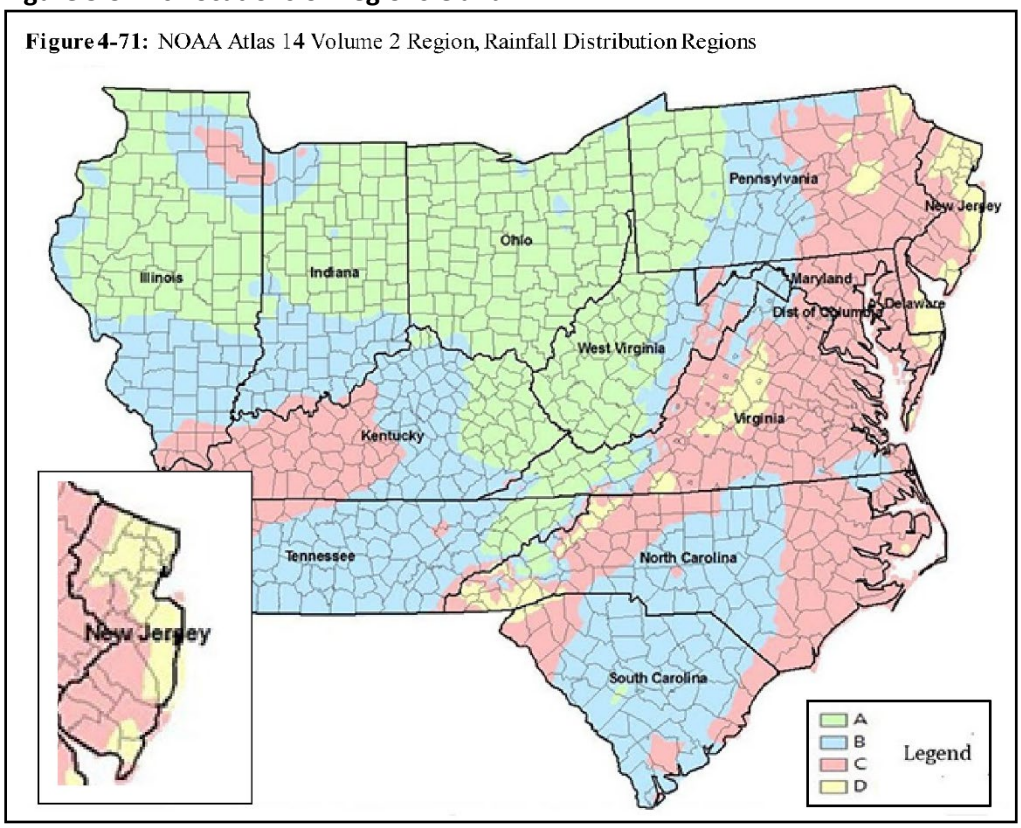
   Excel format versions of the NOAA_C and NOAA_D rainfall distributions, which are derived from the data provided at the above link, are available online from the Department's website, under the heading for *Chapter 5*, via the following link:

   https://dep.nj.gov/wp-content/uploads/stormwater/bmp/noaa_c_d_depth_intensity_june_2020.xlsx

It should be noted that the extents of Regions C and D were updated in the *NEH* published August 2019, as shown in Figure 5-9. This figure is a reprint of Figure 4-71 in Chapter 4 of the *NEH* and is found online at the link below:

https://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=43924.wba.

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



The updated extents of Regions C and D shown in Figure 5-9, and either shaded in orange or yellow, indicate that both Rainfall Distribution Regions C and D may not follow the county boundaries as stated in the NEW JERSEY BULLETIN NO. NJ210-12-1. If a project is near a rainfall distribution region boundary, it might be difficult to identify the respective rainfall distribution region from Figure 5-9. In such a case, the designer is encouraged to contact the Department to help determine the appropriate distribution.

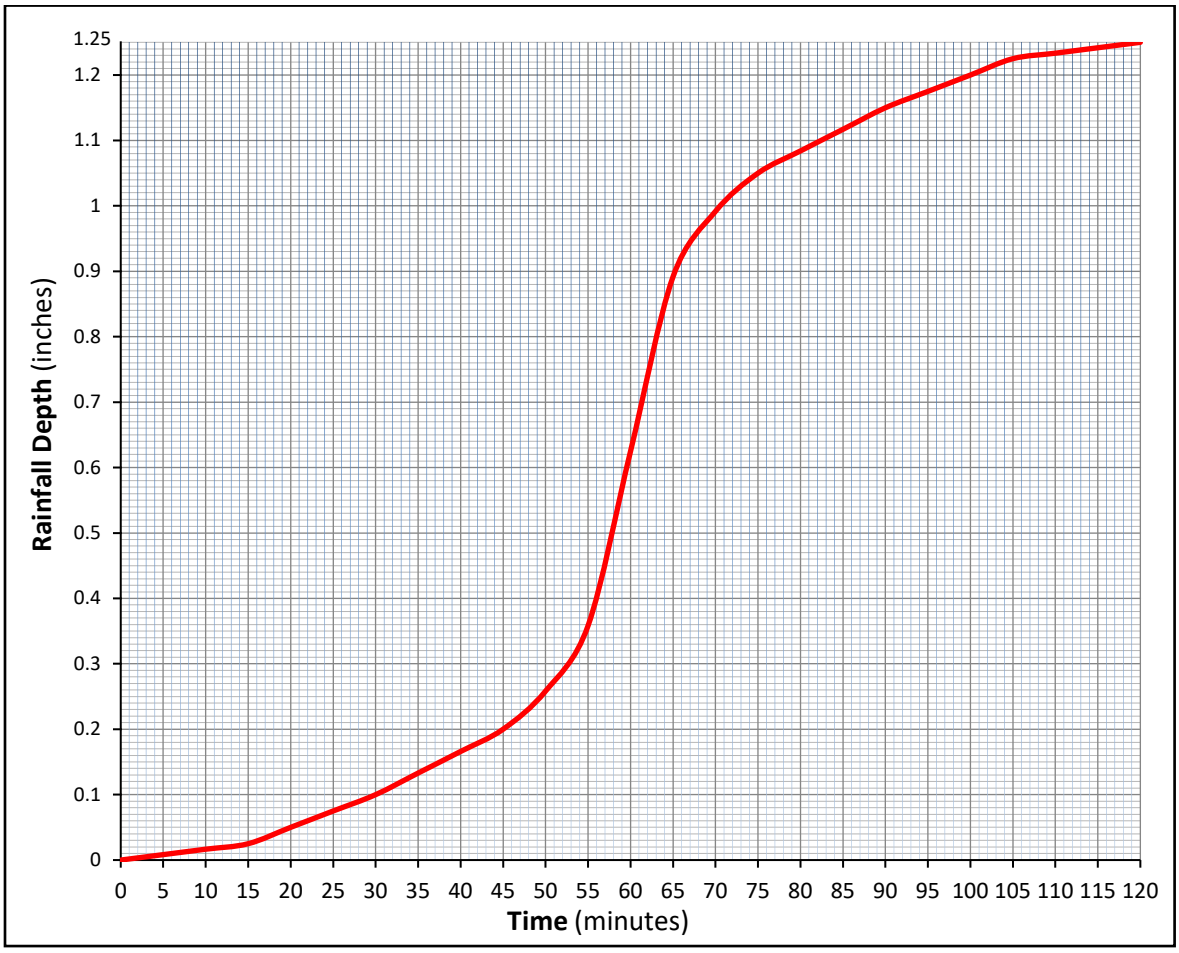
6. **Rainfall Depth for the Stormwater Runoff Water Quality Design Storm**: For stormwater runoff quality control, N.J.A.C. 7:8-5.5 requires using 1.25 inches of rain falling nonuniformly in a 2-hour storm event, which is also known as the Water Quality Design Storm (WQDS).

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, provided below. 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.

The accumulative distribution curve for rainfall depth, shown 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



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:

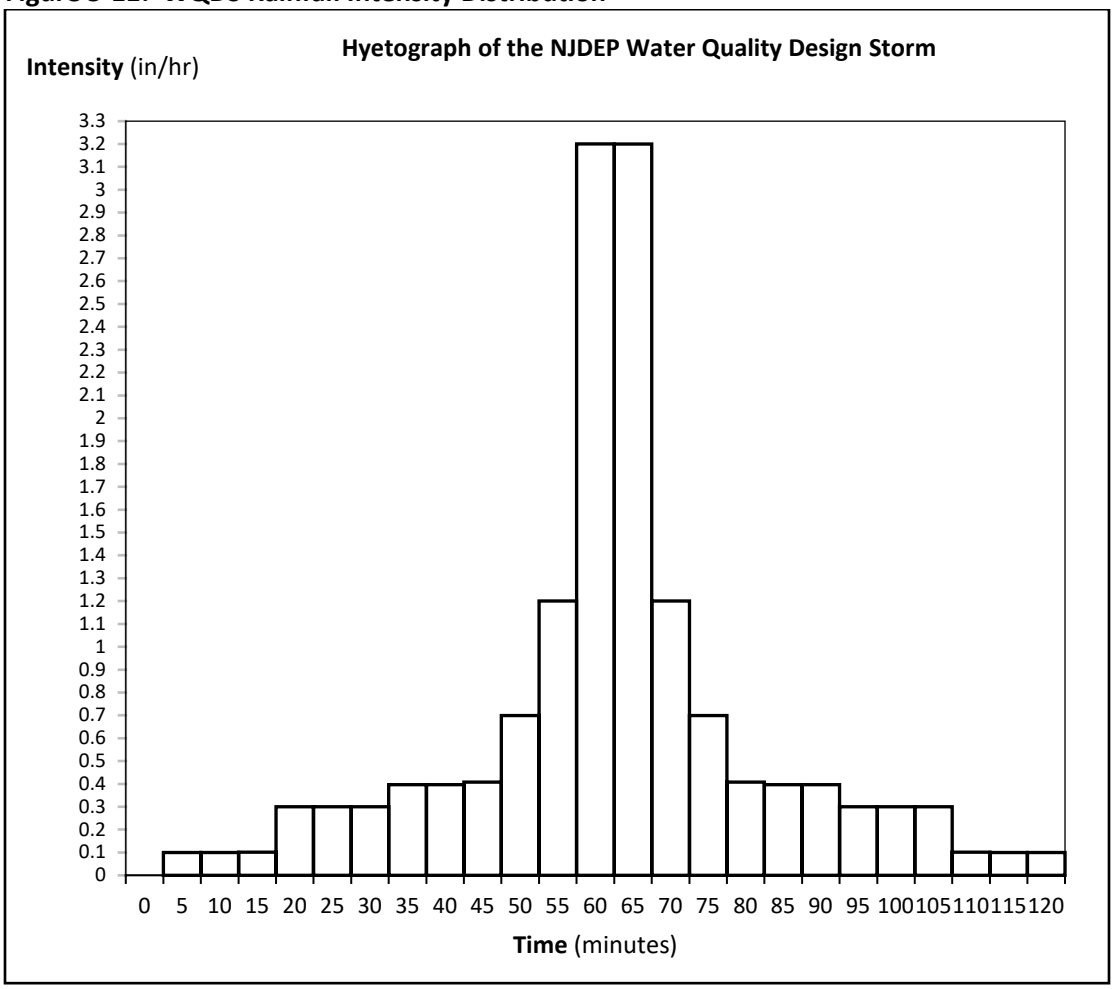
https://www.njstormwater.org/bmp_manual2.htm

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

Figure 5-11 depicts 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. Table 15-1 is reprinted below as Figure 5-12.

Figure 5-12: Manning's Roughness Coefficients for Sheet Flow

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.

- **Sheet Flow Length:**
  - ☐ **For the pre-construction condition,** the user assumes a sheet flow length of 100 ft. Only when there is something physically in contact with the flow of stormwater runoff to prevent sheet flow from occurring, such as a swale, curb or inlet, shall the value used be less than 100 ft.
  - ☐ **For the post-construction condition**, the maximum sheet flow length should not exceed the McCuen-Spiess limitation or 100 ft, whichever is shorter. The McCuen-Spiess limitation is calculated as follows:

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

where S is the slope, in ft/ft, and n is the Manning's roughness coefficient for sheet flow**.**

- **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:

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

where:

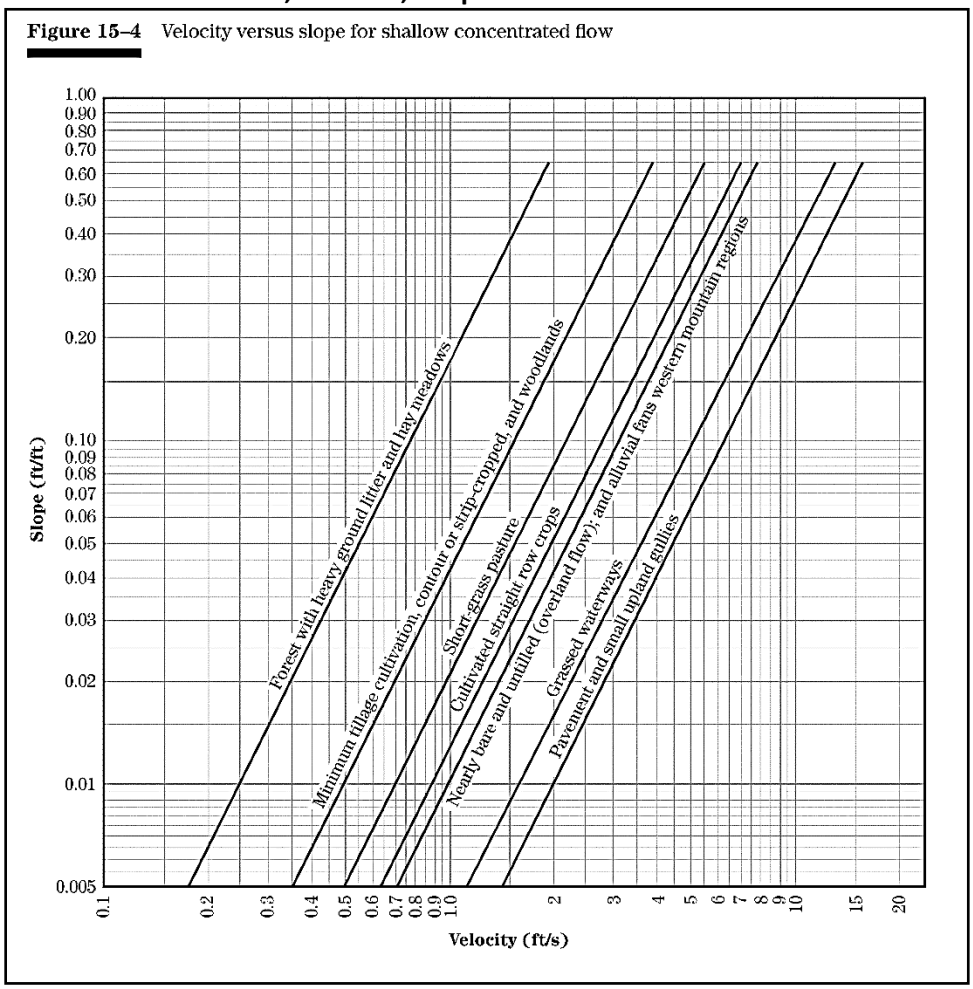
T_t = travel time, hr
n = Manning's roughness coefficient for sheet flow
L = sheet flow length, ft, See Sheet Flow Length discussed above
P_2 = 2-year, 24-hour rainfall, in, which is the current or projected 2-year storm event (see guidance below)
S = slope of land surface, ft/ft

- **Calculating the rainfall depth for the 2-year storm event used in travel time calculations.** Example 5-1, which begins on Page 33, illustrates the calculation:

  - ☐ When calculating the travel time for current storm events, use the precipitation depth for the current 2-year storm event, which is calculated by multiplying either the NOAA NWS PFDS rainfall depth or the NRCS county rainfall data for the 2-year storm by the current precipitation adjustment factors.
  - ☐ When calculating the travel time for projected storm events, use the precipitation depth of the projected 2-year storm event, which is calculated by multiplying either the NOAA NWS PFDS rainfall depth or the NRCS county rainfall data for the 2-year storm by the future precipitation change factors.

