# South Carolina Unit Hydrograph Method Applications Manual

## Final Report

*Prepared by:*

Michael E. Meadows

Department of Civil and Environmental Engineering
University of South Carolina

**FHWA-SC-20-02**

**May 2020**

*Sponsoring Agencies:*

**South Carolina Department of Transportation**
*Office of Materials and Research*
1406 Shop Road
Columbia, SC 29201

**Federal Highway Administration**
*South Carolina Division*
Strom Thurmond Federal Building
1835 Assembly Street, Suite 1270
Columbia, SC 29201

**Technical Report Documentation Page**

| 1. Report No<br>FHWA-SC-20-02 | 2. Government Accession No. | 3. Recipient's Catalog No. |
|---|---|---|
| 4. Title and Subtitle<br>South Carolina Unit Hydrograph Method Applications Manual | | 5. Report Date<br>May 2020<br>6. Performing Organization Code |
| 7. Author/s<br>Michael E. Meadows | | 8. Performing Organization Report No. |
| 9. Performing Organization Name and Address<br>Department of Civil and Environmental Engineering<br>University of South Carolina<br>300 Main Street<br>Columbia, SC 29208 | | 10. Work Unit No. (TRAIS)<br>11. Contract or Grant No.<br>SPR No. 738 |
| 12. Sponsoring Organization Name and Address<br>South Carolina Department of Transportation<br>Office of Materials and Research<br>1406 Shop Road<br>Columbia, SC 29201 | | 13. Type of Report and Period Covered<br>User's Manual<br>14. Sponsoring Agency Code |
| 15. Supplementary Notes | | |

16. Abstract

This manual presents and illustrates the South Carolina Synthetic Unit Hydrograph Method (identified as the "SC UH Method"). A key parameter is peak rate factor (PRF) that relates to UH shape and proportion of runoff volume under the rising limb. PRF varies among watersheds and land use. A table with land use specific PRF values recommended for use in South Carolina is included. This table resulted from multiple stormwater management studies that verified the SC UH Method and PRF values. At each study watershed, rainfall and streamflow data were collected and used to calibrate model parameters by adjusting UH PRF and time to peak parameters to obtain the closest match between simulated and measured hydrographs. A significant outcome of those studies, and one that upholds the purpose and intent of the SC UH Method is each watershed has its own unique PRF and thereby, its own unique UH.

| 17. Key Words<br>SC Synthetic Unit Hydrograph, Peak Rate Factor, User's Manual | 18. Distribution Statement<br>No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161. | | |
|---|---|---|---|
| 19. Security Classification (of this report)<br>Unclassified | 20. Security Classification (of this page)<br>Unclassified | 21. No. Of Pages | 22. Price |

**Form DOT F 1700.7** (8-72) Reproduction of form and completed page is authorized

# DISCLAIMER

The contents of this report reflect the views of the author who is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the South Carolina Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

The State of South Carolina and the United States Government do not endorse products or manufacturers. Trade or manufacturer's names appear herein solely because they are considered essential to the object of this report.

# ACKNOWLEDGEMENTS

The author acknowledges the South Carolina Department of Transportation and the Federal Highway Administration for encouraging, supporting, and funding the development of this Manual. Special thanks to the project Steering Committee Members:

Tom Knight – Chairman, Technical Adviser, and Editor
Rachel Schilling
Bobby Holtzclaw
Elizabeth Mesa
Anthony Lusk
Blake Gerken – FHWA

A special thank you is extended to Stephen Benedict, past chairman, and Randall Mungo, ad hoc committee member, for their work with the committee before their retirements.

# EXECUTIVE SUMMARY

This manual was written to present and illustrate the South Carolina Synthetic Unit Hydrograph Method (identified as the “SC UH Method”). The SC UH Method uses the two-parameter gamma distribution to describe and enumerate the unit hydrograph. A key parameter is peak rate factor (PRF) that relates to UH shape and proportion of runoff volume under the rising limb. A table with land use specific PRF values is included in this manual. This table resulted from multiple stormwater management studies that verified the SC UH Method and PRF values. At each study watershed, rainfall and streamflow data were collected and used to calibrate model parameters by adjusting UH PRF and time to peak parameters to obtain the closest match between simulated and measured hydrographs. The optimal UH PRF parameter values then were validated during model verification studies in which the parameter values were not adjusted. A significant outcome of those studies, and one that upholds the purpose and intent of the SC UH Method is each watershed has its own unique PRF and thereby, its own unique UH.

Manual chapters 1 through 5 provide discussions of the origin and evolution of the SC UH Method, an explanation of the components, notably the unit hydrograph Peak Rate Factor, discussion of stormwater management studies that verified the method, and example applications. Chapter 6 discusses the spreadsheet developed to facilitate user application of the SC UH Method.

Chapter 3 includes discussion of two new concepts and procedures that are part of the SC UH Method and are incorporated into the Spreadsheet. One is modifying the NRCS Curve Number for rainfall durations less than 24-hours and the other is identifying critical storm durations that produce the maximum peak flow and/or maximum runoff volume. A significant outcome of CN modification is most critical durations are not 24-hours, particularly for peak flow prediction, which gives reason and justification to challenge regulations that prescribe a single design storm duration that is not a critical duration and could lead to an unsafe design.

An important fact learned during the development of the SC UH Method is that watersheds are like people. As the author tells students in his Engineering Hydrology class: watersheds are just like each of us. Every person has a different height, weight, complexion, etc. Every watershed has a different area, hydraulic length, slope, land use, soils, etc. People have different personalities and so do watersheds (i.e., different watersheds have different CNs, PRFs, and Tc) which are watershed personality parameters. The SC UH method has been tested and proven at multiple watersheds.

# TABLE OF CONTENTS

# LIST OF FIGURES

# LIST OF TABLES

# CHAPTER 1: INTRODUCTION

The purpose of this manual is to present and illustrate the South Carolina Synthetic Unit Hydrograph Method (hereinafter identified as the "SC UH Method"). This includes a discussion of the origin and evolution of the method, an explanation of the components, notably the unit hydrograph Peak Rate Factor, discussion of stormwater management studies that verified the method, example applications, and an overview of the spreadsheet developed for users to implement the SC UH Method.

The South Carolina UH Method uses the two-parameter gamma distribution to describe the unit hydrograph. A key parameter in the UH peak flow equation is peak rate factor (PRF) that relates to the UH shape and proportion of runoff volume under the rising limb. A table with land use specific PRF values recommended for use in South Carolina is included in this Manual. Unit hydrographs with high PRF values have steeper rising limbs and greater volume under the rising limb than unit hydrographs with low PRF values. PRF values are higher at watersheds that are more efficiently and hydraulically better drained than watersheds with low PRF values. PRF is defined as an index of watershed hydraulic efficiency.

Historically, most unit hydrograph applications used the standard Natural Resource Conservation Service (NRCS) unit hydrograph with PRF equal to 484 (known as the 484 UH). The same UH PRF was/is used without modification for different land use conditions and this has/does result in significant design mistakes. The NRCS acknowledges unit hydrographs with PRF values other than 484. In 2007 the NRCS included dimensionless unit hydrographs with PRF values other than 484 in Chapter 16 of the National Engineering Handbook, Part 630 Hydrology. The NRCS document suggests different watersheds have different PRF values but does not provide guidance to assign land use specific PRF values. Without proper guidance how can designers decide what PRF value to use?

A table with land use specific PRF values is included in this manual. PRF is a unique and fundamental component of the SC UH Method. This table resulted from multiple stormwater management studies that verified the SC UH Method and PRF values. At each study watershed, rainfall and streamflow data were collected and used to calibrate model parameters. This involved adjusting UH PRF and time to peak parameters to obtain the closest match between simulated and measured hydrographs. The optimal PRF values for different land uses were evaluated statistically and, per recommendations by the NRCS, summarized as single values for unique land uses.

This manual includes the discussion of a spreadsheet developed to facilitate user application of the SC UH Method. Primary input includes rainfall and watershed data used to simulate runoff volumes and hydrographs for 1, 2, 3, 6, 12, and 24-hour storms with annual exceedance probabilities (AEP) of 1, 2, 4 and 10%. Runoff volumes and hydrographs also are computed for 24-hour storms with 1, 2, 4, 10, 20, 50, and 100% AEP.

The spreadsheet also calculates stormwater and sediment pond performances for 1, 2, 3, 6, 12 and 24-hour storms of user specified AEP, annual erosion with the Universal Soil Loss Equation (USLE), and single event erosion for 1, 2, 3, 6, 12, and 24-hour storms of user specified AEP with the Modified Universal Soil Loss Equation (MUSLE).

A unique feature is this spreadsheet modifies the NRCS Curve Number (CN) for rainfall durations less than 24 hours. A significant outcome of CN modification is most critical durations are not 24-hours, particularly for peak flow prediction, which gives reason and justification to challenge and change regulations that prescribe a single design storm duration that is not a critical duration and could lead to a wrong and unsafe design.

## 1.1 Conventions and Assumptions

This manual assumes readers have basic understanding and experience with runoff hydrograph simulation using the NRCS methods for temporal rainfall distribution, watershed lag time, time of concentration, curve number, and the standard 484 UH. These methods are reviewed in addition to the presentation of the SC UH Method with example applications.

# CHAPTER 2: BACKGROUND

Research related to the SC UH Method began in the 1970s at the University of Tennessee during stormwater research using the Tennessee Valley Authority (TVA) Double Triangle Model (DTM). TVA developed the DTM to simulate stormwater hydrographs at gaged and un-gaged basins in the Tennessee River Valley using only rainfall and watershed data. The DTM used a unit response function (URF) (i.e., UH) to represent watershed response to a given storm. The URF structure was a quadrilateral formed by adding two triangles, hence the name Double Triangle Model. The two triangles were interpreted as initial and delayed responses, respectively. The URF for highly impervious watersheds and/or watersheds well drained by sewers was dominated by the first triangle; whereas, for forested watersheds and watersheds where shallow groundwater response was important, either the second triangle was dominant or both triangles were nearly equal.

The DTM was shown to be a good tool to analyze watershed runoff data. It could be used to investigate the effects of different land uses and examine initial and delayed responses. During studies conducted by the TVA and the University of Tennessee, optimal URF parameters were determined for nearly 500 events at over 30 watersheds in urban, agricultural, and forested land use conditions. Normalized results were examined to determine if a means could be found to explain variations among watersheds. The best relationships were found by averaging the URFs according to the major land use category. The categories chosen were urban with extensive storm sewer systems, urban with minor storm sewer systems, row crop agriculture, and forest. PRF values for the DTM were determined for the different land uses as follows: urban with extensive storm sewer systems--555, urban with minor storm sewer systems--270, row crop agriculture--316, and forest--182.

The DTM was chosen as a diagnostic tool for the Senn Branch[1] study (Meadows and Nevils, 1987), the first stormwater management study to apply the SC UH Method. Plans were underway to convert portions of Senn Branch watershed from mostly forest and agriculture to suburban and commercial land use. A stormwater study was performed wherein rainfall and streamflow gages were installed at multiple locations. The Double Triangle Model was used to analyze the monitoring data at this watershed and at Rocky Branch[2] and Smith Branch[3] watersheds. The reason three watersheds were studied was because Rocky Branch and Smith Branch had land use patterns like the patterns planned for Senn Branch which could provide insight about stormwater runoff changes to expect after development.

---

[1] Senn Branch watershed is in Lexington County west of I-26 between Leaphart Road and Ephrata Drive. The creek crosses Hwy 378 downhill from Lexington Medical Center at 2720 Sunset Blvd, West Columbia, SC.

[2] Rocky Branch drains portions of downtown Columbia, the University of South Carolina (UofSC) campus, Five Points, Shandon, and other residential areas.

[3] Smith Branch drains portions of downtown Columbia, the residential and commercial districts between North Main Street and Highway 277 (Bull Street), and the area around Prisma Health, formerly Palmetto Richland Hospital.

Stormwater management alternatives were evaluated for Senn Branch watershed with the Drain:Edge computer program (Meadows, 1986), which was the first program to use the SC UH Method. Drain:Edge was a stormwater simulation program developed at the University of South Carolina (UofSC) Department of Civil and Environmental Engineering. Drain:Edge was based on an extended version of HYMO (Williams and Hann, 1972), a problem-oriented computer language for modeling stormwater runoff from watersheds. HYMO was developed by the United States Department of Agriculture, Agricultural Research Service (USDA-ARS) for planning flood control projects, forecasting, and watershed research. Drain:Edge was created by replacing the hydrograph generation algorithm with an early version of the SC UH Method. Drain:Edge also included algorithms for routing hydrographs through storm sewers, drainage ditches, culverts, and stormwater ponds.

Runoff hydrographs from individual subwatersheds were simulated, routed through any ponds and channels, and added in real time at channel confluences. Initial tests were conducted using a common PRF for all subwatersheds. Several PRFs were tried, and in each case, the results were unacceptable because of the poor reproduction of measured hydrographs. The subwatersheds then were grouped into three categories and assigned PRFs based on land use and basin geomorphology. After several trials, it was determined the best value of subwatershed PRFs was: 555 for the developed and paved areas with sewers; 341 for areas with some development but with little drainage systems improvements; and 182 for undeveloped and unimproved areas.

A milestone study related to the SC UH Method was a study at UofSC jointly funded by the U.S. Geological Survey (USGS) and the South Carolina Department of Highways and Public Transportation, now the South Carolina Department of Transportation (SCDOT) (Meadows and Ramsey, 1991). The goal for that study was to develop a regionalized synthetic unit hydrograph method for South Carolina. The unit hydrograph was described with the two-parameter gamma distribution. Regional equations for PRF and time to peak were developed based on an analysis of nearly 400 events at 24 watersheds monitored by the USGS. The regional equations involved various combinations of watershed area, percentage imperviousness, hydraulic length, main channel slope and basin development factor (BDF) that is based on (1) channel improvements, (2) channel linings, (3) storm sewers, and (4) curb and gutter streets (Sauer, et al. (1981)).