- **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. The value for the flow velocity can be determined from the graphical source, reprinted as Figure 5-13 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. The velocities plotted in each are average values and are a function of watercourse slope and the cover condition of the channel. To use this figure, find the slope on the y-axis, noting it is a logarithmic in scale, and project a horizontal line over to the line which best fits the cover present for this segment of flow. Then draw a vertical line to the x-axis and read the velocity, taking note that the x-axis is also logarithmic in scale.

Figure 5-13: Velocity Versus Slope for Shallow Concentrated Flow From NEH, Part 630, Chapter 15



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:

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

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 33.

- **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

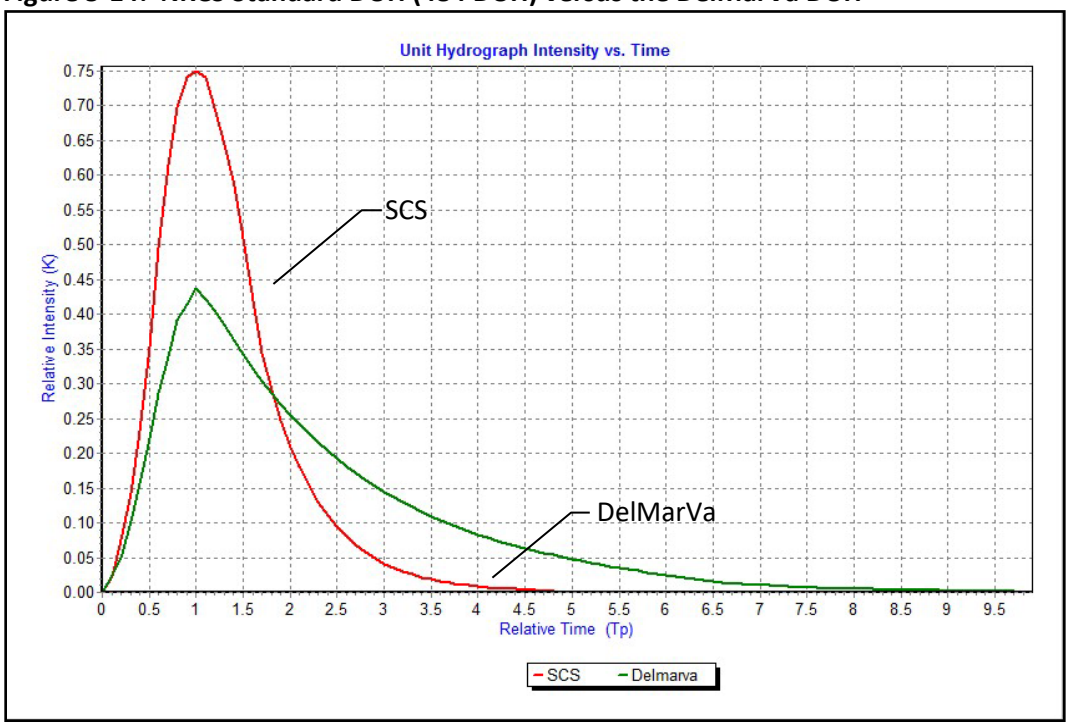
flow constrictions and ponding areas without considering the peak flow and time attenuation effects of such areas, meaning the flow must be routed as a pond. As noted in the NJDEP Stormwater Management rules, such areas can occur at hedgerows, undersized culverts, fill areas, sinkholes and isolated ponding 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 methodology 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 5-14, found on the following page, illustrates the differences between the 484 DUH and the DelMarVa DUH.

Figure 5-14: 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 5-15, which may be found on the next page, and GIS data is available online from NJDEP's Bureau of Geologic Information Systems at:

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

Alternatively, the NJDEP Bureau of GIS website allows the user to locate an address to obtain further information. Use the link provided below:

https://gisdata-njdep.opendata.arcgis.com/datasets/physiographic-provinces-of-new-jersey/

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 application of the DelMarVa DUH is required.

Figure 5-15: Physiographic Provinces of NJ

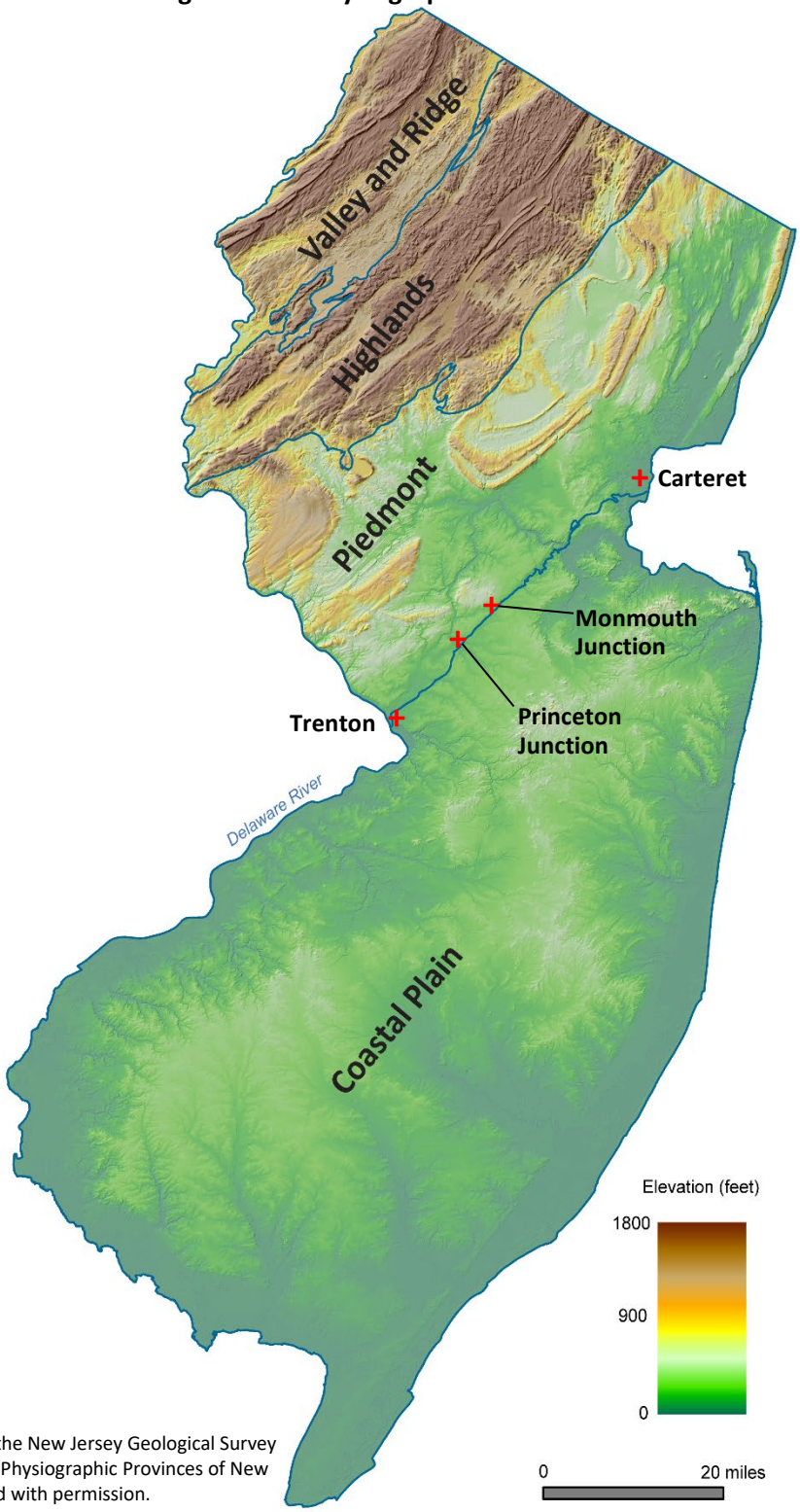


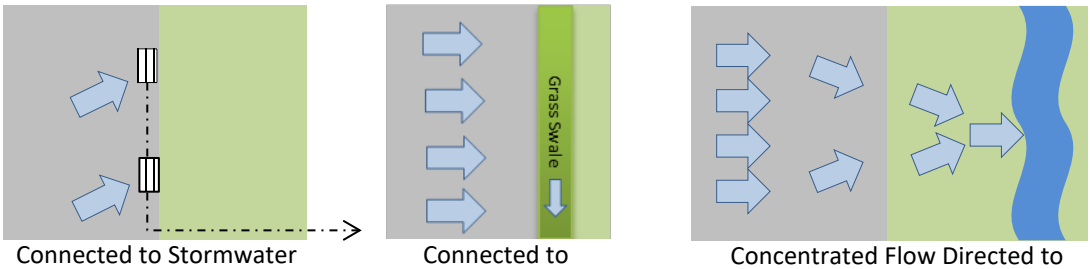
Image modified from the New Jersey Geological Survey Information Circular, "Physiographic Provinces of New Jersey, 2006" and used with permission.

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 5-16: 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

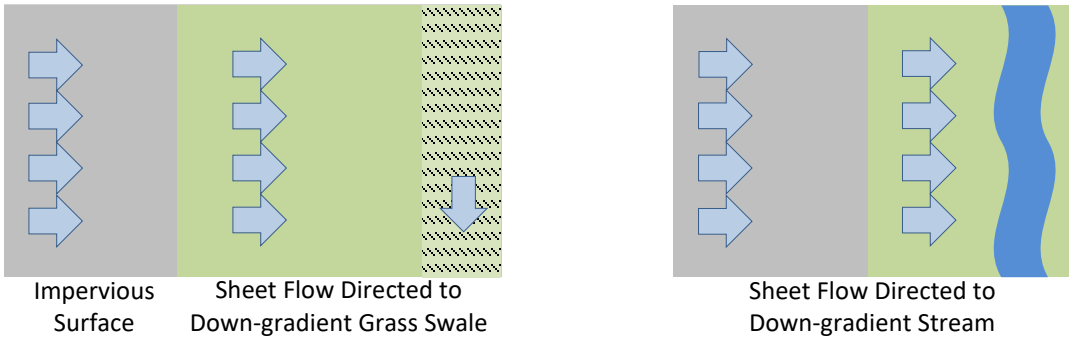
- collected by a storm drain, which then connects to a conduit or channel to a downstream BMP, stormwater collection system or stream or
- flowing over a pervious surface by shallow concentrated flow or channelized flow and then into a channel to a down-gradient stream or other flowing water body.

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 5-17: 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 combined total area of the impervious surface and the receiving down-gradient pervious surface. If the total impervious area is equal to or greater than 30 percent, the NRCS composite CN with unconnected impervious area method cannot be used because the absorptive capacity of the remaining 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 36.

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.

13. **Calculating the Peak Flow Rate of the WQDS:** The NRCS methodology must be used when calculating the WQDS. The rainfall depth of the 2-year storm, P_2, used in the calculation of the time of concentration for the WQDS shall be based upon using the projected 2-year storm event.

Examples Using the NRCS Methodology

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-construction condition hydrographs for current and projected rainfall precipitation depths produced by a project in which impervious cover is reduced. Take note Examples 5-6 and 7, which begin on Page 47, 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 5-8 illustrates the use of current and projected rainfall depths in the design calculations for a 2- acre major development project with GI BMPs. The methods used in this example would also be followed for sizing a small-scale GI BMP to provide the required pre-treatment for a subsurface small-scale infiltration basin. Finally, Example 5-9 depicts the

process for calculating the peak flow rate for the runoff produced by the WQDS falling on 0.25 acres of regulated motor vehicle surface to be treated by a Manufactured Treatment Device.

Example No.	Scenario Description	Page No.
5-1	Calculate of Time of Concentration	33
5-2	Use the NRCS CN Method for an Unconnected Impervious Surface to Calculate the Runoff Volume for a Site	36
5-3	Use the NJDEP Two-Step Method for an Unconnected Impervious Surface to Calculate the Runoff Volume for a Site	38
5-4	Demonstration of Why a Composite CN Generates an Incorrect Runoff Volume	39
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	41
5-6	A Re-development Project with Two Drainage Areas, Each Discharging to Separate Points,	47
5-7	The Same Re-development Project with Two Drainage Areas, having One Combined Discharge Point	68
5-8	A 2- Acre Development Project with GI BMPs Illustrating the Use of Current and Projected Storm Events in the Design Calculations to address the Water Quality, Runoff Quantity Control and Groundwater Recharge Requirements	71
5-9	Calculate the Volume and Peak Flow Rate for the Runoff Produced by the WQDS Falling on 0.25 Acres of Regulated Motor Vehicle Surface to be Treated by a Manufactured Treatment Device	83

Example 5-1: Calculate Time of Concentration

For a site located in Mercer County, 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 using current rainfall data.

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, to which the current precipitation adjustment factor has been applied

S = slope of land surface, ft/ft

The maximum sheet flow length is calculated by using the formula from the McCuen-Spiess limitation calculated below or substituting 100 ft for the maximum length if the result calculated by the McCuen-Spiess limitation is greater than 100 ft:

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

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 also reprinted on Page 24. 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.

The NOAA 2-year 24-hour rainfall depth, was obtained in Example A-1, and is 3.33 in. In Example B-1, the current rainfall of 3.36 in was calculated by applying the 2-year precipitation adjustment factor to the NOAA rainfall. Although not shown in this example, the projected rainfall depth for the projected 2-year storm is calculated by multiplying the NOAA data by the future precipitation change factor for the projected 2-year storm.

Using the McCuen-Spiess limitation, the maximum 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, as shown below. If instead of using the maximum sheet flow length of 17.68 ft calculated above, 15.00 ft is used because it reflects the actual conditions on the site conditions, the travel time for sheet flow is calculated to be 8.00 minutes, as shown below.

$$T_t = \frac{0.007[(0.40)(15.00)]^{0.8}}{(3.36)^{0.5}(0.005)^{0.4}}$$

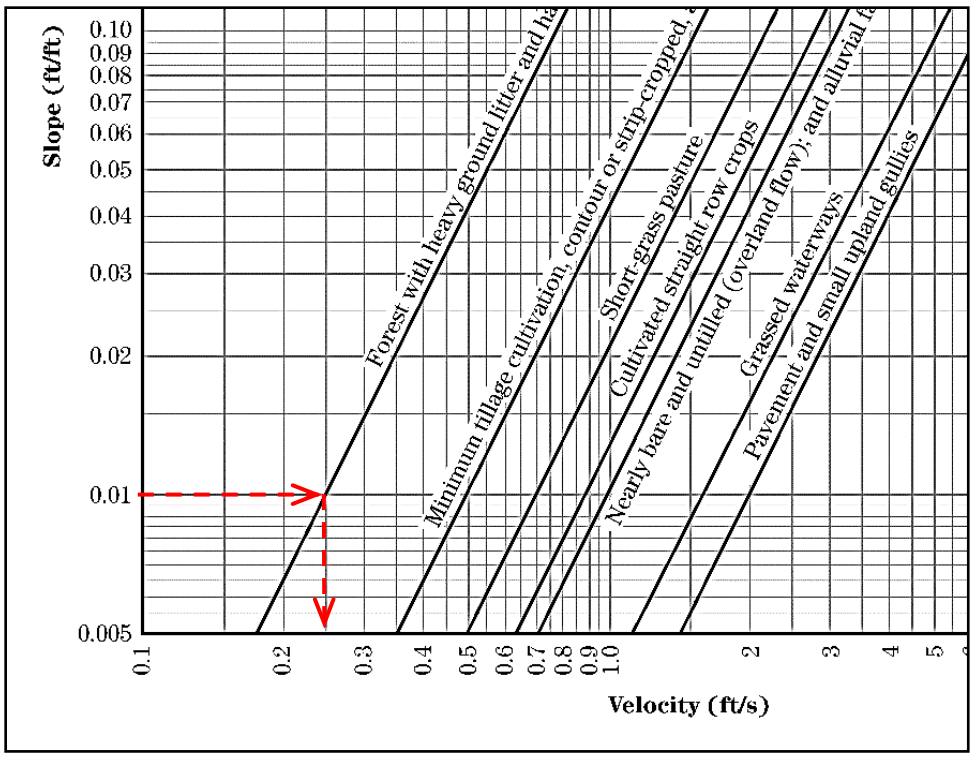
$$= 0.133 \text{ hr} = 8.00 \text{ min}$$

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 15.00 ft, the length of the shallow concentrated flow segment is determined to be 985.00 ft. The value for the flow velocity can be determined from Figure 15-4 in the *NEH*, Part 630, Chapter 15. 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:

$$T_t = \frac{985.00}{0.25\ x\ 3600} = 1.09 \text{ hr} = 65.5 \text{ min}$$

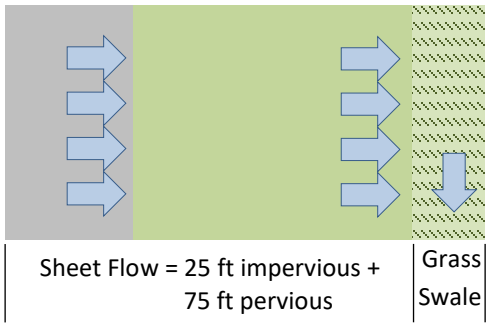
Step 3: Since no channel flow is specified in the example, the time of concentration for the post-construction condition is the sum of the travel times under sheet flow and shallow concentrated flow, as follows:

$T_c = 8.00 + 65.5 = 73.5$ min, using Figure 15-13 for a shallow concentrated flow length of 985.00 ft and a sheet flow length of 15.00 ft.

Take note that the time of concentration for the projected 2-year storm similarly follows these steps but uses the future precipitation change factor for the projected 2-year storm to calculate the projected rainfall depth.

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 illustrated is the current 2-year storm, in which the current rainfall depth is 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 calculate 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 32), the NRCS composite CN with unconnected impervious area method is applicable.

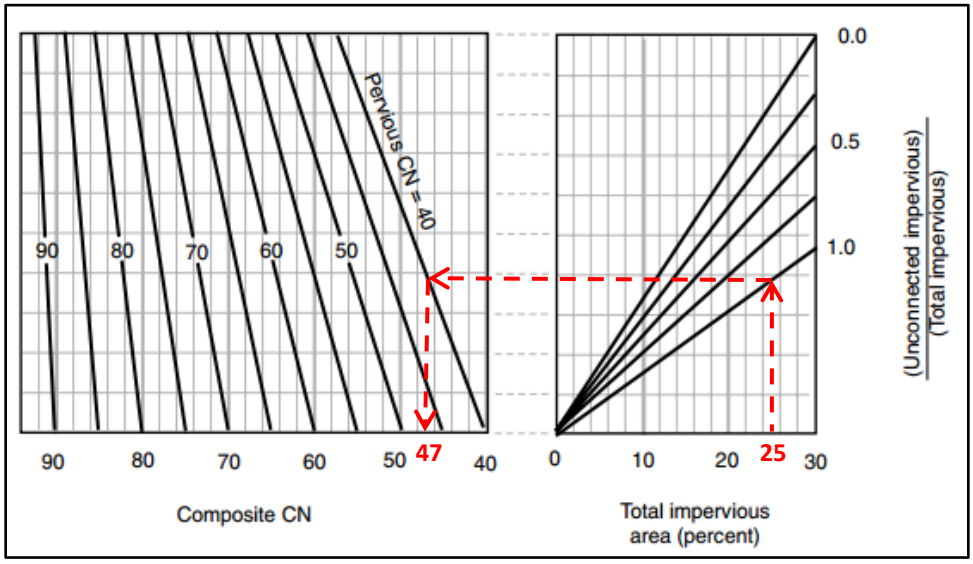
Step 2: Calculate the Ratio of Unconnected Impervious Surface to Total Impervious Surface

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 depth of 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, the depth of runoff generated by the 2-year storm is calculated as follows:

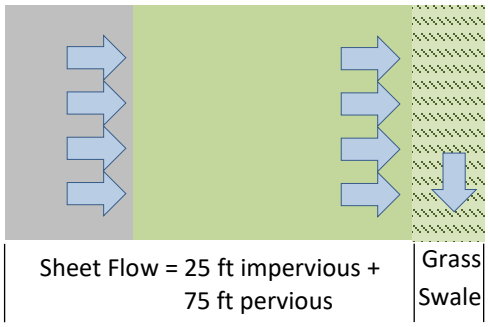
$$Q = \frac{(3.5 - 0.2\ x\ 11.3)^2}{(3.5 + 0.8\ x\ 11.3)} = \frac{(1.24)^2}{(12.5)} = 0.123\ \text{in}$$

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. The design storm event illustrated is the current 2-year storm, in which the current rainfall depth is 3.5 inches

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 30 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:

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

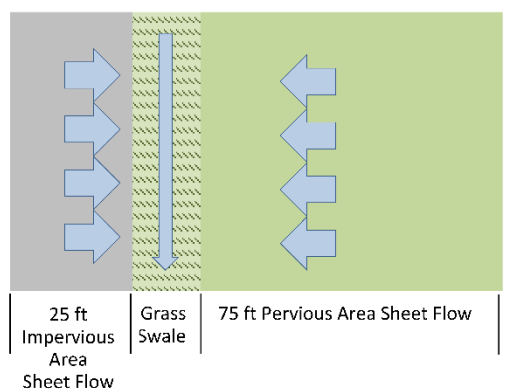
$$Q = \frac{(4.59 - 0.2\ x\ 15.64)^2}{(4.59 + 0.8\ x\ 15.64)} = 0.125 \text{ in}$$

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:

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

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

$$Q = \frac{(3.5 - 0.2\ \ x\ 8.60)^2}{(3.5 + 0.8\ x\ 8.60)} = 0.305\ 245 \text{ in}$$

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 Page 38).

For the pervious surface,

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

$$Q = \frac{((3.530\ x\ 1.00) - 0.2\ \ x\ \ 15.64)^2}{((3.530\ x\ 1.00) + 0.8\ \ x\ \ 15.64)} = 0.009 \text{ in}$$

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

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

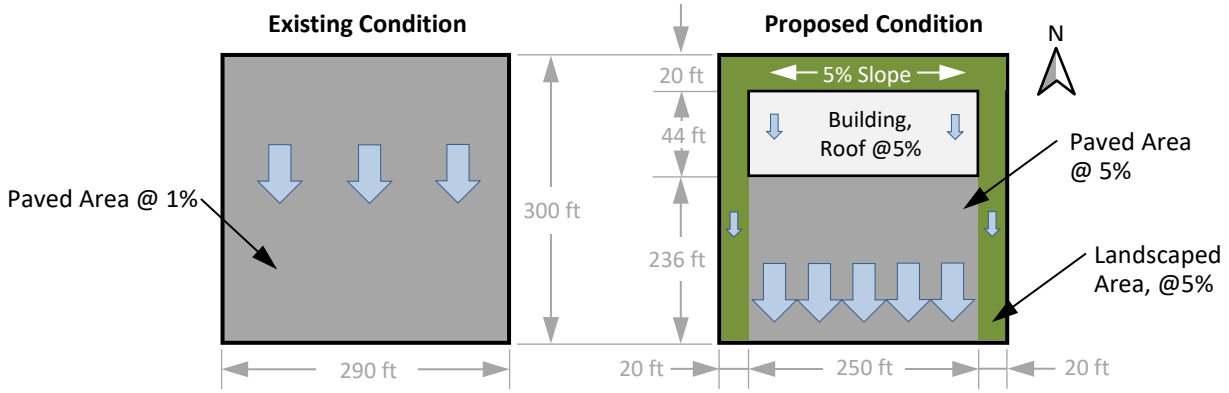
Adding these separately calculated volumes together yields the total runoff volume entering the grass swale equal to 1,373.3 cf. The previous, i.e. composite, calculation is only 37% of this volume.

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. 7:8-5.6(b)1 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 current and projected two-, 10- and 100-year storm events, as defined and determined pursuant to N.J.A.C. 7:8-5.7(c) and (d), respectively, 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.

For a site with a latitude of 40.2333° and a longitude of -74.7667°, located in Trenton, Mercer County, an approximately 2 acre paved parking lot, i.e., measuring 300 ft north to south and 290 ft east to west, is to be redeveloped as an office complex consisting of a new building measuring 44 ft x 250 ft, a parking lot measuring 236 ft by 250 ft and perimeter landscaped areas 20 ft in depth, measured in from the lot boundary lines. The existing lot has a slope of 1% from the north edge of the lot to the south edge of the lot, and the runoff under existing conditions is as overland flow from the north side to the south side. The runoff generated by the proposed building (5% slope) 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 a 5% slope for better drainage. The landscaped area is located on the north, east and west sides of the proposed development, and each area also slopes from north down to the south. The landscaped area will not receive runoff from the impervious surfaces. HSG 'C' soils are present.



Step 1: Calculate the current rainfall depths and the projected rainfall depths for the 2-, 10- and 100-year storms for the site.

The NOAA rainfall data was obtained in Example A-1. In the following table, the respective current precipitation adjustment factors and future precipitation change factors for Mercer County are applied as required to obtain the precipitation depths for current and projected storm events.

Frequency of Storms	NOAA NWS PFDS Precipitation Depth (inches)	Current Precipitation Adjustment Factor	Current Rainfall Depth (inches)	Projected Precipitation Change Factor	Projected Rainfall Depth (inches)
2-year	3.33	1.01	3.36	1.16	3.86
10-year	4.99	1.02	5.09	1.17	5.83
100-year	8.15	1.04	8.48	1.36	11.08

Step 2: Calculate T_c for the existing and proposed conditions.

The site conditions and 2-year current rainfall depth are entered into hydrologic modelling software. 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 24. The pre-construction drainage pattern consists of sheet flow for the first 100 ft, followed by shallow concentrated flow for 200 ft. T_c is calculated to be 3.2 minutes.

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 time of concentration is calculated and shown in the table below: The sheet flow length calculated by McCuen-Spiess limitation criteria, where S is the slope, in ft/ft, and n is the Manning's roughness coefficient for sheet flow, is as follows:

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

and the calculated result exceeds 100 ft. Therefore, the sheet flow length must be limited to the maximum of 100 ft and the shallow concentrated flow length is 200 ft. However, the time of concentration is shorter due to the increased slope.