During Phase I of the Rawls Creek study (Meadows and Eliatamby, 1990), the PRF values were estimated by assigning a unique PRF for each major land use and determining an Area Weighted PRF for each subwatershed. These values were updated during model calibration. In the period between Phases I and II, the study of USGS gaged watersheds was completed. During Phase II, the PRF values were estimated with the regional PRF equations (Meadows, Morris, and Spearman, 1992). Two verification tests were performed. The first focused on Koon Branch, a tributary to Rawls Creek. Frequency flood peaks calculated using the equation predicted PRF values were compared to values obtained with the calibrated model. Results for the 2, 5, 10, 25, 50, and 100-year events at different road crossings showed good agreement, confirming confidence in the revised data, and reinforcing the recommendation to use the equation predicted PRF values. The second verification test compared flood peaks at several crossings along the main channel of Rawls Creek. The results also showed good agreement.

A significant basis for the current PRF and Land Use Table was the Wise Hollow study in Aiken County, SC (Meadows, Morris, and Spearman, 1991). During stormwater management studies prior to the Wise Hollow Study, researchers determined ranges of PRF values for four general land uses: (1) commercial; (2) single family residential; (3) row crop; and (4) forest. The four groups were: (1) areas that are highly developed and paved with sewers; (2) areas with some development but with minor drainage system improvement; (3) areas with a high concentration of agricultural land; and (4) undeveloped and unimproved areas. PRF values (i.e., ranges of values) for these land uses are shown in Table 2.1.

**Table 2.1**
**Results from Stormwater Management Studies**
**Prior to Wise Hollow Study**

| Land Use | PRF |
|---|---|
| Commercial | 550 |
| Single Family Residential | 320-345 |
| Row Crop | 300-320 |
| Forest | 180 |

The PRF values selected for the Wise Hollow Creek watershed are shown in Table 2.2. For commercial and industrial areas with a high percent imperviousness and are well drained, PRF=550 was chosen, based on the results in Table 2.1. The value for forests in Table 2.1 also was chosen (i.e., PRF=180. The values for pasture and open spaces were determined using the equations developed during the study funded by the USGS and SCDOT. The value for row crop was selected as the lower end of the range given in Table 2.1. The values for single and multi-family residential were determined as weighted averages of open space and commercial land use based on estimates of average values for percent imperviousness.

**Table 2.2**
**PRF Values for Specific Land Uses**

| Land Use | PRF |
|---|---|
| Urban | |
| Single Family Residential | 325 |
| Multi-family Residential | 375 |
| Commercial | 550 |
| Industrial | 550 |
| Open Spaces | 250 |
| Agricultural | |
| Forest | 180 |
| Pasture | 200 |
| Row Crop | 300 |

Implementation of single PRF values for each land use followed recommendations by the NRCS to have a method that provided a single PRF value for each unique land use, analogous to the

NRCS curve number method. The values in Table 2.2 were tested during stormwater management studies wherein simulated hydrographs were statistically compared to monitored hydrographs. At each study watershed, rainfall and streamflow data were collected and used to calibrate model parameters by adjusting UH PRF and time to peak parameters to obtain the closest match between simulated and measured hydrographs. The optimal UH PRF parameter values then were validated during model verification studies in which the parameter values were not adjusted. The validation results affirmed the PRF values in Table 2.1. The ultimate outcome from the results of the stormwater management studies is the table of Recommended PRF Values included in Chapter 3.

# CHAPTER 3: OVERVIEW OF THE SC UH METHOD

The following sections outline different elements of the SC UH Method. The first two sections discuss a recommended source of rainfall data and updated distribution curves. The third section overviews the NRCS Curvilinear UH (i.e., the 484 UH) that was a primary reason for the development of the SC UH Method. The next two sections discuss the NRCS CN Model, CN values, adjustments for rainfall durations less than 24 hours, and pending modifications to the CN model created by changing the initial abstractions from 0.2S to 0.05S where S is the watershed retention factor that is a function of CN. The next section discusses the watershed lag time and time of concentration. The last two sections discuss the SC UH and PRF values for South Carolina.

## 3.1 Rainfall Data

When using the SC UH Method to simulate stormwater hydrographs, the recommended source for rainfall data is the Precipitation-Frequency Atlas of the United States, National Oceanic and Atmospheric Administration (NOAA) Atlas 14, Vol 2, Version 3, NWS-NOAA, Silver Spring, MD, 2004. Recently updated data can be accessed with the internet PFDS at http://hdsc.nws.noaa.gov/hdsc/pfds/.

The PFDS is a point-and-click interface developed to deliver NOAA Atlas 14 precipitation frequency estimates and associated information. By clicking on a state in the online map or selecting a state name from the drop-down menu (Figure 3.1), an interactive map of the state will be displayed (Figure 3.2). A user can identify a specific location where precipitation frequency estimates are needed by moving the crosshair to the location or double clicking on the location. Figure 3.2 shows the selection of Blythewood, SC for Point Precipitation Frequency Estimates.

Precipitation depths and intensities for Storm Durations Ranging from 5-Minutes to 60-Days with Average Recurrence Intervals (Return Periods) from 1-Year to 1000-Years and their confidence intervals can be displayed directly as tables or graphs via separate tabs. Links to supplementary information (e.g., ASCII grids of estimates, associated temporal distributions of heavy rainfall, time series data at observation sites, and cartographic maps) can also be found.

Note the location information in the right-hand portion of Figure 3.2. Under the heading Data Description at the top of the figure are drop down boxes to select Data type (**Precipitation Depth**[4] or Precipitation Intensity), precipitation Units (**English** or Metric), and Time series type (Partial duration or **Annual maximum**). For Blythewood, SC selected data are Precipitation Depth, English units, and Partial duration as shown in Figure 3.3.

[4] Bolded options indicate SCDOT preferences.