Surface Description	Slope, S, (ft/ft)	Manning's Roughness Coefficient, n, For Sheet Flow	McCuen-Spiess Limitation Sheet Flow Length (ft)	Sheet Flow Length to be used in the Model (ft)	Length of Shallow Concentrated Flow (ft)	Calculated T_c (min) for Current Rainfall Depth
Building	0.05	0.011	2,036	44	236	1.3
Landscaped Area	0.05	0.150	149	100*	345**	10.3
Parking Area	0.05	0.011	2,036	100	136	0.9

*Length = 2036 * 0.011/.150 = 149 ft; therefore, a maximum of 100 ft shall be used

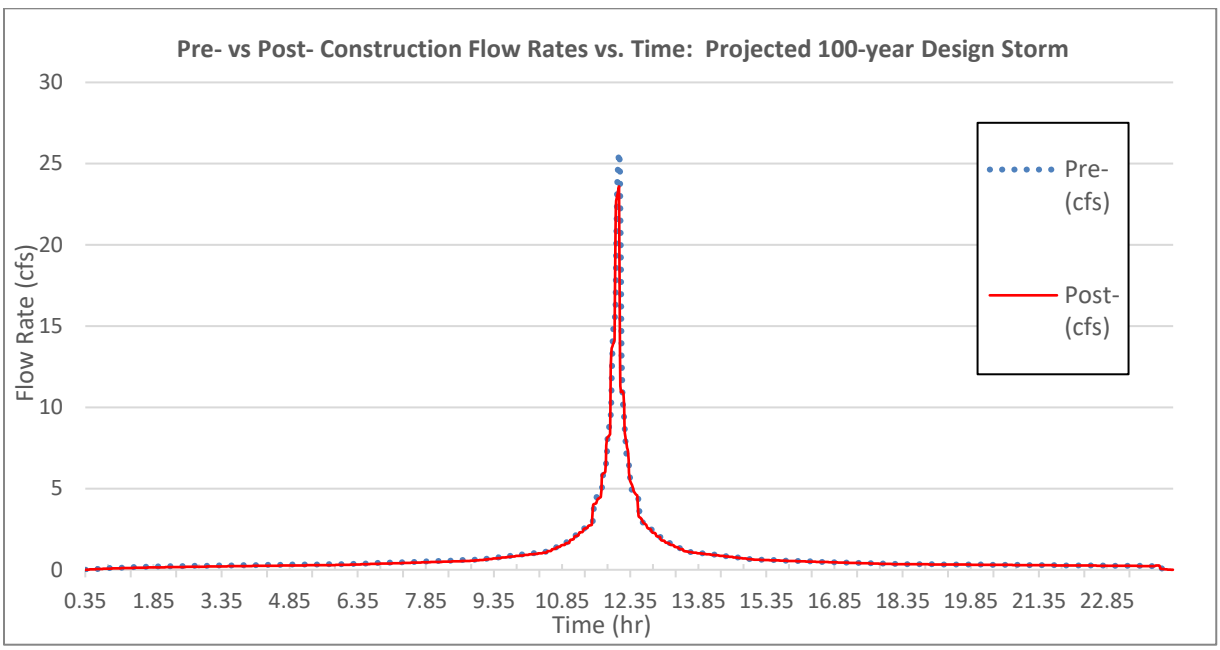
** = The half of the lot width subtracted by the sheet flow length + the lot length = 290/2 – 100 + 300 = 345 ft

Step 3: Calculate the peak flow rates for the projected 100-year storm for the pre- and post-construction conditions using the future precipitation change factor and compare the hydrographs.

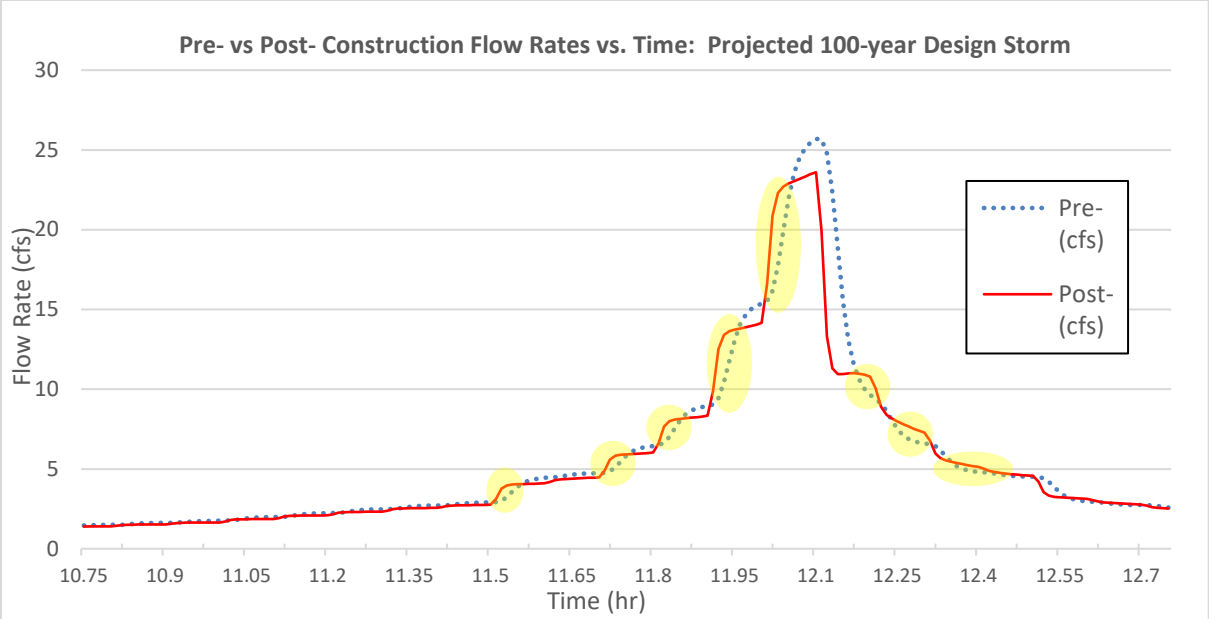
A summary of the results from the modelling software is shown in the table below:

Projected 100-year Design Storm		
Parameter	**Pre-Construction Condition**	**Post-Construction Condition**
Peak Flow Rate & Time of Peak =	25.71 cfs @ 12.10 hr	23.60 cfs @ 12.10 hr
Runoff Volume=	78,689.7 cf	75,250.3 cf

Pre- and post-condition hydrographs for the projected 100-year storm, after applying the future precipitation change factor to the NOAA NWS PFDS rainfall data, are depicted below. The hydrographs were created by exporting the hydrograph tables as CSV data and then using a spreadsheet program to graph the information.



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, e.g., starting at 11.92 hours, one would see the post-construction hydrograph has a higher flow rate than the pre-construction hydrograph in certain time periods. Some of these exceedances are highlighted in yellow in the following graphic.



A portion of the hydrograph data is also listed in the table below, for which the exceedances are highlighted in yellow. Take note this is just one example of two clusters of exceedances for this storm.

Time (hr)	Pre-construction (cfs)	Post-construction (cfs)	Difference in Flow Rate, Post - Pre, (cfs)
11.91	9.04	9.86	0.82
11.92	9.44	12.52	3.08
11.93	10.42	13.39	2.97
11.94	11.77	13.63	1.86
11.95	13.07	13.73	0.66
11.96	14.03	13.8	-0.23
11.97	14.63	13.87	-0.76
11.98	15	13.95	-1.05
11.99	15.22	14.03	-1.19
12	15.37	14.16	-1.21
12.01	15.53	16.59	1.06
12.02	16.16	20.87	4.71
12.03	17.74	22.30	4.56
12.04	19.96	22.71	2.75
12.05	22.05	22.90	0.85

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)1, 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 projected 100-year storm have already shown noncompliance, this example does not continue further to calculate

hydrographs for the projected 2- and 10-year storms. Additionally, a similar comparison for the 2-, 10- and 100-year design storms using rainfall data with the current precipitation adjustment factors applied would be required to demonstrate compliance for the current design storms.

On the other hand, if the design engineer chooses to demonstrate compliance with the quantity control requirements under N.J.A.C. 7:8-5.6(b)3, e.g., that the post-construction peak runoff rates for the projected 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 peak flow rate for the projected 100-year storm will be as follows:

	Projected 100-year Design Storm		
	Pre-Construction Peak Flow Rate	**Allowable Post-Construction (80% Reduction) Peak Flow Rate**	**Post-Construction Peak Flow Rate**
Peak Flow Rate =	25.71 cfs	20.57 cfs	23.60 cfs

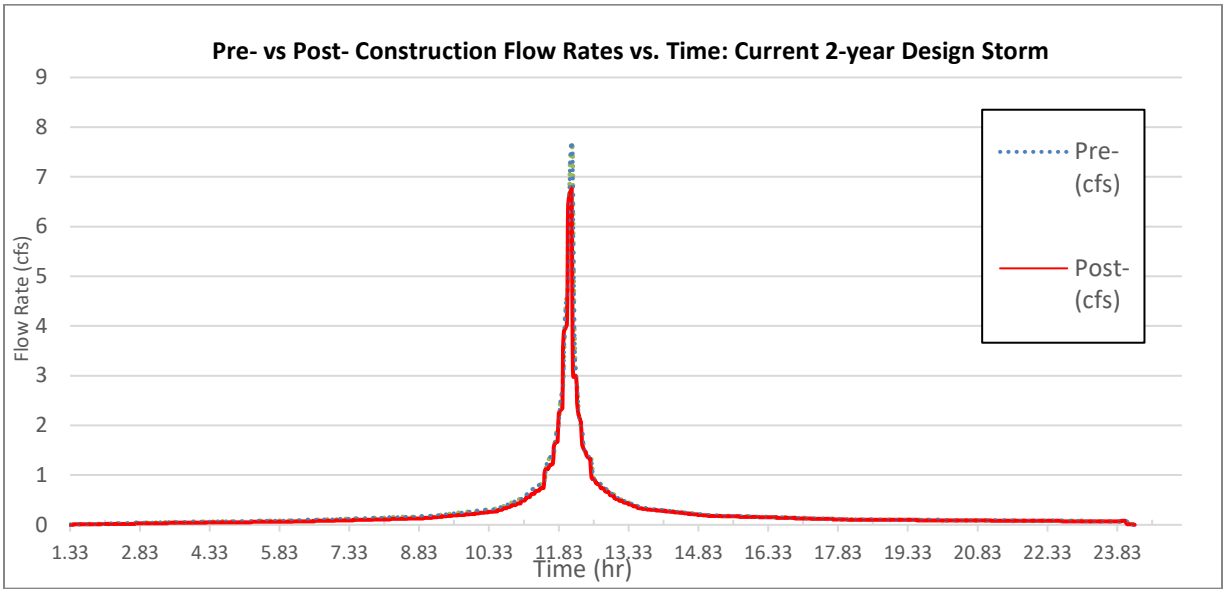
The projected 100-year post-construction peak flow rate, 23.60 cfs, exceeds the allowable flow rate, 20.57 cfs. Therefore, the project is still not in compliance with the stormwater runoff quantity control requirement under the option found in N.J.A.C. 7:8-5.6(b)3. A stormwater runoff quantity control 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.

Step 4: Calculate the peak flow rates for other design storms for the pre- and post-construction conditions and compare the hydrographs

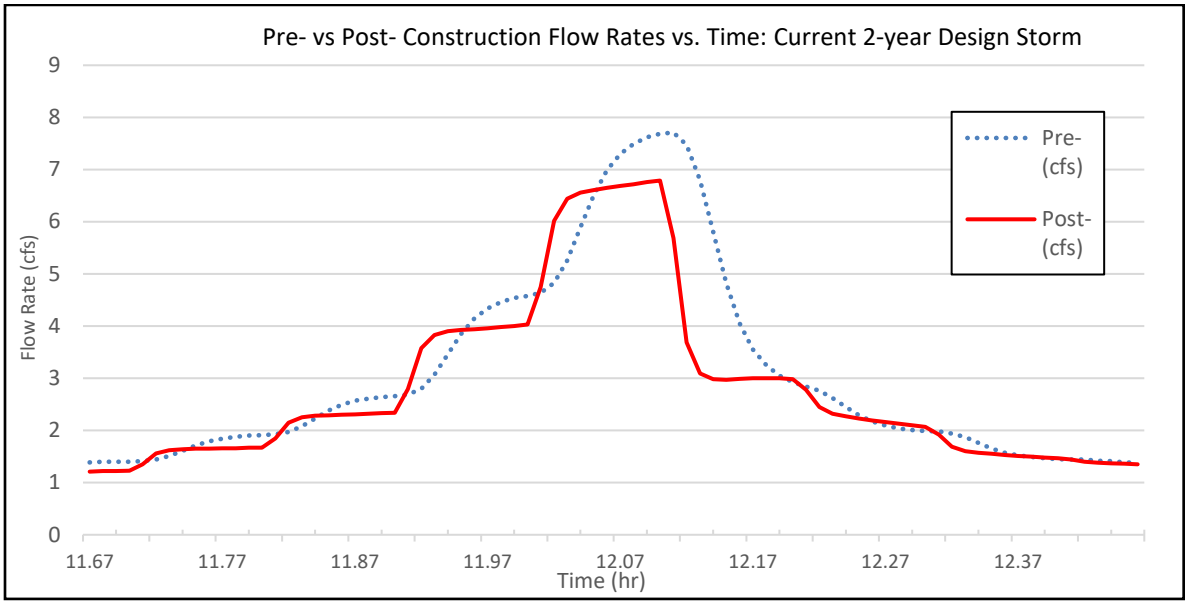
In *Step 3*, the comparison of hydrographs for the pre- and post- projected 100-year design year storms has shown that a reduction of the impervious surface or runoff volume in a redevelopment project may not comply with N.J.A.C. 7:8-5.6(b). The rules require the demonstration of compliance with the requirements in N.J.A.C. 7:8-5.6(b) for the current and projected 2-, 10-, and 100-year storms. Using the same technique in Step 3, a hydrograph comparison of the pre- and post- current 2-year storms, for example, is shown in the table below. To correctly calculate the time of concentration, first P_2 was calculated by multiplying the NOAA NWQ PFDS rainfall depth for the 2-year storm (3.33 in) with the current precipitation adjustment factor (1.01) and is equal to 3.36 in. Next, the modeling software is used to produce the hydrographs, with the peak flow rates and time intervals exported as data for input into a spreadsheet to create a chart. Here is the summary information.

Current 2-year Design Storm		
Parameter	**Pre-Construction Condition**	**Post-Construction Condition**
Peak Flow Rate & Time of Peak =	7.72 cfs @ 12.11 hr	6.79 cfs @ 12.10 hr
Runoff Volume=	22,670 cf	20,507 cf

Pre- and post-condition hydrographs are provided in the chart below. As with those for the projected 100-year storm from *Step 3*, they appear to be nearly identical.



However, an analysis of the data shows several timeframes of data points where the post-construction peak flow rate exceeds the pre-construction peak flow rate. Shown below is a close up of a portion of the hydrograph with some of those time periods of exceedances.



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 the two examples below, 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. Note that in these two examples, the rainfall depths stated are assumed to be those after the current precipitation adjustment factors have been applied. Although not covered here, the method and analysis would be similar when using the future precipitation change factors, after the future precipitation change factors are 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 47, 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 HydroCAD® are each shown on the following pages for the 2-, 10- and 100-year design storms. A designer may use other hydrologic - hydraulic modeling software to perform the routing calculations.

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      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'   1.50 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      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'   1.50 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, 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.

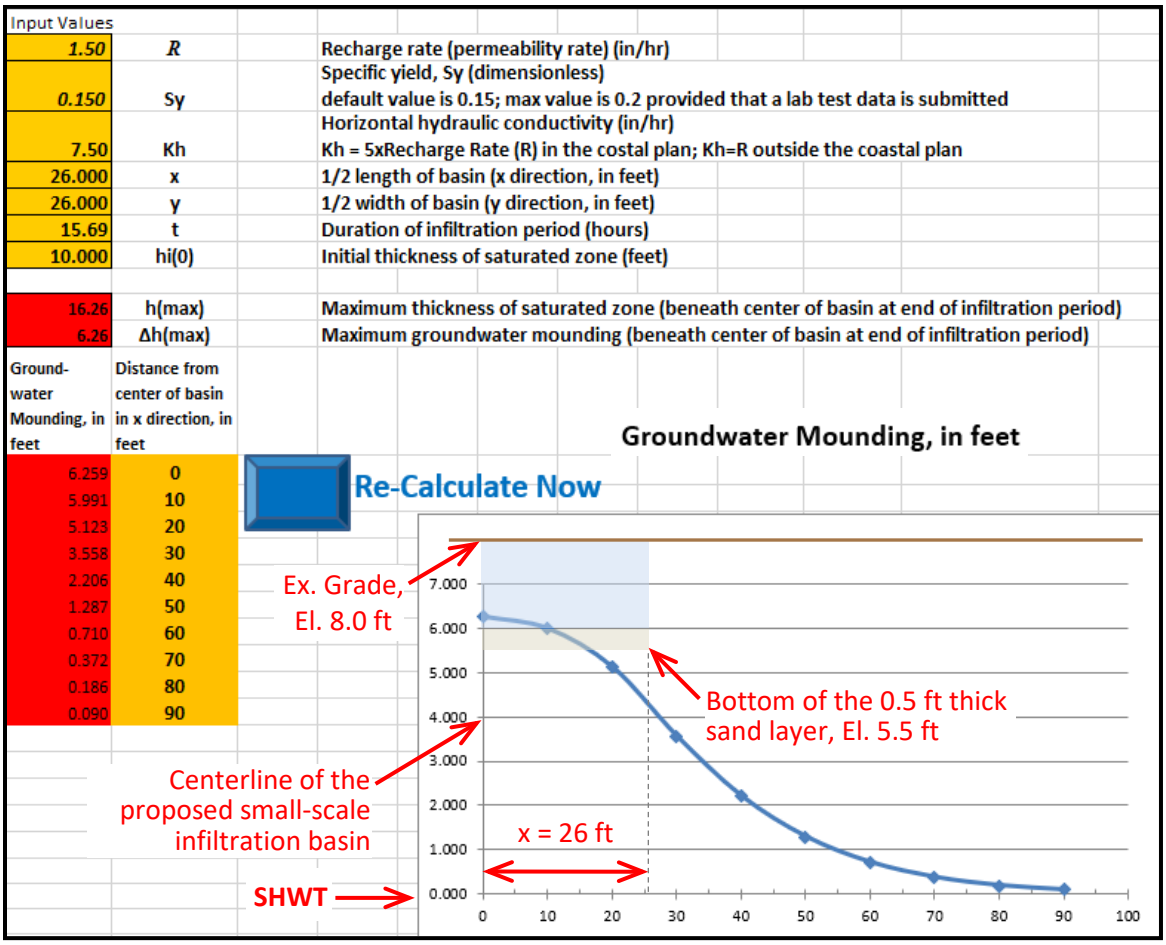
For the calculation of the duration of the infiltration period - *when exfiltration is used in the basin routing calculation* - the time calculated must be determined from the volume of runoff exfiltrated, or discarded, typically found in the modeling software results. As shown in the preceding summary 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)
```

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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. 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.

  i. 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)
```

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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.

ii. 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.

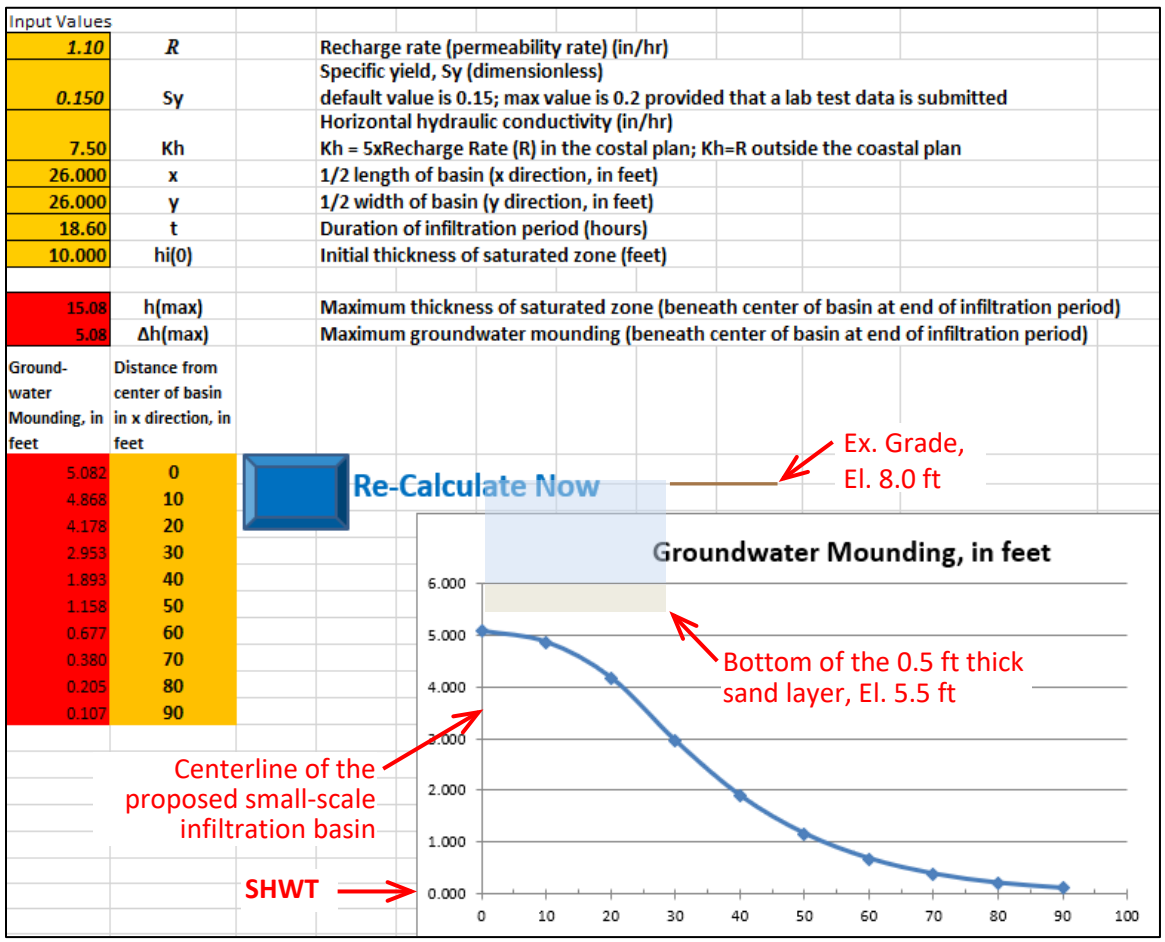
$$\text{Duration of Infiltration period, t } (hr) = \frac{Discarded\ Volume\ via\ Exfiltration\ (cf)\ x\ 12\ in/ft}{Infiltration\ area\ (sf)\ x\ Exfiltration\ rate\ (in/hr)}$$

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

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

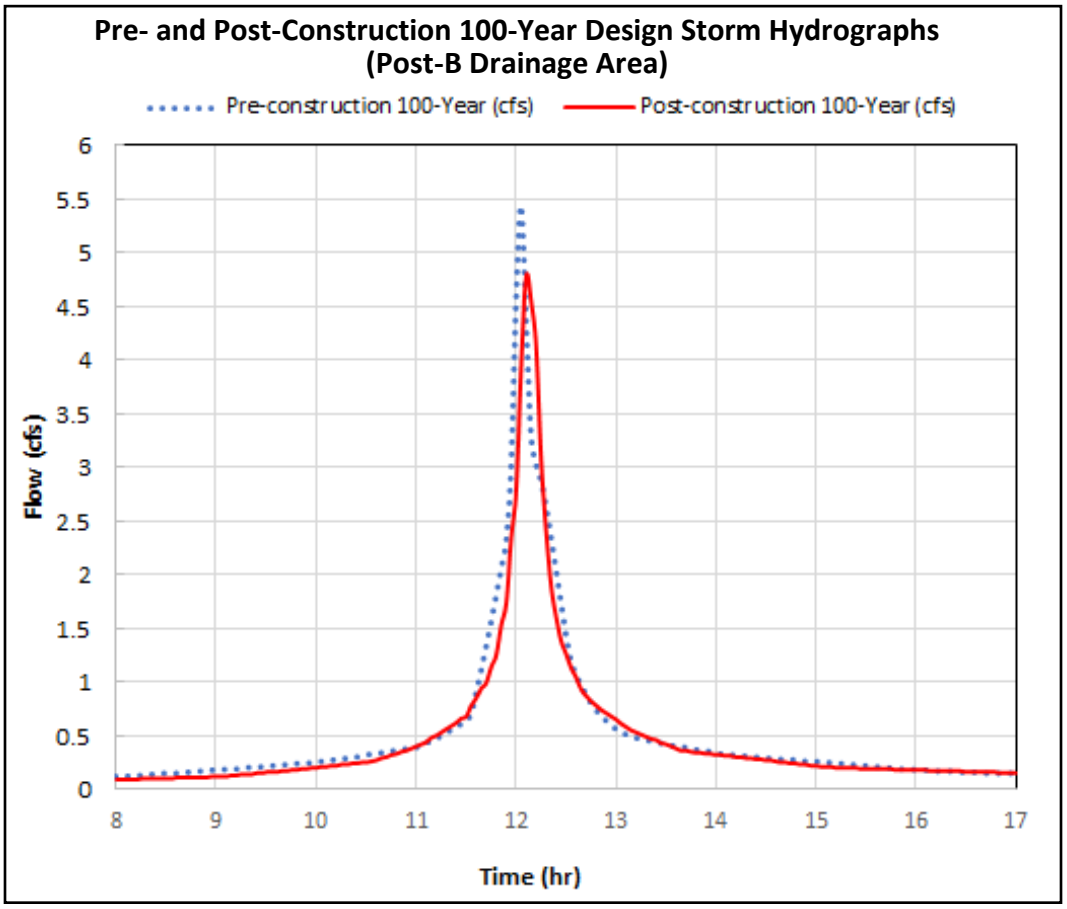
From Page 49, the 2-, 10- and 100-year design storm total peak flow rates from Post-B drainage area are 1.24, 2.41 and 4.74 cfs, respectively. The corresponding allowable design storm peak flow rates for 2-, 10- and 100-year design storms are 0.92, 2.27 and 4.30 cfs. Therefore, Post-B drainage area does 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	1.10	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)

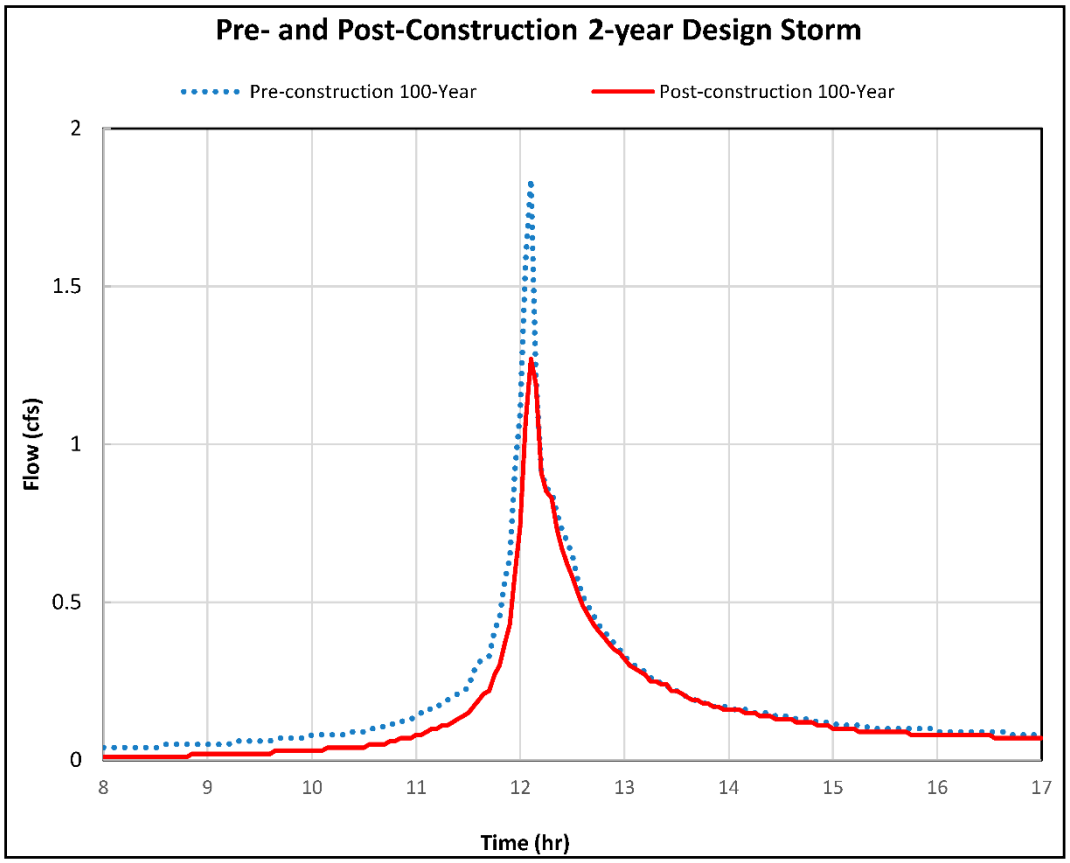
The question here is whether stormwater runoff from drainage areas Post-A and Post-B can be combined together to meet the Stormwater Runoff Quantity Control requirements either under N.J.A.C. 7:8-5.6(b)3 or N.J.A.C. 7:8-5.6(b)1. The table below illustrates combining post-construction flow rates from 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 of 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 visually compared in a plot, 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 appears to not exceed the pre-construction hydrograph.



Furthermore, **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.

Moreover, 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 within 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. Additionally, 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.

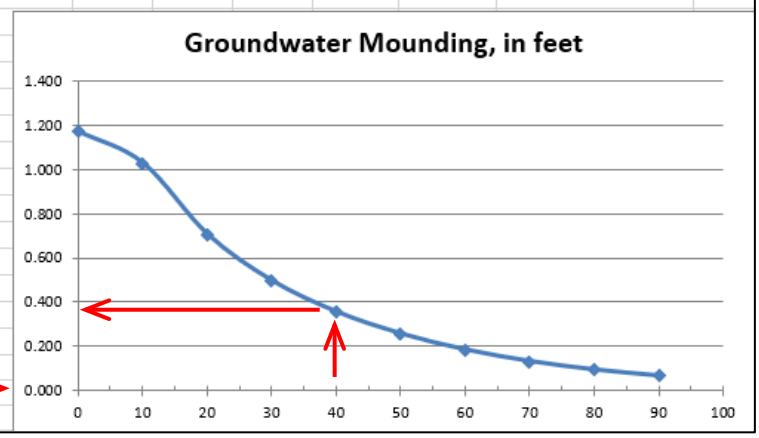
The duration of infiltration period of each basin during the 100-year storm is calculated by using the exfiltration (discarded) volume, 967 cf, the footprint of the basin, 550 sf, and the exfiltration rate, 0.5 in/hr. The result is 42.20 hr.