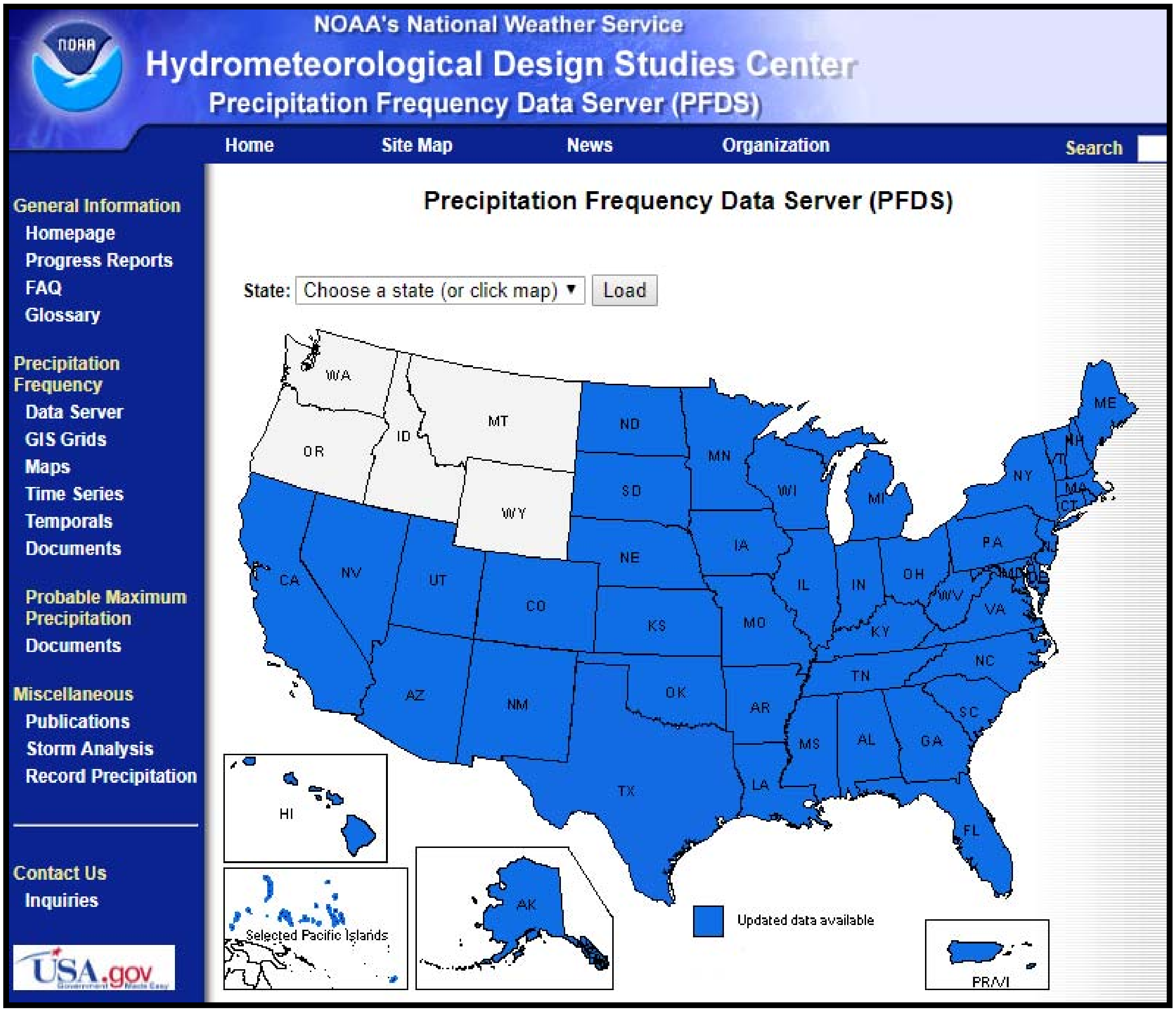



**Figure 3.1. Screen Shot Showing State Selection Map**

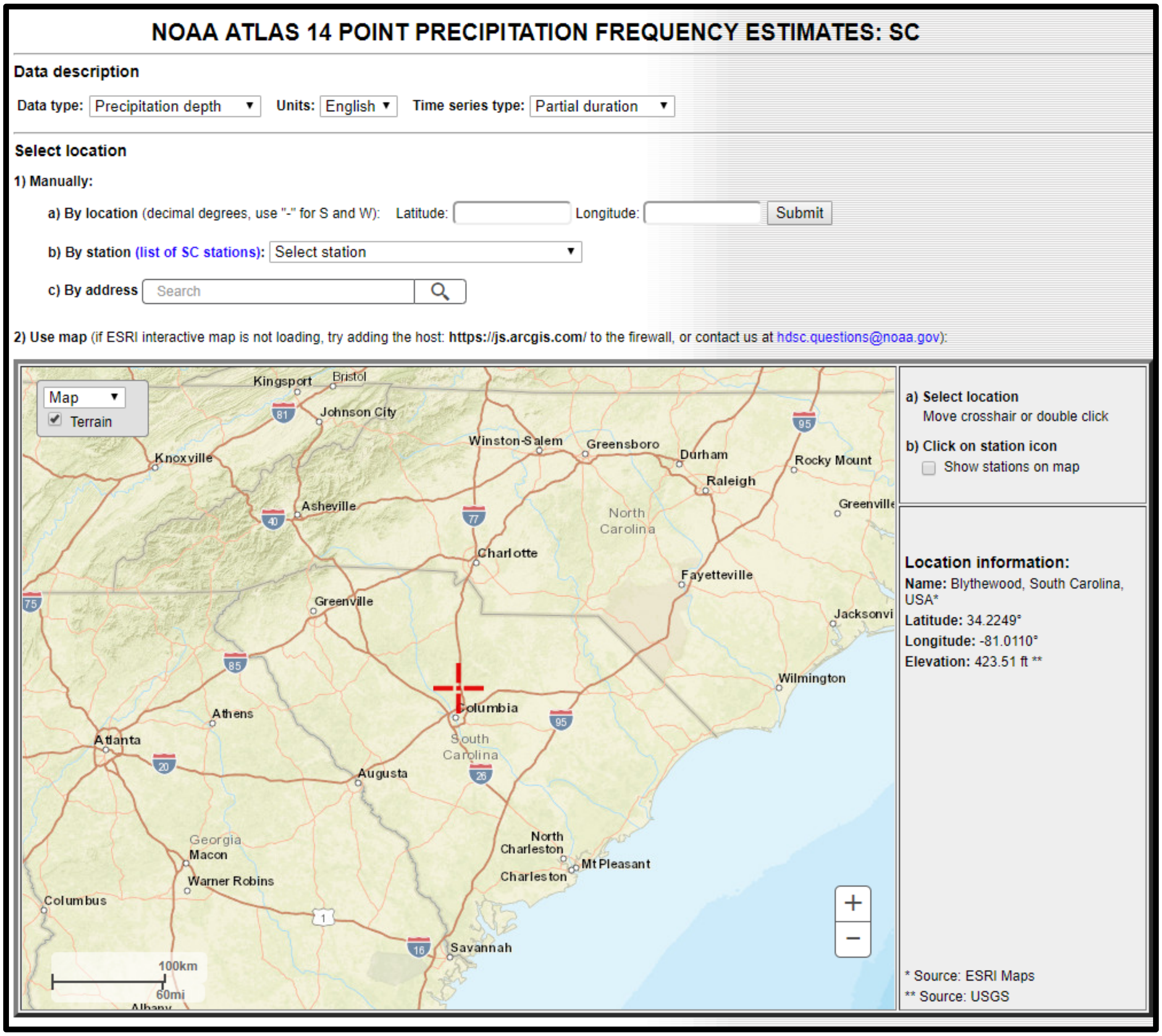



**Figure 3.2. Site Selection Screen Showing Crosshair and Location Information for Blythewood, SC**

| 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 | **0.435** (0.401-0.473) | **0.506** (0.466-0.551) | **0.581** (0.534-0.633) | **0.650** (0.596-0.706) | **0.728** (0.665-0.790) | **0.790** (0.719-0.858) | **0.849** (0.768-0.920) | **0.906** (0.815-0.983) | **0.974** (0.869-1.06) | **1.03** (0.914-1.13) |
| 10-min | **0.695** (0.641-0.756) | **0.810** (0.745-0.882) | **0.931** (0.856-1.01) | **1.04** (0.954-1.13) | **1.16** (1.06-1.26) | **1.26** (1.15-1.37) | **1.35** (1.22-1.46) | **1.44** (1.29-1.56) | **1.54** (1.38-1.68) | **1.63** (1.44-1.77) |
| 15-min | **0.869** (0.801-0.945) | **1.02** (0.936-1.11) | **1.18** (1.08-1.28) | **1.32** (1.21-1.43) | **1.47** (1.34-1.60) | **1.59** (1.45-1.73) | **1.71** (1.54-1.85) | **1.81** (1.63-1.97) | **1.94** (1.73-2.11) | **2.04** (1.81-2.22) |
| 30-min | **1.19** (1.10-1.30) | **1.41** (1.29-1.53) | **1.67** (1.54-1.82) | **1.91** (1.75-2.07) | **2.18** (1.99-2.36) | **2.40** (2.18-2.60) | **2.61** (2.36-2.83) | **2.82** (2.54-3.06) | **3.09** (2.75-3.36) | **3.31** (2.93-3.60) |
| 60-min | **1.49** (1.37-1.62) | **1.76** (1.62-1.92) | **2.15** (1.97-2.33) | **2.48** (2.28-2.70) | **2.90** (2.65-3.15) | **3.25** (2.96-3.53) | **3.60** (3.26-3.90) | **3.96** (3.56-4.29) | **4.43** (3.95-4.81) | **4.83** (4.27-5.26) |
| 2-hr | **1.69** (1.55-1.84) | **2.01** (1.85-2.20) | **2.46** (2.26-2.69) | **2.88** (2.63-3.14) | **3.42** (3.11-3.72) | **3.89** (3.52-4.22) | **4.36** (3.93-4.73) | **4.87** (4.36-5.28) | **5.56** (4.93-6.04) | **6.16** (5.41-6.71) |
| 3-hr | **1.78** (1.63-1.96) | **2.12** (1.94-2.33) | **2.61** (2.38-2.86) | **3.07** (2.80-3.36) | **3.68** (3.34-4.02) | **4.22** (3.81-4.60) | **4.79** (4.29-5.22) | **5.41** (4.81-5.89) | **6.28** (5.51-6.85) | **7.06** (6.13-7.70) |
| 6-hr | **2.12** (1.94-2.34) | **2.52** (2.31-2.78) | **3.10** (2.83-3.42) | **3.65** (3.33-4.02) | **4.40** (3.99-4.83) | **5.07** (4.56-5.54) | **5.77** (5.15-6.30) | **6.53** (5.78-7.12) | **7.62** (6.66-8.32) | **8.61** (7.42-9.40) |
| 12-hr | **2.49** (2.27-2.76) | **2.97** (2.71-3.29) | **3.67** (3.34-4.06) | **4.34** (3.94-4.79) | **5.27** (4.75-5.79) | **6.10** (5.46-6.70) | **6.99** (6.20-7.66) | **7.96** (6.99-8.73) | **9.38** (8.10-10.3) | **10.7** (9.07-11.7) |
| 24-hr | **2.98** (2.77-3.21) | **3.58** (3.33-3.85) | **4.47** (4.16-4.81) | **5.22** (4.85-5.61) | **6.33** (5.84-6.80) | **7.27** (6.67-7.81) | **8.30** (7.54-8.93) | **9.42** (8.48-10.2) | **11.1** (9.82-12.0) | **12.5** (10.9-13.5) |
| 2-day | **3.51** (3.27-3.77) | **4.21** (3.92-4.52) | **5.22** (4.86-5.62) | **6.07** (5.64-6.53) | **7.29** (6.73-7.84) | **8.32** (7.64-8.96) | **9.42** (8.59-10.2) | **10.6** (9.60-11.5) | **12.3** (11.0-13.4) | **13.8** (12.2-15.1) |
| 3-day | **3.73** (3.48-4.00) | **4.47** (4.17-4.80) | **5.53** (5.15-5.93) | **6.40** (5.95-6.87) | **7.65** (7.08-8.21) | **8.70** (8.00-9.34) | **9.81** (8.97-10.6) | **11.0** (9.98-11.9) | **12.7** (11.4-13.8) | **14.1** (12.5-15.4) |
| 4-day | **3.95** (3.70-4.24) | **4.74** (4.43-5.07) | **5.83** (5.44-6.25) | **6.73** (6.27-7.21) | **8.01** (7.43-8.58) | **9.07** (8.37-9.73) | **10.2** (9.35-10.9) | **11.4** (10.4-12.3) | **13.1** (11.8-14.2) | **14.5** (12.9-15.8) |
| 7-day | **4.59** (4.32-4.89) | **5.47** (5.14-5.83) | **6.67** (6.26-7.10) | **7.65** (7.17-8.14) | **9.05** (8.44-9.62) | **10.2** (9.46-10.8) | **11.4** (10.5-12.2) | **12.7** (11.6-13.5) | **14.5** (13.1-15.6) | **16.0** (14.4-17.3) |
| 10-day | **5.21** (4.92-5.52) | **6.19** (5.84-6.57) | **7.47** (7.04-7.92) | **8.52** (8.01-9.02) | **9.99** (9.36-10.6) | **11.2** (10.4-11.9) | **12.4** (11.5-13.2) | **13.7** (12.6-14.6) | **15.6** (14.2-16.7) | **17.1** (15.5-18.4) |
| 20-day | **6.98** (6.59-7.39) | **8.24** (7.80-8.72) | **9.74** (9.21-10.3) | **10.9** (10.3-11.6) | **12.6** (11.8-13.3) | **13.9** (13.0-14.7) | **15.2** (14.2-16.1) | **16.5** (15.4-17.6) | **18.3** (16.9-19.6) | **19.8** (18.1-21.2) |
| 30-day | **8.58** (8.12-9.04) | **10.1** (9.57-10.6) | **11.8** (11.2-12.4) | **13.1** (12.4-13.8) | **14.9** (14.0-15.7) | **16.2** (15.3-17.1) | **17.5** (16.5-18.5) | **18.9** (17.6-20.0) | **20.6** (19.1-21.9) | **21.9** (20.2-23.4) |
| 45-day | **10.7** (10.2-11.2) | **12.5** (11.9-13.2) | **14.4** (13.7-15.2) | **15.9** (15.1-16.7) | **17.8** (16.9-18.7) | **19.3** (18.2-20.3) | **20.7** (19.5-21.8) | **22.1** (20.7-23.3) | **23.8** (22.3-25.2) | **25.1** (23.4-26.7) |
| 60-day | **12.7** (12.2-13.3) | **14.9** (14.2-15.6) | **17.0** (16.2-17.8) | **18.6** (17.7-19.5) | **20.6** (19.6-21.6) | **22.1** (21.0-23.2) | **23.5** (22.2-24.7) | **24.8** (23.4-26.1) | **26.4** (24.9-27.9) | **27.6** (25.9-29.2) |

[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.

**Figure 3.3. Point Precipitation Frequency Estimates for Storm Durations Ranging from 5-Minutes to 60-Days with Average Recurrence Intervals (Return Periods) from 1-Year to 1000-Years for Blythewood, SC**

In the upper right-hand side of Figure 3.2 are location options: (a) Select location (move crosshair or double click) and b) Click on station icon. By clicking on the box followed by "show stations on map" official rain gauge stations are located on the map as shown in Figure 3.4. Data for specific stations are obtained by clicking on the station icon.

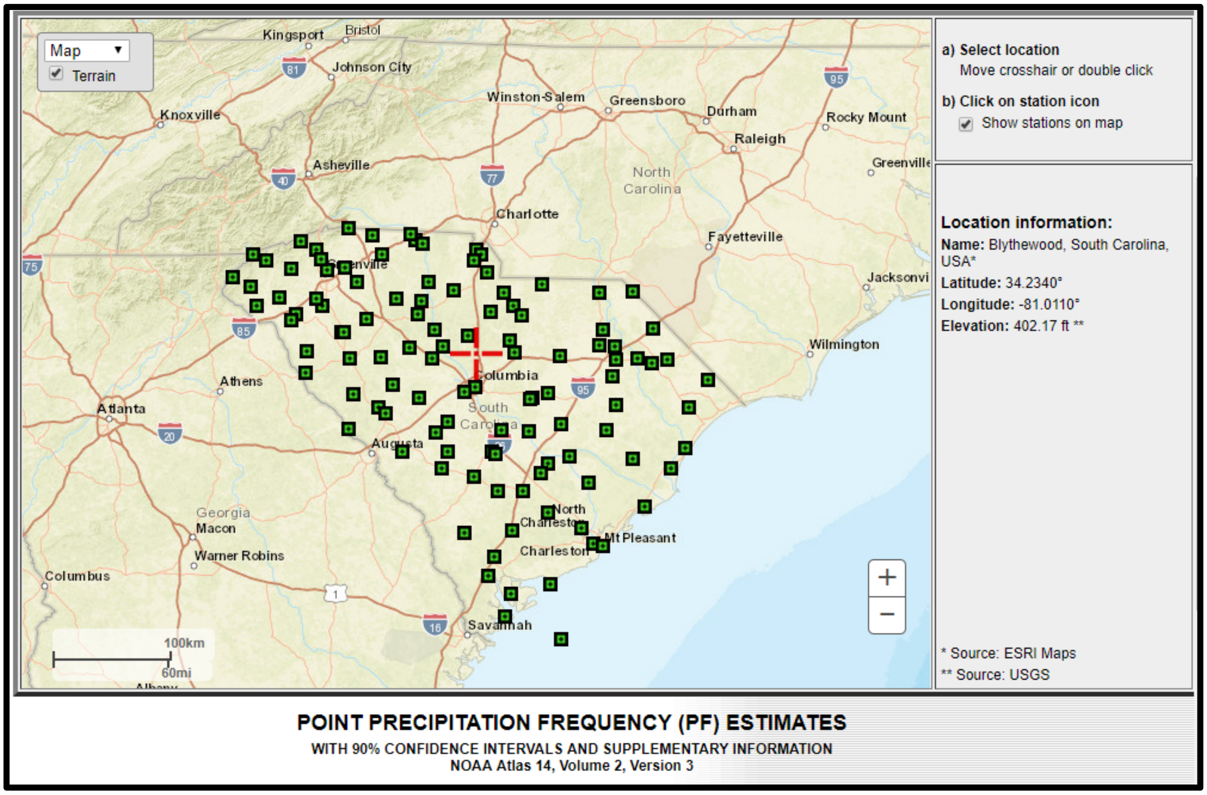



**Figure 3.4. Map Showing Icons for South Carolina Rain Gauge Stations**

### 3.2 Rainfall Distribution Curves for South Carolina

The recommended rainfall distribution curves for South Carolina are the NOAA A, B, C, and D curves documented in Supplement Number 1 to the Engineering Field Handbook Chapter 2 (EFH-2)[5]. These curves replace Types II and III rainfall distributions that were the standard for many years and are still used by some municipalities and agencies. SCDOT requires the use of the NOAA curves.

This Supplement to EFH-2 covers use of rainfall data developed by the NOAA Atlas 14 and rainfall distributions based on the NOAA Atlas14 data. The rainfall data and distributions replace data from Weather Bureau Technical Paper 40 (TP-40) and the standard NRCS rainfall distributions Type 1, Type 1A, Type II and Type III.

NOAA completed Volume 2 of Atlas 14 precipitation-frequency analysis in 2004[6]. It is the first

[5] Engineering Field Handbook Chapter 2: Estimating Runoff and Peak Discharges South Carolina EFH-2 Supplement Number 1

[6] NOAA periodically updates the data and analyses in Atlas 14.

comprehensive precipitation-frequency analysis for the Ohio Valley and neighboring states since TP-40 was completed in 1961. Data are available for specific locations on the website (http://hdsc.nws.noaa.gov/hdsc/pfds/). Data for representative locations in South Carolina counties are included in the rainfall database **county.SC** developed for use with the EFH-2 computer program. NOAA periodically updates the data and analyses in Atlas 14.

Designated rainfall distributions for South Carolina counties are shown on maps included in Appendix 1 of the Supplement to EFH-2. Map details include Approximate Geographic Boundaries, Primary Roads and County Splits. Figure 3.5 shows the South Carolina rainfall distribution map.

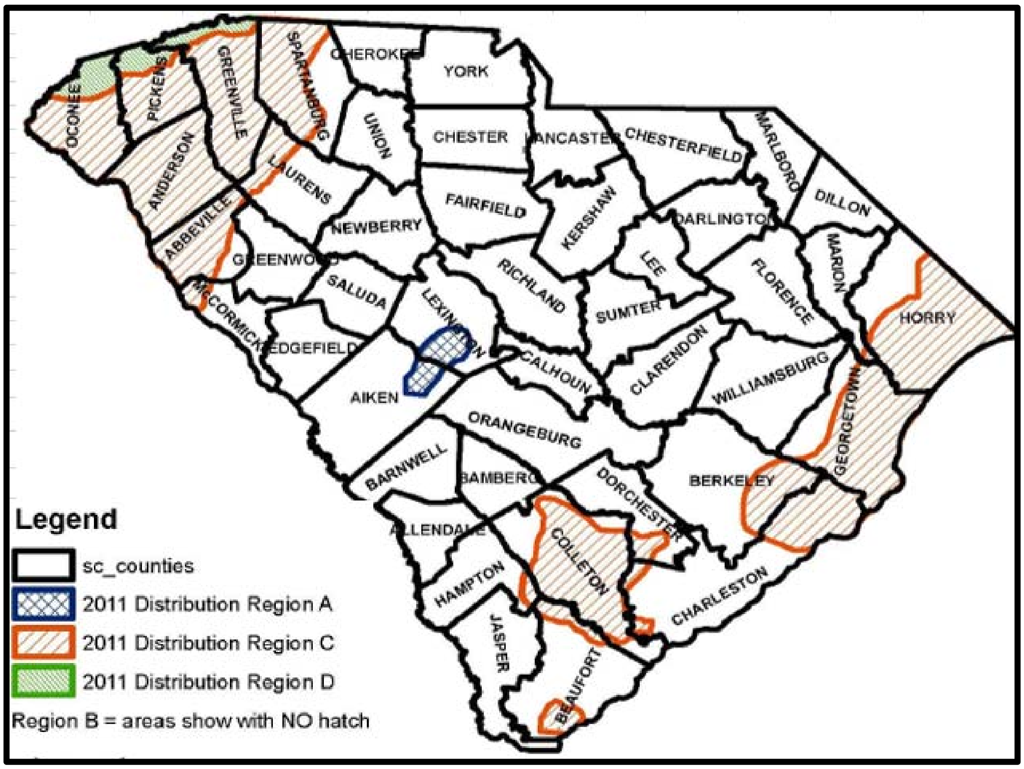



**Figure 3.5. South Carolina Rainfall Distribution Map**

Four rainfall distributions (NOAA A, B, C, and D) were developed for the Ohio Valley and neighboring states (DC, DE, IL, IN, KY, MD, NC, NJ, OH, PA, SC, TN, VA, and WV). All four extend into South Carolina as shown in Figure 3.5. The curves are shown in Figure 3.6.

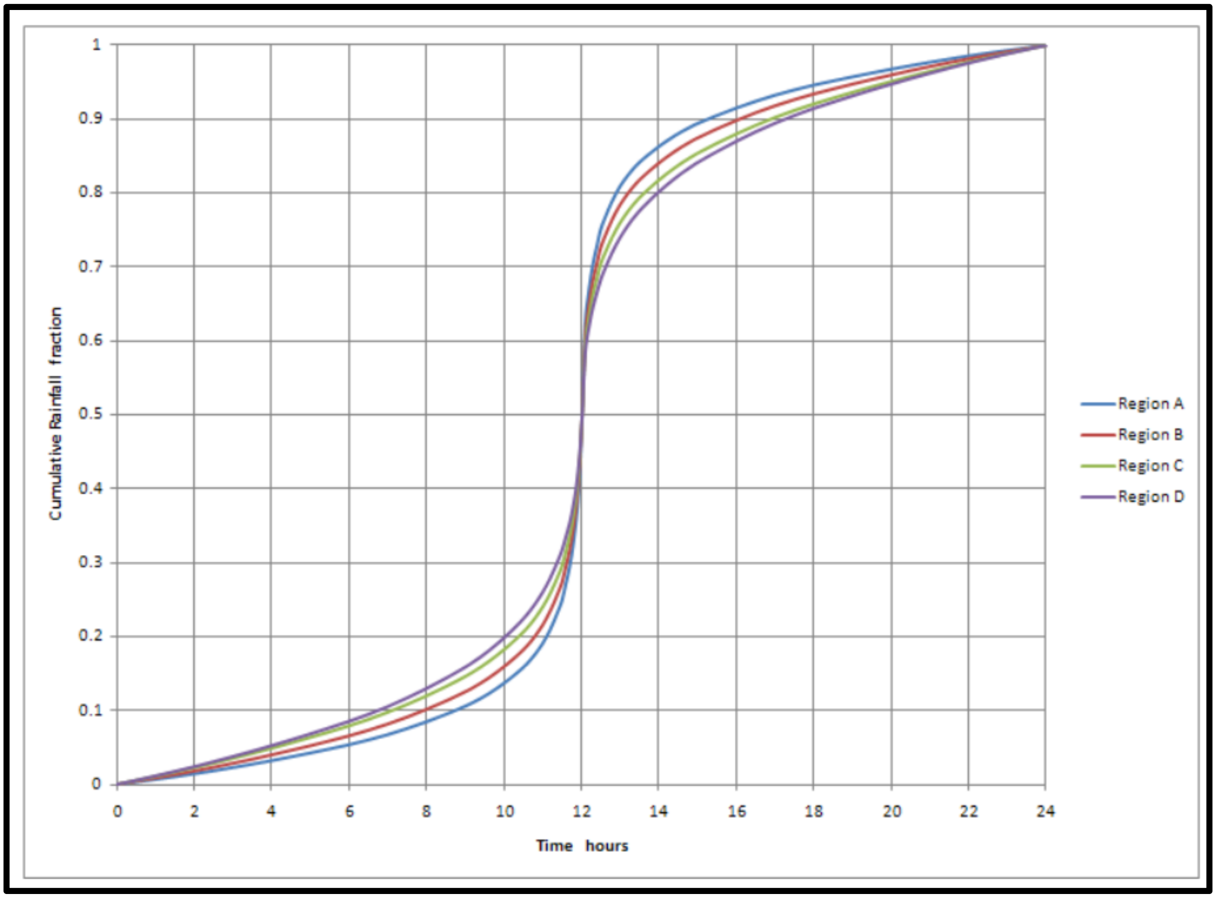



**Figure 3.6. NOAA Rainfall Distribution Curves for South Carolina**

Each of the NOAA A, B, C, and D distributions is symmetrically centered around 12 hours. To make a 1-hour rainfall distribution, extract and expand the distribution from 11.5 to 12.5 hours. To make a 2-hour distribution, extract and expand the distribution from 11 to 13 hours. Follow the same procedure to make D-hour distributions for rainfalls with duration less than or equal to 24 hours. The first example in Chapter 5 illustrates this procedure for 1 and 3-hour rainfalls.

### 3.3 NRCS Curvilinear Unit Hydrograph Method

The NRCS curvilinear unit hydrograph was developed based on the analysis of natural unit hydrographs from a wide range of watershed sizes and geographic locations. A natural unit hydrograph is one derived from measured event data, i.e., measured rainfall and runoff. The data were collected at experimental watersheds located in several states, physiographic regions[7], and climatic zones. To compare the many unit hydrographs, because of differences in scale, the NRCS

---

[7] The upstate of South Carolina lies mostly in the Piedmont physiographic province. From the Fall Line to the coast is in the Coastal Plain province, which is divided into the Upper or Inner Coastal Plain (roughly from the Fall Line to I-95) and the Lower or Outer Coastal Plain (from about I-95 to the coast).

formed each dimensionless, thereby removing scale effects. They plotted the dimensionless unit hydrographs on transparency paper, overlaid them onto a light table, and drew an average dimensionless unit hydrograph. The resulting unit hydrograph shape is curvilinear, hence the name. In the original documentation for this method and many contemporary references, it is presented in dimensionless form and identified as the NRCS dimensionless unit hydrograph. It also is known as the 484-unit hydrograph, because the peak rate factor value is 484.

To devise a methodology applicable to un-gaged watersheds, the NRCS regionalized the curvilinear unit hydrograph by developing statistical equations to estimate the model parameters in terms of map data, where map data refers to watershed physical measures determined from maps, other published information sources, and computer spatial databases and analysis tools such as GIS. The parameter prediction equations are used to compute unit hydrograph parameters, such as peak flow rate and time to peak, which are used to scale the dimensionless shape and get a unit hydrograph for the watershed under consideration. It must be noted the peak rate factor is always 484 and is not varied among watersheds. Consequently, the NRCS curvilinear unit hydrograph method predicts the same unit hydrograph shape for all watersheds.

A major reason for developing the SC UH Method is the NRCS UH method uses the same PRF at all watersheds. One major issue is when there has been a land use change and engineers are designing peak flow controls. Using the same PRF for pre- and post-land use change conditions results in over-estimating the pre-land use change peak flow which leads to wrongly designed controls. To illustrate, runoff hydrographs were simulated at a given watershed with changes only to PRF. The watershed land use is open space, rainfall depth is 3.13 inches, CN is 69.2, and UH time to peak is 30 minutes. Simulations were made for PRF values of 484 and 250 which is the SC UH PRF for open space. Simulation results with PRF equal to 484 were unit hydrograph peak flow rate equal to 37.8 cfs and runoff peak flow rate equal to 64.9 cfs. For PRF equal to 250 the unit hydrograph peak flow rate was 19.5 cfs and the runoff peak flow rate was 39.1 cfs. The difference between runoff peak flow rates (64.9-39.1=25.8 cfs) is extremely large and would result in grossly wrong stormwater management system design in terms of under-sizing or oversizing a culvert, a bridge, or a stormwater pond.

With respect to SCDOT, since few, if any, watersheds have PRF equal to 484, their predictions for hydrographs and peak flows at road crossings using the NRCS UH will lead to incorrectly sized culverts and bridges. Since most watershed PRF values probably are less than 484, the bridges or culverts would be overdesigned which is a positive for the crossing by lowering impacts and staying in operation during extreme events but could have a negative impact on downstream crossings. This is due to the routing effects at that crossing on streamflow hydrograph peaks and minimizing timing. This could lead to greater peak flows and earlier peak flow arrivals downstream. If the peak flow arrival time at a downstream location more closely aligns with peak flow arrivals from other contributing areas, the peak flow at that point will be much greater than the original peak flow and could lead to overtopping or failure of the structure at that crossing.

The ordinates for the NRCS dimensionless curvilinear unit hydrograph and the cumulative curve are shown in Table 3.1. Note the mass (volume) under the rising limb (to the point when $t/t_p=1.0$) is 0.375. This means 37.5% of the runoff occurs under the rising limb and 62.5% occurs under the recession limb. The two-parameter gamma distribution unit hydrograph with PRF=484 and shape parameter n=4.7 closely approximates the shape of the NRCS curvilinear unit hydrograph.

**Table 3.1**
**Ordinates of the NRCS Dimensionless Curvilinear Unit Hydrograph**

| Dimensionless Time $t/t_p$ | Dimensionless Flow $Q/Q_p$ | Cumulative Dimensionless Flow |
|---|---|---|
| 0 | 0.000 | 0.000 |
| 0.1 | 0.030 | 0.001 |
| 0.2 | 0.100 | 0.006 |
| 0.3 | 0.190 | 0.012 |
| 0.4 | 0.310 | 0.035 |
| 0.5 | 0.470 | 0.065 |
| 0.6 | 0.660 | 0.107 |
| 0.7 | 0.820 | 0.163 |
| 0.8 | 0.930 | 0.228 |
| 0.9 | 0.990 | 0.300 |
| 1.0 | 1.000 | 0.375 |
| 1.1 | 0.990 | 0.450 |
| 1.2 | 0.930 | 0.522 |
| 1.3 | 0.860 | 0.589 |
| 1.4 | 0.780 | 0.650 |
| 1.5 | 0.680 | 0.700 |
| 1.6 | 0.560 | 0.751 |
| 1.7 | 0.460 | 0.790 |
| 1.8 | 0.390 | 0.822 |
| 1.9 | 0.330 | 0.849 |
| 2.0 | 0.280 | 0.871 |
| 2.2 | 0.207 | 0.908 |
| 2.4 | 0.147 | 0.934 |
| 2.6 | 0.107 | 0.967 |
| 2.8 | 0.077 | 0.953 |
| 3.0 | 0.055 | 0.977 |
| 3.2 | 0.040 | 0.984 |
| 3.4 | 0.029 | 0.989 |
| 3.6 | 0.021 | 0.993 |
| 3.8 | 0.015 | 0.995 |
| 4.0 | 0.011 | 0.997 |
| 4.5 | 0.005 | 0.999 |
| 5.0 | 0.000 | 1.000 |

### 3.4 NRCS Curve Number Model

#### 3.4.1 Introduction

The recommended method to determine rainfall excess (direct storm runoff) for the SC UH Method is the NRCS curve number runoff model. Developed in the 1950s for internal use, the curve number method for estimating direct runoff from rainstorms is now widely used for applications such as engineering design, forensic analysis, and environmental impact studies. The Soil Conservation Service (SCS) originally published the Curve Number method in Chapters 9 and 10 of the NRCS National Engineering Handbook, Section 4, "Hydrology", or "NEH-4" (SCS, 1964). Chapter 9 was revised in 1969 and later amended in 1985. Chapter 10 was revised in 1972 and later amended in 1985. In the mid-1990s, NRCS reorganized the National Engineering Handbook into Parts and NEH-4 became NEH Part 630, Hydrology. In 2004, Chapters 9 and 10 were updated and incorporated into the NRCS National Engineering Handbook Part 630, Hydrology. The most current official versions of Chapters 9 and 10 are available through the NRCS eDirectives website, under the National Engineering Handbook. The NRCS CN runoff equation is

$$Q_{CN} = \frac{(P-I_a)^2}{P-I_a+S} = \frac{(P-0.2S)^2}{P+0.8S} \tag{3.1}$$

where

$$S = \frac{1000}{CN} - 10 \tag{3.2}$$

and

$$I_a = 0.2S \tag{3.3}$$

$Q_{CN}$[8] is runoff volume in watershed inches, P is the cumulative rainfall depth in inches, CN is curve number, S is watershed retention, and $I_a$ is initial abstractions which include rainfall lost to interception by vegetation, rooftops, etc., depression storage, and initial high rate infiltration (Figure 3.7).

[8] $Q_{CN}$ is the author's terminology. NRCS uses Q which also is the variable for volumetric flow rate. The author uses $Q_{CN}$ in his course notes and technical literature to avoid confusion on behalf of the readers and users.