The results, depicted on the following page, show that the maximum height of the groundwater mounding, Δh(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
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

Re-Calculate Now



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*.

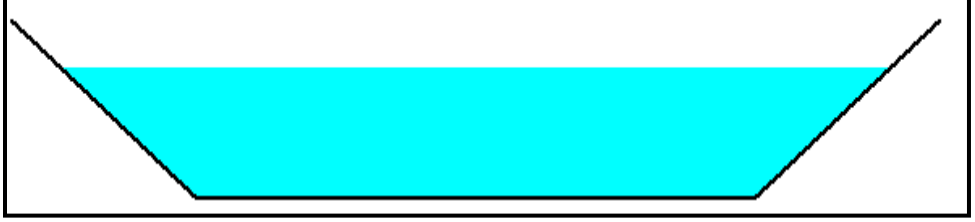
Grass Swale Summary Report for the 100-year Design Storm

```
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'
```

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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'
```

‡

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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**

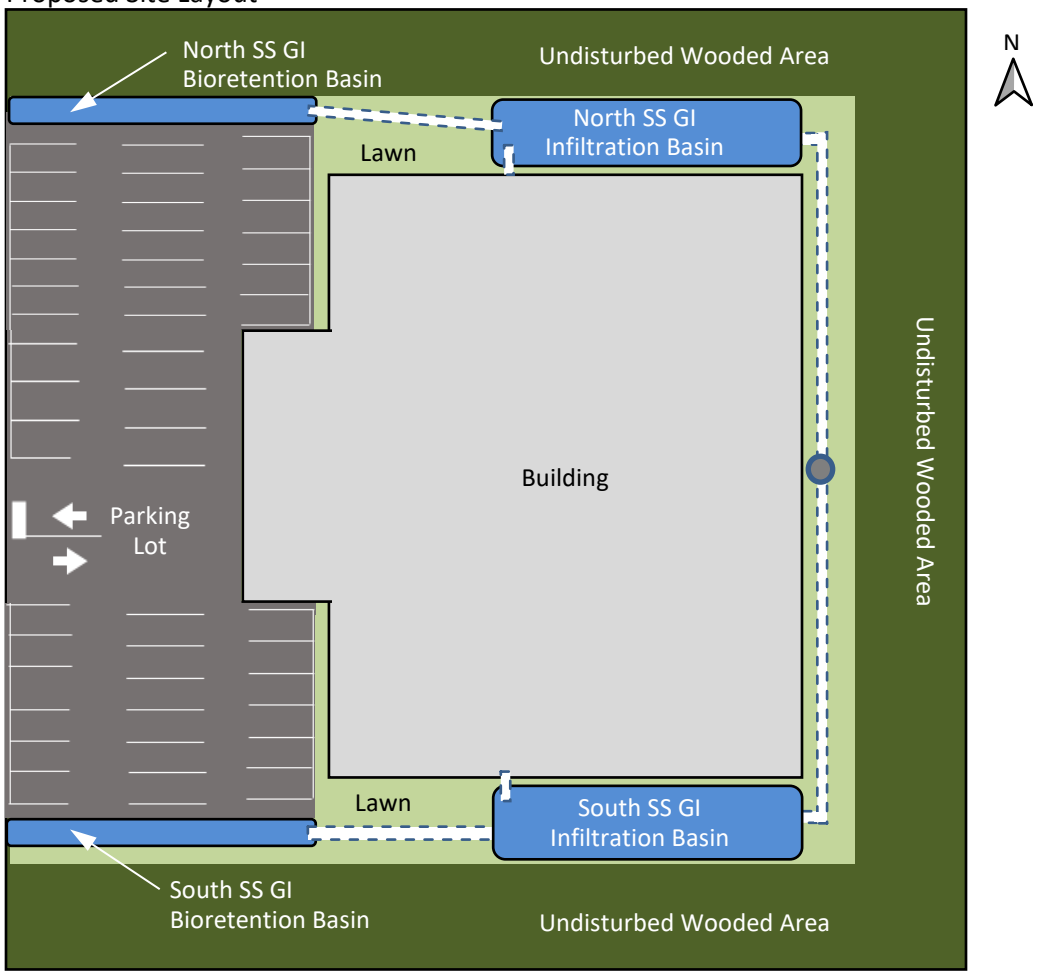
As the results show in the table above, each of the combined post-construction peak flow rates for the 2-, 10- and 100-year design storms are less than the respective allowable design storm peak flow rates. Therefore, the site as a whole meets the stormwater runoff quantity standards of N.J.A.C. 7:8-5.6(b)3.

Example 5-8: A 2-Acre Development Project with GI BMPs Illustrating the Use of Current and Projected Rainfall Data in the Design Calculations

A 2.5-ac7re wooded lot is located in Somerset County. The latitude and longitude of the site are 40.4573° and -74.5309°, respectively. The proposed development consists of a one-acre office building, a 0.5-acre parking lot, and a 0.5-acre lawn area, plus 0.5 acres of existing woods to remain undisturbed. Design a stormwater management system to meet the Stormwater Management rules. The soil present onsite is HSG 'B.'

In accordance with N.J.A.C. 7:8-5.2(a) and 5.3, green infrastructure BMPs must be incorporated in the stormwater management design to meet the design and performance standards of the Stormwater Management rules for groundwater recharge, stormwater runoff quantity control and water quality. The proposed stormwater management measures consist of two proposed small-scale green infrastructure bioretention BMPs designed to infiltrate ("SSGI bioretention BMPs") to provide water quality treatment and groundwater recharge for runoff from the parking lot and two small-scale green infrastructure infiltration BMPs ("SSGI infiltration BMPs") to provide water quantity control and groundwater recharge for the entire site. A proposed site layout and stormwater routing diagram are shown below; however, the drawing is not to scale.

Proposed Site Layout



Note: Not to Scale

A few site-specific parameters are highlighted below:

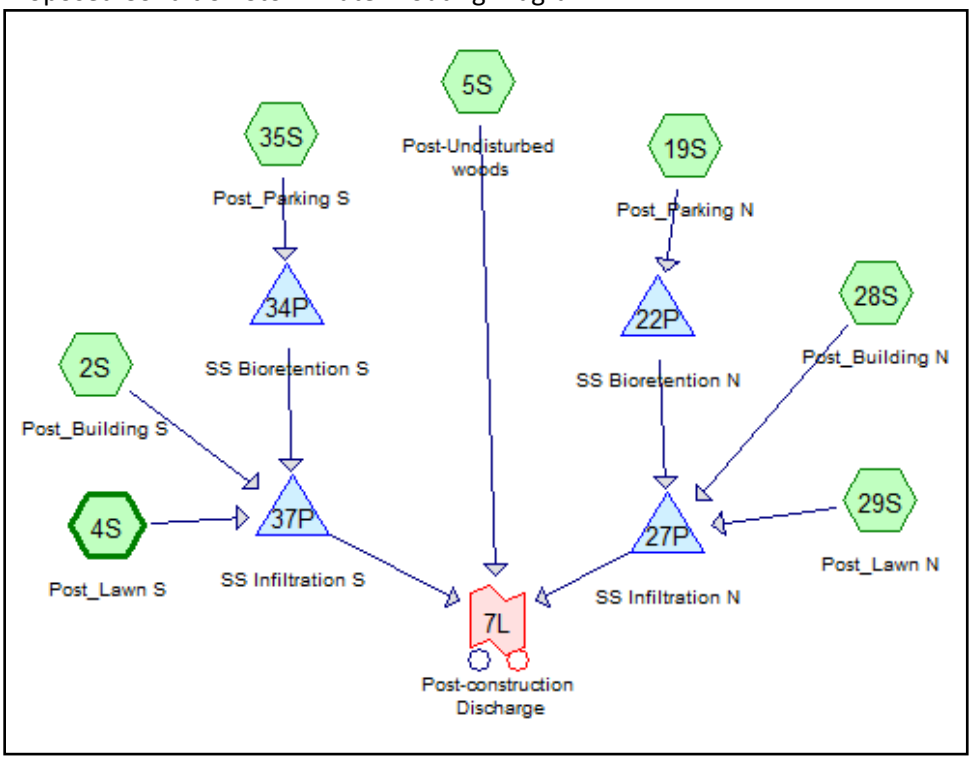
a. The soil present onsite is HSG 'B' with wooded land cover, and the Curve Number for this soil group, assuming the required good hydrologic condition, is 55.

b. Because the site is located within Somerset County, Region C of the NOAA rainfall distribution is used.

c. The SCS Standard DUH shall be used because the site lies outside of the coastal plain.

d. Exfiltration is not used in this design example.

Proposed Condition Stormwater Routing Diagram



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

Take note that the selections and configurations of the BMPs used to meet the green infrastructure, water quality, water quantity and groundwater recharge standards are for example purposes only and are not to be interpreted as the only way to design stormwater management measures. All designs and calculations of the selected BMPs will follow the design criteria and requirements in each of the corresponding BMP chapters.

Step 1: Obtain the Current Precipitation Depths

From the NOAA NWS PFDS, the site is located using the latitude and longitude provided above and the precipitation depths for the required design storms with a duration of 24 hours are obtained and provided in the table below.

Frequency of Storms	NOAA NWS PFDS Precipitation Depth (inches)
2-year	3.30
10-year	5.02
100-year	8.42

Step 2: Calculate the Current and Projected Precipitation Depths

To calculate the current and projected precipitation depths, the current precipitation adjustment factors and the future precipitation change factors must be obtained from Table 5-5 and Table 5-6, respectively, of the Stormwater Management rules. The site is in Somerset County. The current precipitation adjustment factors for the 2-, 10- and 100-year current storm events in Somerset County are 1.00, 1.03 and 1.09, respectively. The future precipitation change factors for the 2-, 10- and 100-year projected storm events in Somerset County are 1.19, 1.24 and 1.48, respectively.

Multiply the current precipitation adjustment factor with the precipitation depths obtained in *Step 1* to obtain the current precipitation depths. Multiply the future precipitation change factors with the precipitation depths obtained in *Step 1* to obtain the projected precipitation depths for the design storms. The current and projected precipitation depths for each storm frequency are listed in the table below.

Design Storms	NOAA NWS PFDS Precipitation Depth (inches)	Current Precipitation Depths (inches)	Future Projected Precipitation Depths (inches)
2-year	3.30	3.30	3.93
10-year	5.02	5.17	6.22
100-year	8.42	9.18	12.46

Step 3: Design the SSGI Bioretention Basins for the Water Quality Design Standard

The Water Quality Design Storm (WQDS) is 1.25 inches of precipitation in two hours. The runoff volume of the WQDS calculated by using the NRCS Methodology is 1,878 cf. Two SSGI bioretention BMPs are designed to treat the WQDS runoff from the proposed parking lot. Each proposed SSGI bioretention BMP will receive and treat 938.9 cf of total WQDS runoff. Each SSGI bioretention BMP is designed to have a footprint of 950 sf and a depth of 2 ft. Although the ground elevations may vary at different locations of a project site, for the sake of simplicity in illustrating this design example, the proposed ground elevations of the two proposed SSGI bioretention BMPs are both assumed to be at EL. 102.00 ft. The bottoms of the proposed SSGI bioretention BMPs are both assumed to be at EL. 100.00 ft.

The proposed outlet structure has a 24-inch by 12-inch opening as the orifice with a 12-inch pipe located down-gradient of the orifice to convey the runoff from storms larger than the WQDS to the down-gradient SSGI infiltration BMPs. The invert of the 24-inch by 12-inch orifice shall be set at 0.99 ft above the BMP bottom to provide a storage and infiltration volume of 938.9 cf.

The WQDS routing calculation for the proposed SSGI bioretention BMP is depicted below. Exfiltration is not used in the routing. Take note that the design of the outlet control structures and the configuration of the BMPs are just one example of the many possible variations that an engineer may incorporate into the design.

WQDS Routing Summary Report

Ex_5-8 NoExfiltration *NJ DEP 2-hr WQL Rainfall=1.25"*
Prepared by NJDEP Printed 7/12/2023
HydroCAD® 10.00-25 s/n 07311 © 2019 HydroCAD Software Solutions LLC Page 139

Summary for Pond 34P: SS Bioretention S

Inflow Area = 10,890 sf,100.00% Impervious, Inflow Depth = 1.03" for WQL event
Inflow = 0.77 cfs @ 1.08 hrs, Volume= 938.9 cf
Outflow = 0.00 cfs @ 0.00 hrs, Volume= 0.0 cf, Atten= 100%, Lag= 0.0 min
Primary = 0.00 cfs @ 0.00 hrs, Volume= 0.0 cf

Routing by Dyn-Stor-Ind method, Time Span= 0.00-36.00 hrs, dt= 0.01 hrs
Peak Elev= 100.988' @ 2.09 hrs Surf.Area= 950 sf Storage= 938.9 cf

Plug-Flow detention time= (not calculated: initial storage exceeds outflow)
Center-of-Mass det. time= (not calculated: no outflow)

Volume	Invert	Avail.Storage	Storage Description
#1	100.000'	1,900.0 cf	**Custom Stage Data (Conic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)	Wet.Area (sq-ft)
100.000	950	0.0	0.0	950
101.000	950	950.0	950.0	1,059
102.000	950	950.0	1,900.0	1,169

Device	Routing	Invert	Outlet Devices
#1	Device 2	100.990'	**24.0" W x 12.0" H Vert. Orifice/Grate** C= 0.600
#2	Primary	100.000'	**12.0" Round Culvert** L= 150.00' CPP, projecting, no headwall, Ke= 0.900 Inlet / Outlet Invert= 100.000' / 98.500' S= 0.0100 '/' Cc= 0.900 n= 0.011 Concrete pipe, straight & clean, Flow Area= 0.79 sf

Primary OutFlow Max=0.00 cfs @ 0.00 hrs HW=100.000' TW=95.500' (Dynamic Tailwater)
└─**2=Culvert** (Controls 0.00 cfs)
└─**1=Orifice/Grate** (Controls 0.00 cfs)

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

Routing for the projected 100-year storm is also an essential calculation which will ascertain whether the proposed BMP will not be overtopped during the projected 100-year storm. The result shows that the outlet structure of each of the proposed SSGI bioretention BMPs has sufficient capacity to pass runoff flow to its down-gradient SSGI infiltration BMP without being overtopped during the projected 100-year storm which addresses the impacts of climate change.

Summary Report for the Projected 100-year Storm

Ex_5-8 NoExfiltration *NOAA 24-hr C Proj_100yr Rainfall=12.46"*
Prepared by NJDEP Printed 7/12/2023
HydroCAD® 10.00-25 s/n 07311 © 2019 HydroCAD Software Solutions LLC Page 79

Summary for Pond 34P: SS Bioretention S

Inflow Area = 10,890 sf,100.00% Impervious, Inflow Depth = 12.22" for Proj_100yr event
Inflow = 3.66 cfs @ 12.09 hrs, Volume= 11,088.2 cf
Outflow = 3.25 cfs @ 12.11 hrs, Volume= 10,147.7 cf, Atten= 11%, Lag= 1.2 min
Primary = 3.25 cfs @ 12.11 hrs, Volume= 10,147.7 cf

Routing by Dyn-Stor-Ind method, Time Span= 0.00-36.00 hrs, dt= 0.01 hrs
Peak Elev= 101.687' @ 12.11 hrs Surf.Area= 950 sf Storage= 1,602.7 cf

Plug-Flow detention time= 97.2 min calculated for 10,144.8 cf (91% of inflow)
Center-of-Mass det. time= 50.3 min (783.0 - 732.6)

Volume	Invert	Avail.Storage	Storage Description
#1	100.000'	1,900.0 cf	**Custom Stage Data (Conic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)	Wet.Area (sq-ft)
100.000	950	0.0	0.0	950
101.000	950	950.0	950.0	1,059
102.000	950	950.0	1,900.0	1,169

Device	Routing	Invert	Outlet Devices
#1	Device 2	100.990'	**24.0" W x 12.0" H Vert. Orifice/Grate** C= 0.600
#2	Primary	100.000'	**12.0" Round Culvert** L= 150.00' CPP, projecting, no headwall, Ke= 0.900 Inlet / Outlet Invert= 100.000' / 98.500' S= 0.0100 '/' Cc= 0.900 n= 0.011 Concrete pipe, straight & clean, Flow Area= 0.79 sf

Primary OutFlow Max=3.25 cfs @ 12.11 hrs HW=101.687' TW=98.885' (Dynamic Tailwater)
└─**2=Culvert** (Inlet Controls 3.25 cfs @ 4.14 fps)
└─**1=Orifice/Grate** (Passes 3.25 cfs of 3.74 cfs potential flow)

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Step 4: Calculate the Peak Flow Rates Allowed under the Stormwater Runoff Quantity Control Standards

In accordance with the 3rd option under N.J.A.C. 7:8-5.6(b), the proposed design must demonstrate that the post-construction peak runoff rates for the current and projected 2-, 10- and 100-year storm events, respectively, are 50, 75, and 80 percent, respectively, of the pre-construction peak runoff rates. Therefore, we must calculate the runoff volume and peak flow rate for the pre-construction condition in accordance with the land cover, soil type, and precipitation depths identified in *Step 2*, if choosing this option.

Hydrologic modeling software is used to perform the runoff volume and flow rate calculations. The results of the peak flow rates for the current and projected 2-, 10- and 100-year design storms are shown in the following table.

Pre-construction Condition				
Design Storm	Current Precipitation Peak Flow Rate (cfs)		Projected Precipitation Peak Flow Rate (cfs)	
	To be disturbed	Undisturbed	To be disturbed	Undisturbed
2-year	0.19	0.05	0.49	0.12
10-year	1.40	0.35	2.35	0.59
100-year	5.61	1.40	9.75	2.44

The peak rate reductions required, as stated above, are applied to the peak flow rates calculated for the disturbed areas, with the results shown below in the column "Post-Construction Target." These values are summed with the peak flow rate for the undisturbed area to calculate the total allowable peak flow rate for each of the current and projected design storms.

Post-Construction Condition						
Design Storm	Current Precipitation Peak Flow Rate (cfs)			Projected Precipitation Peak Flow Rate (cfs)		
	Post-Construction Target	Undisturbed	Total Allowable	Post-Construction Target	Undisturbed	Total Allowable
2-year	0.10	0.05	0.15	0.24	0.12	0.36
10-year	1.05	0.35	1.40	1.76	0.59	2.35
100-year	4.49	1.40	5.89	7.80	2.44	10.24

Step 5: Design the SSGI Infiltration Basins to meet the Quantity Control Standard

To meet the allowable peak runoff rates calculated in *Step 4*, two identical SSGI infiltration basins are designed to collect the clean roof runoff from the proposed building, the parking lot runoff that has been treated by the proposed SSGI bioretention BMPs and the runoff from the lawn areas. Each of the proposed two SSGI infiltration BMPs is designed with a 5,020 sf footprint and depth of 4.5 ft, meaning the top of the basin bottom surface is set at El. 95.50. The outlet structure of each proposed BMP consists of a 6-inch orifice, a 12-inch by 6-inch orifice, a 5 ft by 5 ft top broad-crested weir and a 12-inch culvert discharge pipe. Take note that the 6-inch orifice is set at EL. 97.10, or 1.60 ft above the bottom of the BMP to accommodate storage volume for the runoff that will provide groundwater recharge; recharge is discussed in *Step 6*. Routing calculations for both of the current and projected 100-year storms follow below.

Summary Report for the Current 100-year Storm

Ex_5-8 NoExfiltration *NOAA 24-hr C Adj_100_yr Rainfall=9.18"*
Prepared by NJDEP Printed 7/12/2023
HydroCAD® 10.00-25 s/n 07311 © 2019 HydroCAD Software Solutions LLC Page 21

Summary for Pond 37P: SS Infiltration S

Inflow Area = 43,560 sf, 75.00% Impervious, Inflow Depth = 7.54" for Adj_100_yr event
Inflow = 8.93 cfs @ 12.10 hrs, Volume= 27,360.8 cf
Outflow = 2.21 cfs @ 12.35 hrs, Volume= 19,168.6 cf, Atten= 75%, Lag= 15.0 min
Primary = 2.21 cfs @ 12.35 hrs, Volume= 19,168.6 cf

Routing by Dyn-Stor-Ind method, Time Span= 0.00-36.00 hrs, dt= 0.01 hrs
Peak Elev= 98.430' @ 12.35 hrs Surf.Area= 5,020 sf Storage= 14,710.2 cf

Plug-Flow detention time= 260.7 min calculated for 19,163.3 cf (70% of inflow)
Center-of-Mass det. time= 162.6 min (930.1 - 767.5)

Volume	Invert	Avail.Storage	Storage Description
#1	95.500'	22,590.0 cf	**Custom Stage Data (Conic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)	Wet.Area (sq-ft)
95.500	5,020	0.0	0.0	5,020
96.500	5,020	5,020.0	5,020.0	5,271
97.100	5,020	3,012.0	8,032.0	5,422
97.500	5,020	2,008.0	10,040.0	5,522
98.500	5,020	5,020.0	15,060.0	5,773
99.500	5,020	5,020.0	20,080.0	6,025
100.000	5,020	2,510.0	22,590.0	6,150

Device	Routing	Invert	Outlet Devices
#1	Device 4	99.300'	**20.0' long x 0.5' breadth Broad-Crested Rectangular Weir** Head (feet) 0.200 0.400 0.600 0.800 1.000 Coef. (English) 2.80 2.92 3.08 3.30 3.32
#2	Device 4	97.900'	**12.0" W x 6.0" H Vert. Orifice/Grate** C= 0.600
#3	Device 4	97.100'	**6.0" Vert. Orifice/Grate** C= 0.600
#4	Primary	95.500'	**12.0" Round Culvert** L= 50.00' RCP, groove end projecting, Ke= 0.200 Inlet / Outlet Invert= 95.500' / 95.000' S= 0.0100 '/' Cc= 0.900 n= 0.011 Concrete pipe, straight & clean, Flow Area= 0.79 sf

Primary OutFlow Max=2.21 cfs @ 12.35 hrs HW=98.430' TW=94.000' (Dynamic Tailwater)
└─**4=Culvert** (Passes 2.21 cfs of 6.45 cfs potential flow)
  ├─**1=Broad-Crested Rectangular Weir** (Controls 0.00 cfs)
  ├─**2=Orifice/Grate** (Orifice Controls 1.22 cfs @ 2.45 fps)
  └─**3=Orifice/Grate** (Orifice Controls 0.98 cfs @ 5.00 fps)

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Summary Report for the Projected 100-year Storm

Ex_5-8 NoExfiltration *NOAA 24-hr C Proj_100yr Rainfall=12.46"*
Prepared by NJDEP Printed 7/12/2023
HydroCAD® 10.00-25 s/n 07311 © 2019 HydroCAD Software Solutions LLC Page 81

Summary for Pond 37P: SS Infiltration S

Inflow Area = 43,560 sf, 75.00% Impervious, Inflow Depth = 10.68" for Proj_100yr event
Inflow = 12.23 cfs @ 12.10 hrs, Volume= 38,779.8 cf
Outflow = 3.83 cfs @ 12.31 hrs, Volume= 30,582.7 cf, Atten= 69%, Lag= 12.4 min
Primary = 3.83 cfs @ 12.31 hrs, Volume= 30,582.7 cf

Routing by Dyn-Stor-Ind method, Time Span= 0.00-36.00 hrs, dt= 0.01 hrs
Peak Elev= 99.257' @ 12.31 hrs Surf.Area= 5,020 sf Storage= 18,860.1 cf

Plug-Flow detention time= 218.6 min calculated for 30,574.3 cf (79% of inflow)
Center-of-Mass det. time= 135.5 min (897.7 - 762.2)

Volume	Invert	Avail.Storage	Storage Description
#1	95.500'	22,590.0 cf	**Custom Stage Data (Conic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)	Wet.Area (sq-ft)
95.500	5,020	0.0	0.0	5,020
96.500	5,020	5,020.0	5,020.0	5,271
97.100	5,020	3,012.0	8,032.0	5,422
97.500	5,020	2,008.0	10,040.0	5,522
98.500	5,020	5,020.0	15,060.0	5,773
99.500	5,020	5,020.0	20,080.0	6,025
100.000	5,020	2,510.0	22,590.0	6,150

Device	Routing	Invert	Outlet Devices
#1	Device 4	99.300'	**20.0' long x 0.5' breadth Broad-Crested Rectangular Weir** Head (feet) 0.200 0.400 0.600 0.800 1.000 Coef. (English) 2.80 2.92 3.08 3.30 3.32
#2	Device 4	97.900'	**12.0" W x 6.0" H Vert. Orifice/Grate** C= 0.600
#3	Device 4	97.100'	**6.0" Vert. Orifice/Grate** C= 0.600
#4	Primary	95.500'	**12.0" Round Culvert** L= 50.00' RCP, groove end projecting, Ke= 0.200 Inlet / Outlet Invert= 95.500' / 95.000' S= 0.0100 '/' Cc= 0.900 n= 0.011 Concrete pipe, straight & clean, Flow Area= 0.79 sf

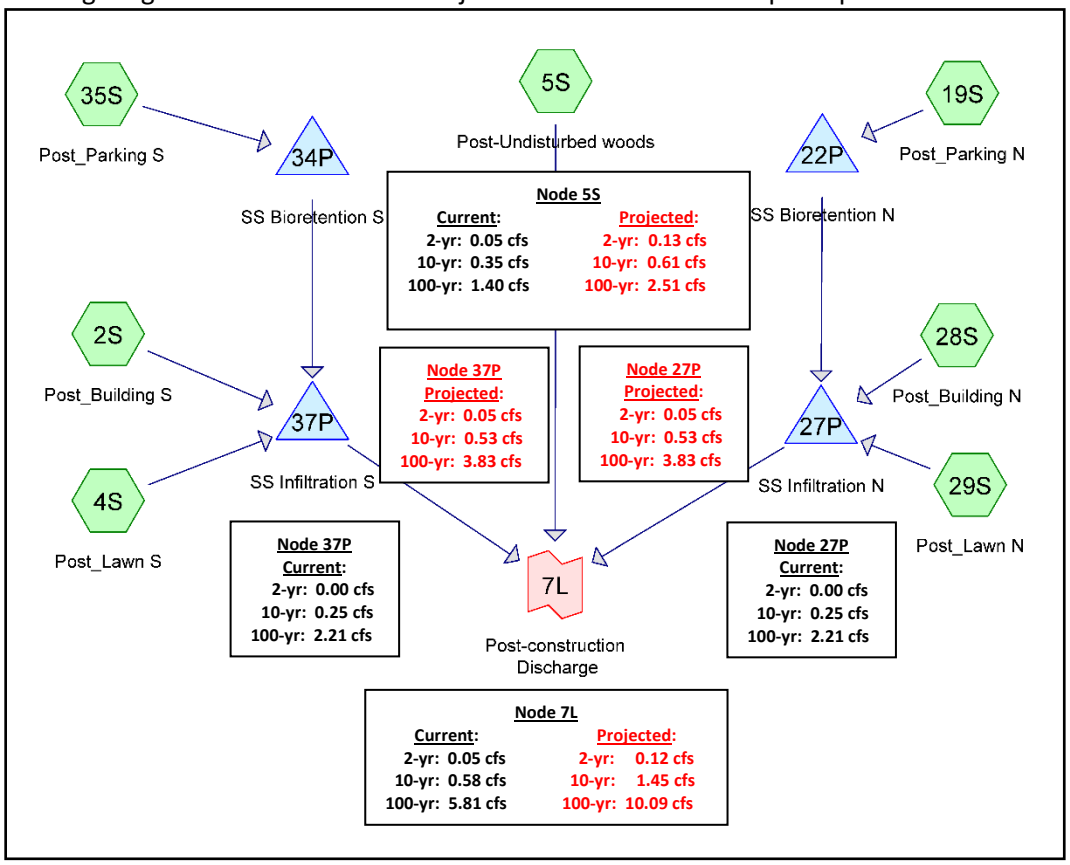
Primary OutFlow Max=3.83 cfs @ 12.31 hrs HW=99.257' TW=94.000' (Dynamic Tailwater)
└─**4=Culvert** (Passes 3.83 cfs of 7.46 cfs potential flow)
  ├─**1=Broad-Crested Rectangular Weir** (Controls 0.00 cfs)
  ├─**2=Orifice/Grate** (Orifice Controls 2.53 cfs @ 5.06 fps)
  └─**3=Orifice/Grate** (Orifice Controls 1.31 cfs @ 6.65 fps)

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

Take note that the configuration of outlets shown above is just one example of the many possible configurations and outlet designs. Designers and reviewers shall not interpret the design and arrangement shown in the routing calculations literally.

The post-construction runoff peak rates to be discharged offsite, including the discharges from the proposed SSGI infiltration BMP and the runoff from the undisturbed woods, have been added as text to the routing diagram provided below. Note that the peak flow rate from each element, or "node," cannot be arithmetically added to calculate the total discharge from the site, i.e., Node 7L, because the time to peak for each element is different. The post construction runoff peak flow rates for the off-site discharges are also provided in the next table.

Routing Diagram with Current and Projected Peak Flow Rates Superimposed



Source: HydroCAD® Diagram; HydroCAD is a registered trademark of HydroCAD Software Solutions LLC. Used with permission. Each numbered element is called a "node."

Post-Construction Condition Off-Site Discharges				
	Current Precipitation Peak Flow Rate (cfs)		Projected Precipitation Peak Flow Rate (cfs)	
Design Storm	Total Allowable	Discharge From the Site	Total Allowable	Discharge From the Site
2-year	0.15	0.05	0.36	0.12
10-year	1.40	0.58	2.35	1.45
100-year	5.89	5.81	10.24	10.09

Step 6: Demonstrate Compliance with the Groundwater Recharge Standards

The design demonstrates that the increase in stormwater runoff volume from the pre-construction to the post-construction condition for the projected 2-year storm is infiltrated through the proposed SSGI bioretention BMPs and SSGI infiltration BMPs. The projected 2-year design storm, with a precipitation depth of 3.93 inches, will produce the pre- and post-construction condition runoff volumes noted below:

Pre-construction Condition	Post-construction Condition	Increase
4,557 cf	22,443 cf	17,886 cf

Each of the SSGI bioretention BMPs provides an infiltration volume of 939 cf (rounding up from 938.9) below the first orifice. Therefore, the SSGI infiltration basins have to provide a total of 16,008 cf, which means 8,004 cf in each BMP. Each SSGI infiltration BMP is designed with a footprint of 5,020 sf. With the bottom of the SSGI infiltration basin set at EL. 95.50, the lowest orifice of the basin is set at 1.60 ft above the bottom of the BMP, which places the orifice at EL. 97.10 in order to provide sufficient storage volume for the runoff to be infiltrated. A summary report from the modeling software of the proposed SSGI infiltration basin follows.

Summary Report for the Projected 2-year Storm

Ex_5-8 NoExfiltration *NOAA 24-hr C Proj_2yr Rainfall=3.93"*
Prepared by NJDEP Printed 7/12/2023
HydroCAD® 10.00-25 s/n 07311 © 2019 HydroCAD Software Solutions LLC Page 121

Summary for Pond 37P: SS Infiltration S

Inflow Area = 43,560 sf, 75.00% Impervious, Inflow Depth = 2.71" for Proj_2yr event
Inflow = 3.44 cfs @ 12.10 hrs, Volume= 9,825.2 cf
Outflow = 0.05 cfs @ 20.38 hrs, Volume= 1,648.9 cf, Atten= 99%, Lag= 496.9 min
Primary = 0.05 cfs @ 20.38 hrs, Volume= 1,648.9 cf

Routing by Dyn-Stor-Ind method, Time Span= 0.00-36.00 hrs, dt= 0.01 hrs
Peak Elev= 97.227' @ 20.38 hrs Surf.Area= 5,020 sf Storage= 8,670.1 cf

Plug-Flow detention time= 815.0 min calculated for 1,648.9 cf (17% of inflow)
Center-of-Mass det. time= 557.5 min (1,339.8 - 782.2)

Volume	Invert	Avail.Storage	Storage Description
#1	95.500'	22,590.0 cf	**Custom Stage Data (Conic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)	Wet.Area (sq-ft)
95.500	5,020	0.0	0.0	5,020
96.500	5,020	5,020.0	5,020.0	5,271
97.100	5,020	3,012.0	8,032.0	5,422
97.500	5,020	2,008.0	10,040.0	5,522
98.500	5,020	5,020.0	15,060.0	5,773