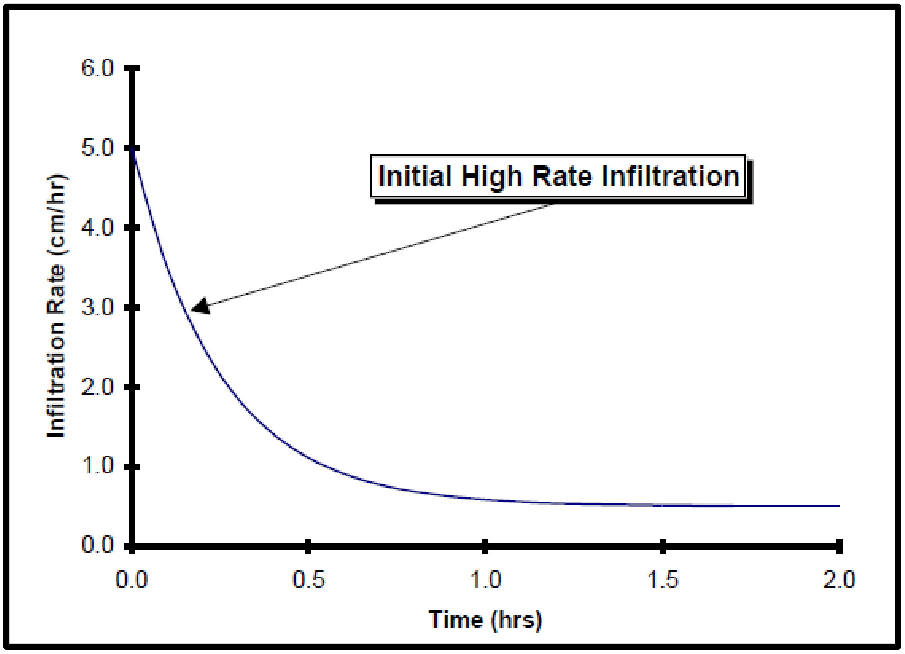



**Figure 3.7. Typical Field Infiltration Curve**

### 3.4.2 Curve Numbers

Curve numbers are given in tables in NEH-4, TR-55 (SCS, 1986)[9], design manuals, and hydrology texts in terms of land use, hydrologic soil group (soil type), watershed wetness, and surface cover conditions. Curve number is defined as an index of watershed runoff potential. As such its value ranges between 0 for no runoff and 100 for total runoff (all rainfall goes to runoff). Locations with zero curve number have infiltration rates greater than rainfall intensity, such as the area in and around the Aiken Horse Farms where there are sandy soils and great depth to the groundwater table, retention ponds, and wetlands with no outflow.  Practically, the maximum curve number for impervious surfaces is 98. CN equal to 100 is unlikely as some water will be retained on the watershed by surface tension. The CN will be 100 for rain over an area such as a lake, river, and flooded wetland. Most watersheds have curve numbers greater than 50 and generally in the range from 60 to 90. Table 3.2 lists curve numbers for multiple land uses.

NRCS soil scientists classified more than 4000 soils based on their runoff potential and grouped them into four hydrologic soil groups (HSG) identified by the letters A, B, C, and D. HSG-A soils are soils with high infiltration rates and low runoff potential. HSG-D soils are soils with low infiltration rates and high runoff potential. Table 3.3 lists soil characteristics for each HSG classification.

---

[9] TR-55 was revised and completely rewritten as a windows-based program WinTR-55. The program and supporting materials may be downloaded from http://www.wsi.nrcs.usda.gov/products/W2Q/H&H/Tools_Models/ WinTR55.html.

Information about HSG classification for watershed soils can be found in NRCS county soil surveys, which contain maps showing soil types. A table in the soil survey associates each soil type and HSG classification. This information is available on-line at the Web Soil Survey: http://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm. Appendix A in TR-55 also gives the HSG information for classified soils.

The CN method of estimating runoff volumes from rainfall is simple and easy to use. It was developed from a large amount of data collected at experimental watersheds across the United States. It works well for a wide range of soil-cover complexes. Although documentation is somewhat limited, it is used in a wide range of design conditions by practicing engineers and hydrologists. Research is on-going to expand the understanding and range of applications of the CN Model which will further its use and longevity.

**Table 3.2**
**Selected NRCS (SCS) Curve Numbers for Average Runoff Conditions ($I_a$=0.2S)**

| Land Use | | Hydrologic Soil Group Classification | | | |
|---|---|---|---|---|---|
| | | A | B | C | D |
| Open Space | | | | | |
| Poor Condition (grass cover < 50%) | | 68 | 79 | 86 | 89 |
| Fair Condition (grass cover 50-75%) | | 49 | 69 | 79 | 84 |
| Good Condition (grass cover > 75%) | | 39 | 61 | 74 | 80 |
| Impervious Areas (paved parking lots, roofs, etc.) | | 98 | 98 | 98 | 98 |
| Streets and Roads | | | | | |
| Paved with curbs and storm sewers | | 98 | 98 | 98 | 98 |
| Paved with open ditches | | 83 | 89 | 92 | 93 |
| Gravel | | 76 | 85 | 89 | 91 |
| Dirt | | 72 | 82 | 87 | 89 |
| | Average % Impervious | | | | |
| Urban Land Use | | | | | |
| Commercial and Business | 85 | 89 | 92 | 94 | 95 |
| Industrial | 72 | 81 | 88 | 91 | 93 |
| Residential | | | | | |
| 1/8 Acre | 65 | 77 | 85 | 90 | 92 |
| 1/4 Acre | 38 | 61 | 75 | 83 | 87 |
| 1/3 Acre | 30 | 57 | 72 | 81 | 86 |
| 1/2 Acre | 25 | 54 | 70 | 80 | 85 |
| 1 Acre | 20 | 51 | 68 | 79 | 84 |
| 2 Acres | 12 | 46 | 65 | 77 | 82 |
| Developing urban areas, newly graded, no grass cover | | 77 | 86 | 91 | 94 |
| Pasture | | | | | |
| Poor | | 68 | 79 | 86 | 89 |
| Fair | | 49 | 69 | 79 | 84 |
| Good | | 39 | 61 | 74 | 80 |
| Woods | | | | | |
| Poor Condition | | 57 | 73 | 82 | 86 |
| Fair Condition | | 43 | 65 | 76 | 82 |
| Good Condition | | 32 | 58 | 65 | 78 |
| | Hydrologic Condition | | | | |
| Row Crop | | | | | |
| Straight Row | Poor | 72 | 81 | 88 | 91 |
| | Good | 67 | 78 | 85 | 89 |
| Contoured | Poor | 70 | 79 | 84 | 88 |
| | Good | 65 | 75 | 82 | 86 |
| Contoured and Terraced | Poor | 66 | 74 | 80 | 82 |
| | Good | 62 | 71 | 78 | 81 |

**Table 3.3**
**Characteristics of Soils According to HSG Classification**

| HSG Class | Soil characteristics | Minimum Infiltration Rate (in/hr) |
|---|---|---|
| A | Deep sand, deep loess, aggregated silts | 0.30-0.45 |
| B | Shallow loess, sandy loam | 0.15-0.30 |
| C | Clay loams, soils low in organic content, soils high in clay content | 0.05-0.15 |
| D | Soils that swell when wet | 0.00-0.05 |

### 3.4.3 Curve Number Modification for Storm Durations Less Than 24 hours

Curve Number modification for storm durations less than 24 hours is a new concept. Beginning around 2014, William Merkel, Hydraulic Engineer with the NRCS, and Richard McCuen, Civil Engineering Professor at the University of Maryland, worked independently to develop duration-based CN adjustment methods. Their reasoning/motivation and methods are described in the following sections. They did not publish their methods. The only known publication is a paper by Meadows (2016) who had email communications with Merkel and McCuen about UH PRF issues and learned about their methods which they shared. After much evaluation, it was decided to incorporate duration-based CN modification into the SC UH Method.

For users of the SC UH Method, an obvious question is which method to use. Numerical experience suggests **Do Not Use the Merkel Method if the 24-hour CN is 65 or less.** In that case, use the McCuen Method. Overall, the two methods yield similar values for most applications. Users can apply the associated spreadsheet to compare the predicted runoff volumes and hydrographs and make their own decision.

#### 3.4.3.1 NRCS CN Modification—Merkel Method

The primary use of the NRCS curve number is to calculate total storm runoff based on total storm rainfall. It must be noted rainfall duration is not factored into the calculation. The method was originally created to determine the mean daily depth of runoff during flood producing events on small agricultural watersheds. CN values were determined using daily rainfall and runoff data. Practically, it did not rain for 24 hours during many, perhaps most, of the events, but since the data were recorded as daily rainfall, 24 hours became the implicit duration for values input to the curve number runoff model. NRCS references do not specifically state the CN applies only to the 24-hour storm. However, it may be inferred from what is published the standard CN applies to the 24-hour duration storm.

As explained by William Merkel, Hydraulic Engineer Retired, USDA-Natural Resources Conservation Service, you should not use the standard curve number for any duration other than 24 hours. If you do, you need to increase it for durations less than 24 hours and decrease it for durations longer than 24 hours. A basic hydrologic principle is that after initial abstractions have been satisfied, water infiltrates into the soil at nearly a steady rate. For a given rainfall depth, if the event duration is extended over a longer time, more rainfall will infiltrate. If the storm occurs over a shorter duration, less rainfall infiltrates and more goes to runoff. This concept was explained by Merkel as follows. "At a watershed with CN value of 80, for 4 inches of rainfall, the runoff is 2.04 inches. For rainfall duration of 1 hour, the runoff would be 2.04 inches and for 24 hours rainfall duration, the runoff also would be 2.04 inches. If you use the standard curve number for a 60-minute storm, it assumes that you have 24 hours of infiltration in just 60 minutes. In these modern times, this concept is technically invalid."

What should be done is to increase the curve number for rainfall events with durations less than 24 hours. A basic concept is to assume the same initial abstraction will occur for all durations. This obviously would not work for durations of 15 minutes or less, but probably is practical for durations of 1 through 24 hours. The second part of the analysis is to assume the relationship of time and infiltration can be related to the value of S (maximum retention). This allows variation of the infiltrated amount based on storm duration and estimation of how much the CN would increase from the 24-hour base value.

Table 3.4 shows steps to compute the adjusted CN value for storm durations less than 24 hours. The computational procedure follows recommendations by William Merkel and is identified as the Merkel method or the NRCS CN adjustment method. For this example, the standard CN is 75 and the storm duration D is 3 hours. Standard CN refers to the CN one obtains from the NRCS CN table based on watershed land use and soils information or it can be the runoff or area weighted CN one computes for a watershed with mixed soils and/or land uses. In this worksheet, this value is labelled 24-hour CN. The objective of the following calculations is to compute the 3-Hour CN. As described above, this value will be higher than the standard 24-hour CN.

**Table 3.4**
**Worktable Modifying Standard CN for a 3-Hour Storm Using the Merkel Method**

| | | |
|---|---|---|
| Line 1 | 24-hr CN = | 75 |
| Line 2 | 24-hr S = | 3.33 |
| Line 3 | 24-hr $I_a$ = | 0.67 |
| Line 4 | Rainfall Duration (D<=24) = | 3 |
| Line 5 | Event Rainfall Depth P (inches) = | 2.50 |
| Line 6 | 24-hr $Q_{CN}$ = | 0.65 |
| Line 7 | 24 hr Infiltration Volume (inches) = P – 24-hr $I_a$ – 24-hr $Q_{CN}$ = | 1.18 |
| Line 8 | 24-hr Infiltration Rate (in/hr) = | 0.05 |
| Line 9 | 3-hr Infiltration (inches) = | 0.15 |
| Line 10 | 3-hr Infiltration plus $I_a$ = | 0.81 |
| Line 11 | 3-hr $Q_{CN}$ (inches) = | 1.69 |
| Line 12 | 3-hr CN = | 91.9 |

Line 1 Enter the standard 24-hr CN value selected from the appropriate table based on land use and HSG classification or the runoff or area weighted CN computed for a watershed with mixed soils and/or land uses.

Line 2 Calculate the 24-hr retention as $24\,\text{hr S} = \frac{1000}{24\,\text{hr CN}} - 10$. (3.4)

Line 3 Calculate the 24-hr Initial Abstractions as $I_a = 0.2 * 24\,\text{hr S}$. (3.5)

Line 4 Enter design storm duration of less than 24 hours.

Line 5 Enter the design rainfall depth obtained from a source such as the Precipitation Data Frequency Server.

Line 6 Calculate the runoff for the design storm using the 24-hr CN equation, Eq 3.6.

$$24\,\text{hr } Q_{CN} = \frac{[\ P - 0.2 * (24\,\text{hr S})\ ]^2}{P + 0.8 * (24\,\text{hr S})} \tag{3.6}$$

Line 7 Calculate the 24-hr infiltration volume by subtracting the 24-hr Initial Abstractions and the 24-hr $Q_{CN}$ from the design storm rainfall depth: P-Ia-24-hr $Q_{CN}$.

(Line 5 – Line 3 – Line 6)

Line 8 Divide Line 7 by 24 to get the 24-hr average steady infiltration rate.

Line 9 Multiply Line 8 by storm duration (D = 3-hr) to get the D-hr infiltration volume.

Line 10 Enter the D-hr infiltration volume calculated in Line 9 plus $I_a$.

Line 11 Calculate the D-hr runoff volume (WS inches) as D-hr rainfall depth minus the sum of D-hr infiltration volume and Initial Abstractions.

Line 12 Calculate the D-hr Curve Number with this equation.

$$\text{D hr CN} = \frac{1000}{10 + 5P + 10 * (\text{D} - \text{hr } Q_{CN}) - 10 * [(\text{D} - \text{hr } Q_{CN})^2 + 1.25 * P * (\text{D} - \text{hr } Q_{CN})]^{0.5}} \tag{3.7}$$

Table 3.5 is the standard table to adjust standard CN for a 1, 2, 3, 6, and 12-hour storms using the NRCS CN adjustment method (Merkel method).

**Table 3.5**
**Worktable to Modify Standard CN for 1, 2, 3, 6, and 12-Hour Storms**

| | 24-hour | | | | |
|---|---|---|---|---|---|
| 24-hr CN = | | | | | |
| 24-hr S = | | | | | |
| 24 hr-$I_a$ = | | | | | |
| D = | 1 Hour | 2 Hour | 3 Hour | 6 Hour | 12 Hour |
| P = | | | | | |
| 24-hr $Q_{CN}$ = | | | | | |
| P-$I_a$-24-hr $Q_{CN}$ = | | | | | |
| 24-hr Infiltration Rate (in/hr) = | | | | | |
| D-hr Infiltration (inches) = | | | | | |
| D-hr Infiltration plus $I_a$ = | | | | | |
| D-hr Runoff (inches) = | | | | | |
| D-hr CN = | | | | | |

A significant impact of adjusting CN for durations less than 24-hours is illustrated by the values in Table 3.6. The rainfall data in the second column were obtained from the PFDS for Blythewood, SC for a 10-year return period storm. The rainfall depths are for 1, 2, 3, 6, 12, and 24-hour storms. There are two general columns labeled Duration Modified and Standard CN Not Duration Modified. The two columns under the Duration Modified heading list the Duration Modified CN and the runoff depth determined with the Duration Modified CN for each D-hr storm. The three columns under the Standard CN Not Duration Modified heading list the standard CN, which is 74 for all storms, the runoff depths for each storm determined with the standard CN, and the percent difference, labeled Std $Q_{CN}$ Error, between the runoff depths computed with the Duration Modified CN and the Standard CN. Note the Standard CN underpredicts the runoff depths for all events with durations less than 24 hours. The magnitude of the underprediction increases significantly with shorter storm durations.

**Table 3.6**
**Comparison of Runoff Depths Calculated with Duration Modified Curve Numbers and Standard 24-hr Curve Number**

| | | **Duration Modified** | | **Standard CN Not Duration Modified** | | |
|---|---|---|---|---|---|---|
| **D hrs** | **D-hr P** | **D-hr CN** | **D-hr $Q_{CN}$** | **CN** | **Std $Q_{CN}$** | **Std $Q_{CN}$ Error** |
| 1 | 2.50 | 92.6 | 1.75 | 74.0 | 0.61 | -65.1% |
| 2 | 2.92 | 92.2 | 2.10 | 74.0 | 0.86 | -59.2% |
| 3 | 3.11 | 91.6 | 2.23 | 74.0 | 0.98 | -56.1% |
| 6 | 3.70 | 89.5 | 2.59 | 74.0 | 1.38 | -46.7% |
| 12 | 4.38 | 84.8 | 2.78 | 74.0 | 1.88 | -32.4% |
| 24 | 5.25 | 74.0 | 2.57 | 74.0 | 2.57 | 0.0% |

### 3.4.3.2 Curve Number Modification—McCuen Method

The NRCS rainfall-runoff model has limited use in the analysis of small volume, short duration storms because it was developed primarily for flood producing events. Short duration storms are becoming increasingly important because of their association with water quality issues such as first-flush events. Also, some locations consider shorter duration events as critical and include them in their list of design events.

A study was conducted by Richard McCuen and his research associates at the University of Maryland to incorporate storm duration into the NRCS rainfall-runoff model so the rainfall-runoff model can be applied to short-duration storm events. The specific objectives of the work were to: (1) revise the maximum potential retention component to incorporate storm duration and (2) evaluate the accuracy of the revised method using short duration events. The revised model was found to increase the accuracy of computed runoff depths and yields runoff depths for small rainfalls for which the standard model computes zero runoff. The revised model has the added advantage that it allows the initial abstraction to vary with storm duration without modifying the initial abstraction coefficient.

To adjust the existing NRCS-CN model to accommodate the storm duration, Equation 2 was modified by replacing the empirical value of 10 with the variable $\gamma$:

$$S = \frac{1000}{CN} - \gamma \tag{3.8}$$

The term $\gamma$ is a function of both storm duration and curve number.

To develop an expression for $\gamma$ that is a function of storm duration, three constraints were considered. First, $\gamma$ must be less than or equal to $\frac{1000}{CN}$. Second, $\gamma$ must be equal to 10 for a storm with duration of 24 hours. Third, $\gamma$ must be equal to 10 when the curve number is 100. The first constraint prevents negative values of S that are not physically realistic. The second constraint ensures the model with the adjusted S computes the same depth of runoff for 24-hour storms as the standard NRCS CN model. The third constraint ensures that when the curve number is 100, the equation yields an S value of zero. Empirical analyses indicated the model should display the following relationships: (1) duration and S are directly related so as duration decreases from 24 hours, S decreases; (2) $\gamma$ and S are inversely related so as $\gamma$ increases, S decreases; and (3) curve number and $\gamma$ are inversely related so $\gamma$ increases as the curve number decreases.

The three constraints were structured into an expression that relates $\gamma$ to the storm duration (D) and 24-hr CN. First, the general linear model related $\gamma$ to D:

$$\gamma = a - bD \tag{3.9}$$

where a and b are best-fit coefficients. Empirical evidence indicated the coefficient b is inversely proportional to the CN:

$$b = c + \frac{d}{CN} \tag{3.10}$$

where c and d are best-fit coefficients. Using the three constraints (i.e., $\frac{1000}{CN} > \gamma$, γ = 10 when D = 24, and γ = 10 when CN = 100), Eqs. 3.8, 3.9, and 3.10 can be combined into the following equation:

$$\gamma = 10 + d\left(\frac{100}{CN} - 1\right)(24 - D) \tag{3.11}$$

where d is a coefficient to be empirically determined.

The empirical coefficients a, b, and c are quantified by the three constraints. Equation 3.11 represents a linear function of γ. The $(24-D)$ term will place too much emphasis on duration, especially for very short durations. The D term has more weight than the CN term and this may result in an inaccurate estimation of γ. To reduce its influence, an exponent (e) was applied to the (24−D) term:

$$\gamma = 10 + d\left(\frac{100}{CN} - 1\right)(24 - D)^e \tag{3.12}$$

To determine values for d and e, Equation 3.12 was calibrated using data monitored at USDA experimental watersheds. The resulting equation is:

$$\gamma = 10 + 0.00256(98 - CN)^{5/3}(24 - D)^{0.5} \tag{3.13}$$

Table 3.7 shows steps to compute the adjusted CN value for storm duration less than 24 hours using the McCuen method. This example uses the same data and equivalent computations as the NRCS CN adjustment. The standard CN is 75 and storm duration D is 3 hours. As a comparison, the NRCS adjusted CN is 91.9 and the McCuen adjusted CN is 89.7.

**Table 3.7**
**Worktable Modifying Standard CN for a 3-Hour Storm Using the McCuen Method**

| | | |
|---|---:|---|
| Line 1 | 24-hr CN = | 75 |
| Line 2 | D-hr = | **3** |
| Line 3 | D-hr P = | 2.50 |
| Line 4 | Gamma = | 12.18 |
| Line 5 | D-hr S = | 1.15 |
| Line 6 | D-hr $Q_{CN}$ = | 1.51 |
| Line 7 | Use D-hr P and D-hr $Q_{CN}$ to compute D-hr CN = | 89.7 |

Line 1 Select value from NRCS CN table based on land use and HSG classification

Line 2 Rainfall duration is D-hr

Line 3 D-hr rainfall depth

Line 4 Compute Gamma using Equation 3.13.

Line 5 $S = \frac{1000}{24\text{ hr CN}} - \gamma$ (3.14)

Line 6 Use D-hr P and D-hr S in the CN Runoff Model

$$\text{D}-\text{hr Q}_{\text{CN}} = \frac{[\ \text{P} - 0.2 * (\text{D} - \text{hr S})\ ]^2}{\text{P} + 0.8 * (\text{D} - \text{hr S})} \tag{3.15}$$

Line 7 Use D-hr P and D-hr $Q_{CN}$ to compute D-hr CN with Equation 3.7

$$\text{D hr CN} = \frac{1000}{10 + 5\text{P} + 10 * (\text{D} - \text{hr Q}_{\text{CN}}) - 10 * [(\text{D} - \text{hr Q}_{\text{CN}})^2 + 1.25\text{P} * (\text{D} - \text{hr Q}_{\text{CN}})]^{0.5}} \tag{3.7}$$

Or use a simpler equation: $\text{D} - \text{hr CN} = \frac{1000}{10 + \text{D} - \text{hr S}}$ (3.16)

### 3.4.3.3 Justification for Modifying CN Value for Durations Less Than 24 Hours

Modifying the CN value for storm durations less than 24 hours is supported by rank-ordered analysis of data at USDA experimental watersheds. Rank ordering was/is used during CN determination of rainfall and runoff data sets. Figure 3.8 is an example plot of rank-ordered CN values and storm durations. The equation shown on the graph is a linear trend line. Similar plots and trendlines at regional USDA experimental watersheds affirm CN decreases with duration.

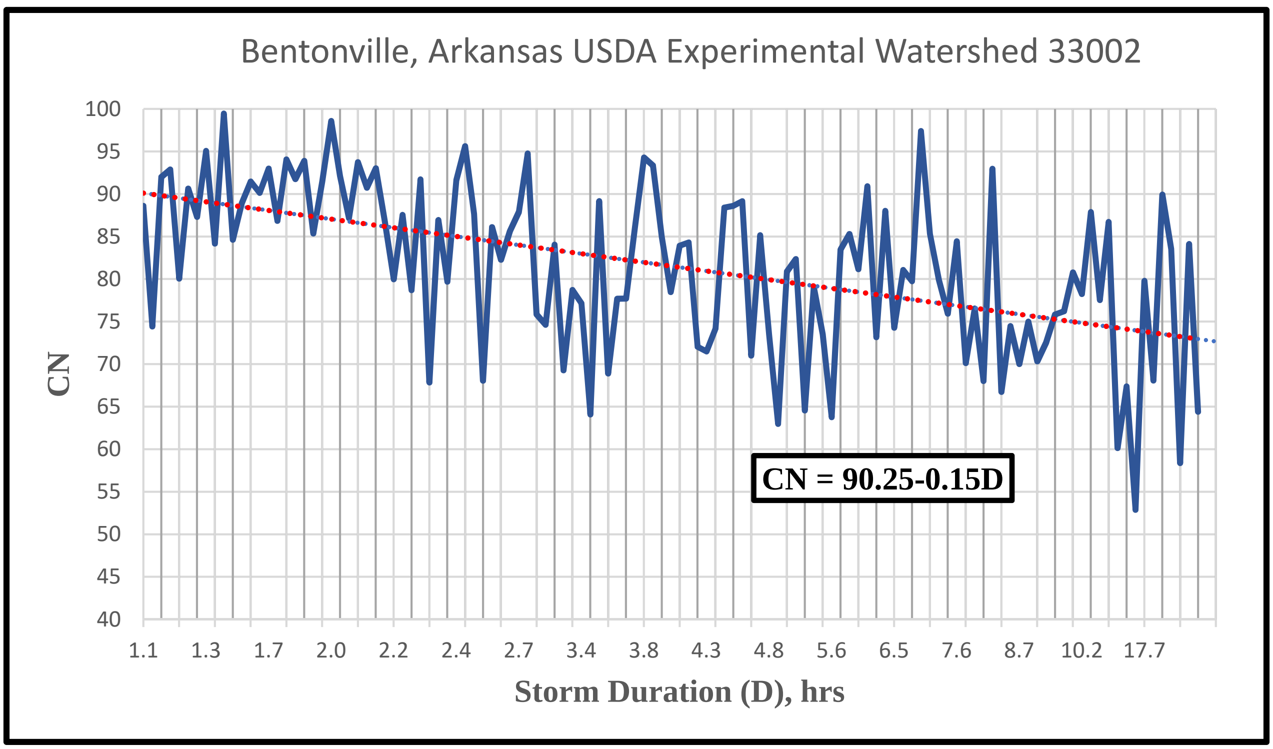



**Figure 3.8. Plot of Rank Ordered CN and Storm Duration Data**

### 3.4.4 Future Changes to the NRCS Curve Number Runoff Model

A national committee was formed and worked together during 2016 and 2017 to assess updates to the NRCS CN model. An outcome of this effort was the revision of four chapters in the NRCS National Engineering Handbook, Part 630 Hydrology. These chapters are: Chapter 8: Land Use and Land Treatment Classes; Chapter 9: Hydrologic Soil-Cover Complexes; Chapter 10: Estimation of Direct Runoff from Storm Rainfall; and Chapter 12: Hydrologic Effects of Land Use and Land Treatment. The upgraded chapters were submitted and are under consideration by NRCS for inclusion in a revised version of the NRCS Handbook.

Two significant recommendations are:

1. Use $I_a = 0.05S$ instead of $I_a = 0.20S$. This changes all tables and charts based on the original $I_a = 0.20S$ assumption. It also redefines S to a different value because the limit difference between the natural P and $Q_{CN}$ is no longer 1.20S, but 1.05S. Empirical relationships between the two "S" values, $S_{05}$ and $S_{20}$, are provided.
2. Use Runoff Weighted CN values instead of Area Weighted CN values to determine the watershed CN.

These recommended changes are a guide and not mandatory until formally approved and accepted by NRCS, which is very likely. The recommended upgrades are supported by data analyses, literature references, and the wisdom gained from prior handbooks. The contents are tempered by the professional opinions and experiences of the CN Committee members.

#### 3.4.4.1 Relationship of $I_a$ to S

Two primary parameters in the CN runoff model are watershed retention, S, and initial abstractions, $I_a$. Data analysis to develop the original model determined the long-used value of $I_a = 0.20S$. Later research (e.g., Jiang, 2001) found a more appropriate relation is

$$I_a = 0.05S \qquad (3.17)$$

This relationship and associated changes are introduced and emphasized in the NEH update. This impacts CN values, the CN model, and all applications of CN and the model.