99.500	5,020	5,020.0	20,080.0	6,025
100.000	5,020	2,510.0	22,590.0	6,150

Device	Routing	Invert	Outlet Devices
#1	Device 4	99.300'	**20.0' long x 0.5' breadth Broad-Crested Rectangular Weir** Head (feet) 0.200 0.400 0.600 0.800 1.000 Coef. (English) 2.80 2.92 3.08 3.30 3.32
#2	Device 4	97.900'	**12.0" W x 6.0" H Vert. Orifice/Grate** C= 0.600
#3	Device 4	97.100'	**6.0" Vert. Orifice/Grate** C= 0.600
#4	Primary	95.500'	**12.0" Round Culvert** L= 50.00' RCP, groove end projecting, Ke= 0.200 Inlet / Outlet Invert= 95.500' / 95.000' S= 0.0100 '/' Cc= 0.900 n= 0.011 Concrete pipe, straight & clean, Flow Area= 0.79 sf

Primary OutFlow Max=0.05 cfs @ 20.38 hrs HW=97.227' TW=94.000' (Dynamic Tailwater)
└─**4=Culvert** (Passes 0.05 cfs of 4.58 cfs potential flow)
  ├─**1=Broad-Crested Rectangular Weir** (Controls 0.00 cfs)
  ├─**2=Orifice/Grate** (Controls 0.00 cfs)
  └─**3=Orifice/Grate** (Orifice Controls 0.05 cfs @ 1.21 fps)

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

Step 8: Perform a Groundwater Mounding Analysis

In accordance with N.J.A.C. 7:8-5.2(h), whenever the stormwater management design includes one or more BMPs that will infiltrate stormwater into the subsoil, the design engineer shall assess the hydraulic impact on the groundwater table and design the site, to avoid adverse hydraulic impacts. The proposed SSGI bioretention BMPs and SSGI infiltration BMP are assessed below.

A. Proposed SSGI Bioretention BMPs

For this portion of the example, assume the saturated hydraulic conductivity is 2 inches/hour and the seasonal high water table (SHWT) is 13 ft below the proposed grade at EL. 103.00 at the location of the proposed SSGI bioretention BMPs. The hydraulic impact on the groundwater table is assessed by using *Hantush Spreadsheet*. Each proposed SSGI bioretention BMP has a footprint of 950 sf. The BMP is configured as 40 ft long and 23.75 ft wide. The BMP will infiltrate runoff from the WQDS, which generates a volume of 939 cf.

The inputs for the *Hantush Spreadsheet* are as follows:

Recharge rate = 1 inch/hour (half of the tested saturated hydraulic conductivity)
Specific yield = 0.15
Horizontal hydraulic conductivity = 1 inch/hour (outside the coastal plan)
½ of the BMP length = 20 ft (half of the length)
½ of the BMP width = 11.875 ft (half of the width)

The duration of infiltration period is equal to the volume of infiltrated runoff divided by the product of the BMP footprint and the recharge rate, and the calculation includes a conversion factor for converting inches to feet.

Duration of Infiltration Period = 11.86 hour = $\dfrac{939 \text{ cubic feet}}{\left(950 \text{ square feet} \times \left(1 \dfrac{\text{inch}}{\text{hour}} \times \dfrac{1}{12} \dfrac{\text{foot}}{\text{inch}}\right)\right)}$

Input Values		
1.00	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
1.00	Kh	Horizontal hydraulic conductivity (in/hr) Kh = 5xRecharge Rate (R) in the costal plan; Kh=R outside the coastal plan
20.000	x	1/2 length of basin (x direction, in feet)
11.875	y	1/2 width of basin (y direction, in feet)
11.86	t	Duration of infiltration period (hours)
10.00	hi(0)	Initial thickness of saturated zone (feet)
15.222	h(max)	Maximum thickness of saturated zone (beneath center of basin at end of infiltration period)
5.222	Δh(max)	Maximum groundwater mounding (beneath center of basin at end of infiltration period)

The results in the above graphic taken from the *Hantush Spreadsheet* show the groundwater mounding height is 5.222 ft above the seasonal high water table (SHWT). Given the condition that the existing grade at the location of the SSGI bioretention BMP is at EL. 103.00 and that the SHWT is located 13 ft below the basin bottom (basin invert), the top of the groundwater mounding is at EL. 95.222 (EL. 103.00 – 13 ft + 5.222 ft). Therefore, the mounding in the

groundwater table caused by the proposed infiltration of runoff will not affect the proposed SSGI bioretention BMP.

B. Proposed SSGI infiltration BMPs

It was stated earlier that the tested saturated hydraulic conductivity is 2 inches/hour, the proposed grade is at EL. 100, the top of the proposed SSGI infiltration basin is at EL. 95.50, and the seasonal high water table is 12 ft below the proposed grade, i.e. at EL. 88.00. The hydraulic impact on the groundwater table is to be assessed using *Hantush Spreadsheet*.

Each of the proposed SSGI infiltration BMPs has a footprint of 5,020 sf and a bottom at EL. 95.5. The BMP has a length of 125.50 ft and a width of 40 ft. The infiltration volume for each SSGI infiltration BMP is runoff volume 8,032 cf, which is the storage volume below the first orifice of the BMP calculated at *Step 7*.

The inputs for the *Hantush Spreadsheet* are as follows:

Recharge rate = 1 inch/hour (half of the tested saturated hydraulic conductivity)
Specific yield = 0.15
Horizontal hydraulic conductivity = 1 inch/hour (outside the coastal plan)
½ of the BMP length = 62.75 ft (half of the length)
½ of the BMP width = 20.00 ft (half of the length)
Duration of infiltration period = 19.2 hour = $\dfrac{8{,}032 \text{ cf}}{(5{,}020 \text{ sf} \times (1 \frac{\text{inch}}{\text{hour}} \times \frac{1}{12} \frac{\text{ft}}{\text{inch}}))}$

Input Values		
1.00	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
1.00	Kh	Horizontal hydraulic conductivity (in/hr) Kh = 5xRecharge Rate (R) in the costal plan; Kh=R outside the coastal plan
62.750	x	1/2 length of basin (x direction, in feet)
20.000	y	1/2 width of basin (y direction, in feet)
19.20	t	Duration of infiltration period (hours)
10.00	hi(0)	Initial thickness of saturated zone (feet)
19.463	h(max)	Maximum thickness of saturated zone (beneath center of basin at end of infiltration period)
9.463	Δh(max)	Maximum groundwater mounding (beneath center of basin at end of infiltration period)

The result shows that the mounding height is 9.46 ft above the SHWT, which places the mounding height at EL. 97.46. However, the BMP bottom is at EL. 95.50. The mounding of the groundwater table caused by the proposed BMP will therefore affect the infiltration of the runoff.

The next step is to adjust the duration of infiltration period and proportionally reduce the recharge rate. Increasing the duration of infiltration period to be 72 hours will require the recharge rate be proportionally reduced to 0.27 inches/hour. The mounding height becomes 6.97 ft, which is at EL. 94.97. The mounding analysis demonstrates that a mounding in the groundwater table caused by the proposed infiltration of runoff is below the BMP bottom at EL. 95.5 when the duration of infiltration is extended to 72 hours. Since the duration of infiltration is within the maximum drain downtime of 72 hours, the result of the mounding analysis is acceptable.

Input Values		
0.27	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
1.00	Kh	Horizontal hydraulic conductivity (in/hr) Kh = 5xRecharge Rate (R) in the costal plan; Kh=R outside the coastal plan
62.750	x	1/2 length of basin (x direction, in feet)
20.000	y	1/2 width of basin (y direction, in feet)
72.00	t	Duration of infiltration period (hours)
10.00	hi(0)	Initial thickness of saturated zone (feet)
16.967	h(max)	Maximum thickness of saturated zone (beneath center of basin at end of infiltration period)
6.967	Δh(max)	Maximum groundwater mounding (beneath center of basin at end of infiltration period)

The above calculations and designs show how to use the current precipitation depth and the projected precipitation depth to demonstrate compliance with the Stormwater Management rules.

Example 5-9: Calculate the Peak Flow Rate for Sizing a Manufactured Treatment Device Treating the WQDS from a 0.25-acre Impervious Motor Vehicle Surface

The following parameters apply for a site located in Mercer County:

Area =	0.25 ac
Slope =	5%
CN Value =	98 (100% Impervious)
Projected 2-year storm rainfall depth	3.84 inches
Hydraulically most distant point to the inlet of MTD	110 ft

Step 1: Calculate Time of Concentration

The calculation of the time of concentration follows the steps in Example 5-1. As stated previously, the rainfall depth for the 2-year storm, P_2, used in the sheet flow calculation shall be that of the projected 2-year storm. The maximum sheet flow length calculated by McCuen-Speiss limitation is 909 ft, which is greater than 100 ft. Therefore, 100 ft is used for the sheet flow length. The shallow concentrated flow length is 10 ft. The total time of concentration is 0.8 minutes.

Step 2: Calculate the WQDS Peak Flow Rate

When calculating the peak flow rate using NRCS methodology for an MTD, the SCS Standard DUH, which is with a peak rate factor of 484, must be used. Using the NRCS methodology with the time of concentration calculated in *Step 1*, the WQDS peak flow rate is 0.77 cfs.

The next step would be to determine the size of the MTD. Refer to *Chapter 9.5* for Green Infrastructure MTDs and *Chapter 11.3* for MTDs that are not certified as green infrastructure for this calculation.

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

Site Condition or Parameter	NRCS Methodology
Applicability	Peak flow rate, runoff volume, hydrograph comparison, sizing inflow rate and volume of BMPs
Groundwater Recharge	Difference of Runoff volumes of pre- and post-construction projected 2-year storms
Mixture of pervious and directly connected impervious surfaces	Calculate the runoff from impervious surface and pervious surface separately
Unconnected impervious surface	*NEH Part 630, Chapter 9* unconnected impervious surface (less than 30%) or NJ DEP Two-Step Technique
Runoff Parameters	Curve Numbers from *NEH, Part 630, Chapter 9*
Rainfall Data	Use either NOAA precipitation data or NRCS County average rainfall data, as follows: ▪ Apply current precipitation adjustment factors to the rainfall data for the current design storms for both the pre- and post-construction conditions ▪ Apply future precipitation change factors to the rainfall data and calculate the projected precipitation depths for the projected design storms for both the pre- and post-construction conditions NJDEP Water Quality Design Storm rainfall is 1.25 inches in 2 hours
Pre-construction Time of Concentration	Assume sheet flow length = 100 ft, unless a physical condition exists that shortens the sheet flow length to less than 100 ft Maximum sheet flow roughness coefficient n = 0.40
Post-construction Time of Concentration	There is no minimum value for T_c An assumed T_c value may not be used Maximum Sheet flow length is 100 feet or the length calculated by McCuen-Spiess limitation, whichever is smaller Maximum sheet flow roughness coefficient n = 0.40

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