The new $I_a/S$ relationship changes the CN runoff model to

$$Q_{CN} = \frac{(P-0.05S_{05})^2}{P+0.95S_{05}} \quad \text{for } P \geq 0.05S_{05} \qquad (3.18)$$

and

$$Q_{CN} = 0 \quad \text{for } P \leq 0.05S_{05} \qquad (3.19)$$

The subscript 05 indicates use of $I_a/S=0.05$ in contrast to the original value of 0.20.

### 3.4.4.2 CN Conversion from $I_a$=0.20S to $I_a$=0.05S

Based on recommendations of the CN Committee, CN values can be converted from the original $I_a$=0.20S to $I_a$=0.05S with the following equations.

$$S_{0.05} = 1.42S_{0.20} \tag{3.20}$$

Substituting Equation 3.20 into Equation 3.2 yields

$$CN_{0.05} = CN_{0.20}/(1.42-0.0042CN_{0.20}) \tag{3.21}$$

Equations 3.20 and 3.21 are given in Chapter 10 Estimation of Direct Runoff from Storm Rainfall of the pending revision to the NRCS National Engineering Handbook.

As explained by Richard Hawkins[10], Chair of the CN Committee, a potentially better equation has been developed that could replace Equation 3.20 pending opinions of the NRCS. The equation is

$$S_{05}=1.3244(S_{20})^{1.089} \tag{3.22}$$

To illustrate CN conversion from $I_a$=0.20S to $I_a$=0.05S**, c**alculate direct runoff from a storm of 3.00 inches using the $I_a$=0.05S and 0.20S versions of the CN model. The watershed area is a 100-acre pasture with $CN_{0.20} = 69$.

Using the original CN model with $I_a/S=0.20$, $CN_{0.20} = 69$, $S_{0.20}$=4.49 inches, and $I_a$=0.89 inches, the runoff is

$$Q_{0.20} = (3.00-0.20(4.49))^2/ (3.00+0.8(4.49)) = 0.67 \text{ inches}$$

The subscript 0.20 refers to the $I_a$=0.20S version of the CN model.

For the revised CN model with $I_a/S=0.05$

$$S_{0.05} = 1.42S_{0.20} = 1.42(4.49) = 6.38 \text{ in}$$

$$CN_{0.05} = CN_{0.20}/(1.42-0.0042CN_{0.20}) = 69/ (1.42-0.0042*69) = 61.1$$

$$Q_{0.05} = (P-0.05S_{0.05})^2/(P+0.95S_{0.05}) \text{ for all } P>0.05\, S_{0.05}$$

For this equation, $0.05S_{0.05} = 0.05(6.38) = 0.32$ inches; $0.95S_{0.05} = 0.95(6.38) = 6.06$ inches. The event runoff is

---

[10] Email communication between Richard Hawkins and the author.

$Q_{0.05}$ = 0.79 inches.

Clearly, $Q_{0.05}$ is not the same as $Q_{0.20}$ and it is not expected to be equal. Also, runoff is generated at lower P values for $CN_{0.05}$, which implies the revised CN model is more conservative for design.

#### 3.4.4.3 Updated NRCS Curve Number Table

Table 3.8 lists $CN_{0.05}$ values for different land uses. These values were determined using Equation 3.21 to convert $CN_{0.20}$ values to $CN_{0.05}$ values. That table was created for this Manual and may not label all land uses the same as shown in the Revised NEH Chapter 9.

#### 3.4.4.4 Runoff Weighted CN Values

The original CN method was developed for application to small drainage areas, assumed to have constant (i.e., “lumped”) properties throughout. Natural watersheds are mixtures of different land uses and soils and different contributing CNs. This mixture is particularly true for larger watersheds. The previous practice has been to average, on an Area Weighted basis, the assigned CNs and uses the average CN in the calculation of runoff from the entire watershed. Using Area Weighted CNs does not account for the effects of extremes such as the higher CN portions of the watershed and rainfall and CN conditions close to the threshold of runoff. This problem can be avoided using what is termed CN determined with the Runoff Weighted method. That approach was suggested by the NRCS National Hydraulic Engineer and the CN model update committee and is included in the spreadsheet associated with this Manual.

Example calculations in Chapter 5 show the application of CNs determined with the Area Weighted and Runoff Weighted methods to determine watershed runoff. To compute area weighted CN values, for each land use and HSG combination multiply the CN by the area. Do this for all land use and HSG combinations. Divide the sum of products by total watershed area. To compute runoff weighted CN values, use the CN for each land use and HSG combination to compute the event runoff with the CN Runoff Model. Do this for all land use and HSG combinations. Multiply the computed runoff from each land use and HSG combination by the area of that combination. Sum the products and divide the sum by the total watershed area. As expected, the CN obtained with the Runoff Weighted method yields greater runoff.

### 3.4.5 Curve Number Adjustments for Antecedent Runoff Conditions

Runoff is affected by the soil moisture before a precipitation event. Antecedent Moisture Condition (AMC) is the relative moisture of the pervious surfaces prior to the rainfall event being considered. Stated differently it is the degree of watershed wetness at the start of a storm.

AMC is now termed Antecedent Runoff Condition (ARC). The watershed is either wet (ARC III), dry (ARC I), or under average conditions (ARC II).

Antecedent moisture is considered low when there has been little preceding rainfall and high when there has been considerable preceding rainfall. For most design applications, watershed conditions are assumed to be ARC II. Routine use of ARC II is a recommendation of this manual.

According to Claudia Hoeft, National Hydraulic Engineer with the USDA-Natural Resources Conservation Service, curve number modification that changes initial abstractions from 0.2S to 0.05S will not impact the interpretation of ARC I, II, and III for different conditions[11].

[11] Information shared by Claudia Hoeft during email communication with Michael E. Meadows on 1 May 2020.

**Table 3.8**

**$I_a$=0.05S NRCS Curve Numbers for Average Runoff Conditions**

| Land Use | | Hydrologic Soil Group Classification | | | |
|---|---|---|---|---|---|
| | | A | B | C | D |
| Open Space | | | | | |
| Poor Condition (grass cover < 50%) | | 60 | 73 | 81 | 85 |
| Fair Condition (grass cover 50-75%) | | 40 | 61 | 73 | 79 |
| Good Condition (grass cover > 75%) | | 31 | 52 | 67 | 74 |
| Impervious Areas (paved parking lots, roofs, etc.) | | 97 | 97 | 97 | 97 |
| Streets and Roads | | | | | |
| Paved with curbs and storm sewers | | 97 | 97 | 97 | 97 |
| Paved with open ditches | | 77 | 85 | 89 | 90 |
| Gravel | | 69 | 80 | 85 | 88 |
| Dirt | | 64 | 76 | 82 | 85 |
| | Average % Impervious | | | | |
| Urban Land Use | | | | | |
| Commercial and Business | 85 | 85 | 89 | 92 | 93 |
| Industrial | 72 | 75 | 84 | 88 | 90 |
| Residential | | | | | |
| 1/8 Acre | 65 | 70 | 80 | 86 | 89 |
| 1/4 Acre | 38 | 52 | 68 | 77 | 82 |
| 1/3 Acre | 30 | 48 | 64 | 76 | 81 |
| 1/2 Acre | 25 | 45 | 62 | 74 | 80 |
| 1 Acre | 20 | 42 | 60 | 73 | 79 |
| 2 Acres | 12 | 37 | 57 | 70 | 76 |
| Developing urban areas, newly graded, no grass cover | | 70 | 81 | 88 | 92 |
| Pasture | | | | | |
| Poor | | 60 | 73 | 81 | 85 |
| Fair | | 40 | 61 | 73 | 79 |
| Good | | 31 | 52 | 67 | 74 |
| Woods | | | | | |
| Poor Condition | | 48 | 66 | 76 | 81 |
| Fair Condition | | 35 | 57 | 69 | 76 |
| Good Condition | | 25 | 49 | 57 | 71 |
| | Hydrologic Condition | | | | |
| Row Crop | | | | | |
| Straight Row | Poor | 64 | 75 | 84 | 88 |
| | Good | 59 | 71 | 80 | 85 |
| Contoured | Poor | 62 | 73 | 79 | 84 |
| | Good | 57 | 68 | 76 | 81 |
| Contoured and Terraced | Poor | 58 | 67 | 74 | 76 |
| | Good | 53 | 63 | 71 | 75 |

To convert CN for ARC II conditions to ARC I and ARC III conditions, first use equations 3.23, 3.24 and 3.25 to determine watershed retention for each condition. Then use Equation 3.26 to determine the condition appropriate CN value. Selected CN values are shown in Table 3.9.

$$S_{II} = \frac{1000}{CN} - 10 \tag{3.23}$$

$$S_I = 2.281 S_{II} \tag{3.24}$$

$$S_{III} = 0.427 S_{II} \tag{3.25}$$

$$CN = \frac{1000}{S+10} \tag{3.26}$$

**Table 3.9**
**Curve Numbers for**
**ARC I, ARC II and ARC III Conditions**

| Curve Number | | |
|---|---|---|
| **ARC I** | **ARC II** | **ARC III** |
| 4.6 | 10 | 20.6 |
| 9.9 | 20 | 36.9 |
| 15.8 | 30 | 50.1 |
| 22.6 | 40 | 61.0 |
| 30.5 | 50 | 70.1 |
| 39.7 | 60 | 77.8 |
| 50.6 | 70 | 84.5 |
| 63.7 | 80 | 90.4 |
| 79.8 | 90 | 95.5 |
| 100.0 | 100 | 100.0 |

## 3.5 Time of Concentration

### 3.5.1 Definition

Time of concentration (Tc) is the time required for runoff to travel from the hydraulically most distant point in the watershed to the outlet. The hydraulically most distant point is the point with the longest travel time to the watershed outlet, and not necessarily the point with the longest flow distance to the outlet.

Probably a better definition is that Tc is the time after the beginning of rainfall excess when all portions of the drainage basin are contributing simultaneously to flow at the outlet. Using an appropriate value for the time of concentration is very important, although it is sometimes hard to judge what is the correct value. Time of concentration will vary depending on the slope and characteristics of the watershed and flow path.

Watershed time of concentration is the sum of travel times through the various flow path segments: sheet flow, shallow concentrated flow, and channelized flow. Sheet flow is non-concentrated flow and generally is the first flow segment. Shallow concentrated flow occurs when the flow first begins to concentrate into small rills, gullies, and gutters, generally the second flow segment. Channel flow represents flow in areas such as storm sewers, perennial channels, and creeks, generally the last flow segment.

Not all flow paths will include the three segments. Mathematically, time of concentration is computed as

$$t_c = T_{ti} = \sum \frac{L_i}{v_i} \tag{3.27}$$

where $T_{ti}$ is travel time through path segment I, $L_i$ is the length of segment I, and $v_i$ is the velocity of flow along segment i. Segment characteristics and how one computes velocity and travel time are discussed in Sections 3.5.2 through 3.5.5.

### 3.5.2 Sheet Flow

Sheet flow is stormwater runoff flowing in a thin layer over the land surface. It is the flow that occurs overland in places where there are no rills or defined channels. Figure 3.9 is a photograph showing pathways for sheet flow runoff from a paved road and adjacent property into a gutter.

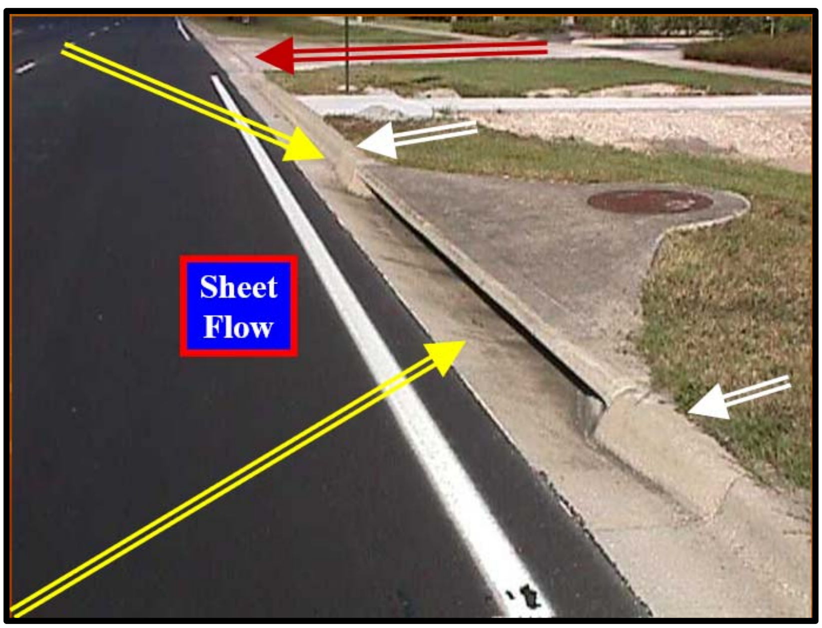



**Figure 3.9. Sheet Flow from Centerline of a Road and Adjacent Property**

Sheet flow occurs in the upper reaches of a watershed. Such flow occurs over short distances and at shallow depths prior to the point where topography and surface characteristics cause the flow to concentrate in rills and swales. The depth of such flow is usually 1 inch or less.

Sheet flow travel time is estimated with Manning's kinematic time of concentration equation.

$$T_t = \frac{0.42}{P_{2,24}^{0.5}} \left[ \frac{nL}{\sqrt{S_o}} \right]^{0.8} \tag{3.28}$$

where $T_t$ is travel time in minutes, $P_{2,24}$ is the 2-yr, 24-hour rainfall depth in inches, n is Manning's n-value, L is the overland length in feet, and $S_o$ is the average overland slope in feet/feet. Equation 3.28 is based on the original kinematic time of concentration for sheet flow (Overton and Meadows, 1976) that has rainfall excess intensity and not rainfall depth in the denominator. Recognizing the likelihood of inconsistent and/or incorrect determination of rainfall excess intensity by users, the NRCS decided to use the slope at the center of the Type II rainfall distribution curve to approximate intensity and a standard 2-year 24-hour rainfall depth instead of excess intensity. As such, every calculation in a common area uses the same rainfall data input to Equation 3.28.

Table 3.10 shows Manning's n-values for select cover conditions. The n-values for many cover conditions, along with values for shallow concentrated and channel flow, are given in Appendix B.

**Table 3.10**
**Select Manning's n-values for Overland Sheet Flow**

| **Surface** | **Manning n-value** |
|---|---|
| Smooth asphalt | 0.011 |
| Smooth concrete | 0.012 |
| Fallow (no residue) | 0.05 |
| Grass | |
| Short grass prairie | 0.15 |
| Dense grasses | 0.24 |
| Bermuda grass | 0.41 |
| Woods | |
| Light underbrush | 0..40 |
| Dense underbrush | 0.80 |
| Agriculture | |
| Cultivated soil with residue cover <=20% | 0.06 |
| Cultivated soil with residue cover >=20% | 0.17 |

### 3.5.3 Limitations to Sheet Flow Length

Based on his review of several technical papers on sheet flow, Merkel (2001) supported the sheet flow limit of 100 feet for Manning's kinematic solution. Kibler and Aron (1982) and others indicated the maximum sheet flow length is less than 100 feet. McCuen and Spiess (1995) indicated using one flow length as the limiting variable could lead to bad designs, and proposed the length limitation should instead be based on:

$$l = \frac{100\sqrt{S}}{n} \qquad (3.29)$$

where l is length, S is slope and n is Manning's n-value. This equation should be used for SCDOT applications.

Table 3.11 shows maximum sheet flow lengths calculated with Equation 3.29 for various cover types, n-values, and slope combinations. Equation 29 is included in the accompanying spreadsheet.

**Table 3.11**
**Example Maximum Sheet Flow Lengths**
**Using the McCuen-Spiess Limitation Criterion**

| Cover type | *n* values | Slope (ft/ft) | Length (ft) |
|---|---|---|---|
| Range | 0.13 | 0.01 | 77 |
| Grass | 0.41 | 0.01 | 24 |
| Woods | 0.80 | 0.01 | 12.5 |
| Range | 0.13 | 0.05 | 172 |
| lGrass | 0.41 | 0.05 | 55 |
| Woods | 0.80 | 0.05 | 28 |

### 3.5.4 Shallow Concentrated Flow

After reaching the limit, sheet flow begins to concentrate in rills, small gullies, and gutters. Shallow concentrated flow has depths of 0.1 to 0.5 feet and is assumed not to have a well-defined channel except for flow pathways like roadside gutters. Figure 3.10 shows surface runoff accumulating in a rill at a construction site where there is no well-defined channel. Figure 3.11 illustrates flow in a roadway gutter that is a well-defined channel. Sources for flow in the gutter in Figure 3.11 are illustrated in Figure 3.9 where there is sheet flow runoff from paved and grass covered surfaces. In areas of the sandhills with high infiltration sands without noticeable channels and in areas with rocky outcroppings or pavements that prevent the formation of channels can have flow lengths of 1000 ft. or greater.

Using the original methods in NEH Chapter 15 to estimate shallow concentrated flow travel time, velocities are developed using Figure 3.12, in which average velocity is a function of watercourse slope and type of channel. It is assumed that shallow concentrated flow can be represented by one of seven flow types. Curves for the different flow types were used to develop the information in Table 3.12 which includes a velocity constant for the different types of shallow concentrated flow. Equations for the curves in Figure 3.12 are of the general form

$$v = kS_o^{0.5} \tag{3.30}$$

where v is the average velocity in feet per second (fps), $S_o$ is the watercourse slope in feet per feet (ft/ft), and k is the velocity constant.


**Figure 3.10. Shallow Concentrated Runoff from a Construction Site**


**Figure 3.11. Shallow Concentrated Flow in a Roadway Gutter**

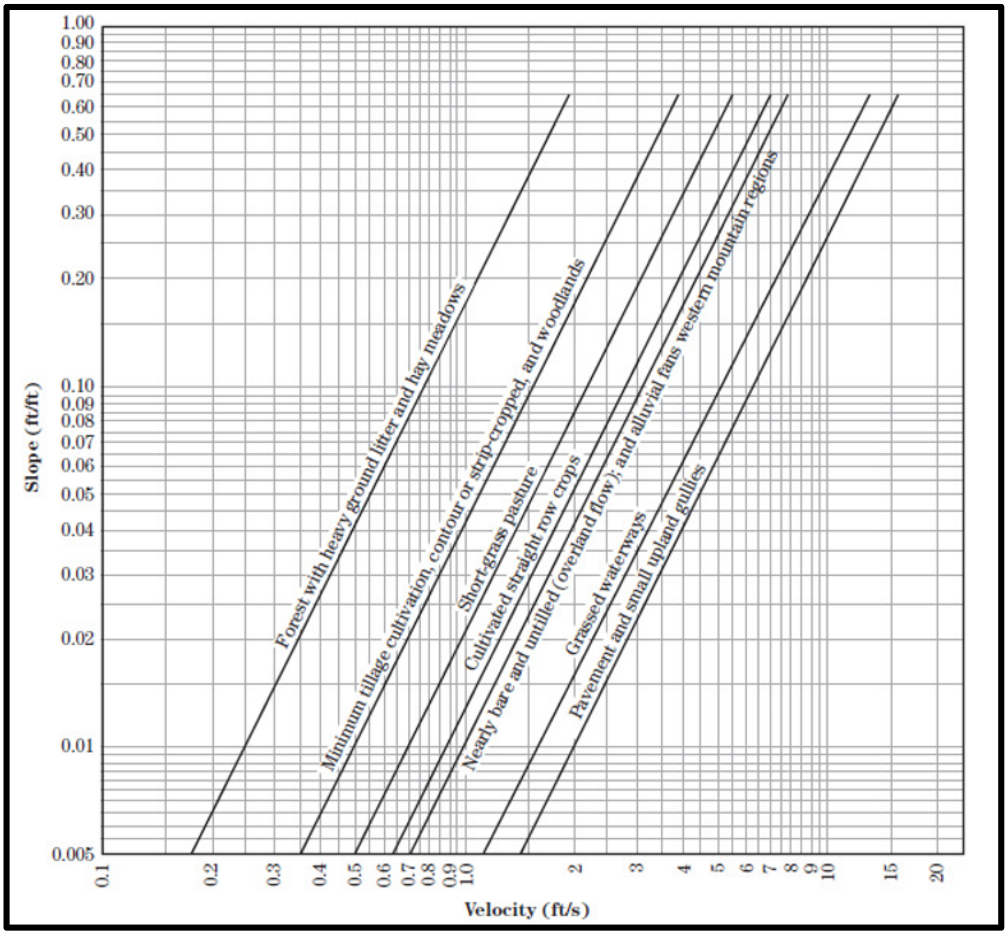



**Figure 3.12. Velocity Versus Slope for Shallow Concentrated Flow Types (NRCS, NEH Part 630, Chapter 15)**

**Table 3.12**
**NRCS Velocity Constants for Shallow Flow Types Shown in Figure 3.12**

| Shallow Flow Types | Depth (ft) | | Manning's n | Velocity Constant |
|---|---|---|---|---|
| Pavement and small upland gullies | 0.2 | | 0.025 | 20.328 |
| Grassed waterways | 0.4 | | 0.050 | 16.135 |
| Nearly bare and untilled (overland flow) | 0.2 | | 0.051 | 9.965 |
| Cultivated straight row crops | 0.2 | | 0.058 | 8.762 |
| Short-grass pasture | 0.2 | | 0.073 | 6.962 |
| Minimum tillage cultivation, contour or strip-cropped, and woodlands | 0.2 | | 0.101 | 5.032 |
| Forest with heavy ground litter and hay meadows | 0.2 | | 0.202 | 2.516 |

### 3.5.5 Channel Flow

Channel flow is flow in storm sewers, ditches/channels, creeks, etc. When estimating travel time for channel flow segments, estimate velocity for bank full or pipe full flow conditions using Manning's equation. Manning's n-values for different channel conditions can be obtained from standard texts or government agency manuals. Typical calculations are shown in Chapter 5 Example Applications Section 5.3. Runoff Hydrograph Simulation Using the SC UH Method.

### 3.5.6 Watershed Lag Time

The NRCS method for watershed lag was developed by Mockus in 1961. It spans a broad set of conditions ranging from heavily forested watersheds with steep channels and a high percent of runoff resulting from subsurface flow, to meadows providing a high retardance to surface runoff, to smooth land surfaces and large paved areas.

In the watershed lag time method, flow length is defined as the longest path along which water flows from the watershed divide to the outlet. During development of the regression equation, the longest flow path was used to represent the hydraulically most distant point in the watershed. Flow length can be measured using aerial photographs, topographic maps, or GIS techniques.

The NRCS Lag Time equation uses lumped parameter data (i.e., single values for watershed average slope), hydraulic length (typically measured along a characteristic path from the watershed boundary to the outlet), and watershed retention S computed with the curve number runoff model.

The equation is

$$Lag\ time\ (hrs),\ t_l = \frac{L^{0.8}(S+1)^{0.7}}{1900\,Y^{0.5}} \tag{3.31}$$

where L is watershed hydraulic length in feet, S =1000/CN – 10, and Y is average watershed slope in percent, not decimal format. If the slope is 2.4%, Y=2.4 and not 0.024.

The Lag Time equation is mostly applied to watersheds in the natural condition, which most often is a single land use or associated land uses, such as forest and pasture or grassland.

To get time of concentration, multiply $t_l$ by 1.67.

$$T_c = 1.67 t_l \tag{3.32}$$

### 3.6 South Carolina Synthetic Unit Hydrograph

### 3.6.1 Unit Hydrograph Theory

UH theory assumes a watershed responds linearly, meaning:

1. The unit hydrograph shape and time parameters do not change during a storm event.
2. The runoff hydrograph from any burst varies directly with the burst rainfall excess amount (runoff volume).
3. The runoff hydrograph from any burst occurs independent of concurrent runoff from any other burst.

The first property implies the same UH applies throughout a rainfall event. The second means individual burst runoff hydrographs can be determined by scaling the UH ordinates in proportion to the burst runoff volume, while maintaining the same time to peak and time base as the UH. The third property is the basic tenet of convolution (i.e., solutions are additive) meaning burst runoff hydrographs are summed in real time to obtain the event runoff hydrograph.

Convolution is the mathematical process whereby one scales the watershed unit hydrograph ordinates to obtain individual burst runoff hydrographs, lag successive burst hydrographs in real time by D-hours, and then add the burst runoff hydrographs in real time to obtain the event runoff hydrograph.

The scaling process involves adjusting the UH ordinates by the ratio of the burst rainfall excess amount (in watershed inches) to the unit hydrograph volume (1-inch).

$$Burst\ Hydrograph = UH \cdot \left(\frac{Burst\ excess\ rain\ in\ inches}{1\text{−}inch}\right) \tag{3.33}$$

This adjusts the UH volume (i.e., area under the UH) to the same volume as the burst rainfall excess amount. The burst runoff hydrograph will have the same time to peak and time base as the UH.

Each burst lags the preceding burst by D-hours. An obvious rule-of-thumb is that any burst (n) lags the start of the storm by (n-1) times D-hours. By setting the computational time interval equal to the burst duration, the lagging process follows this pattern. As such, the duration of the event runoff hydrograph equals the UH time base plus(n-1) times D-hours.

### 3.6.2 SC UH

The SC UH Method uses the two-parameter gamma distribution to describe the unit hydrograph. The gamma distribution UH is

$$Q = Q_p \left[\frac{t}{t_p} e^{1-\frac{t}{t_p}}\right]^{n-1} \tag{3.34}$$

where Q is flow rate in cfs, $Q_p$ is the unit hydrograph peak flow in cfs, $t_p$ is time to peak in hours, it is time in hours, A is watershed area in square miles, and n is the shape parameter that is a function of the PRF as shown in Table 3.13.

**Table 3.13**
**UH PRF and Shape Parameters**

| PRF | Γ-fn n |
|---|---|
| 50 | 1.05 |
| 100 | 1.25 |
| 156 | 1.50 |
| 237 | 2.00 |
| 298 | 2.50 |
| 349 | 3.00 |
| 393 | 3.50 |
| 433 | 4.00 |
| 470 | 4.50 |
| 484 | 4.70 |
| 504 | 5.00 |
| 566 | 6.00 |

The UH peak flow rate is computed as

$$Q_p = \frac{PRF \cdot A}{t_p} \tag{3.35}$$

For drainage area in square miles and time to peak in hours, this equation computes UH peak flow rate in units of cfs. The conversion factors are included in the PRF term**.**

The unit hydrograph time to peak is

$$t_p = t_l + D/2 \tag{3.36}$$

where $t_l$ is watershed lag time and D is rainfall burst duration. Historically, burst duration was estimated as two-tenths of lag time. Most computational tools use the updated rainfall distribution curves that have data at 6-minute (0.1 hours) intervals. Consequently, the default burst duration is 6 minutes. Example problems in this manual and the associated spreadsheet use 6-minute burst duration.

**A rule-of-thumb** is to always round $t_p$ to the nearest integer multiple of burst duration.

A key parameter in the UH peak flow rate equation is the PRF, which relates to UH shape and the distribution of runoff volume under the rising limb. PRF is defined as an index of watershed hydraulic efficiency. High PRF means more volume under the rising limb; low PRF means less volume under the rising limb. Stated differently, high PRF indicates flashy watershed response and low PRF indicates sluggish watershed response.

The influence of different PRF values is illustrated in Figure 3.13 that shows plots of normalized UH shape for four PRF values: 200, 300, 400, and 484. The UH with PRF equal to 484 has more volume under the rising limb and less under the recession limb than the UHs with PRF values of 200, 300, and 400. The UH with PRF equal to 200 has the least volume under the rising limb and the most volume under the recession limb. Watersheds and watershed conditions with different PRF values have different unit hydrographs.

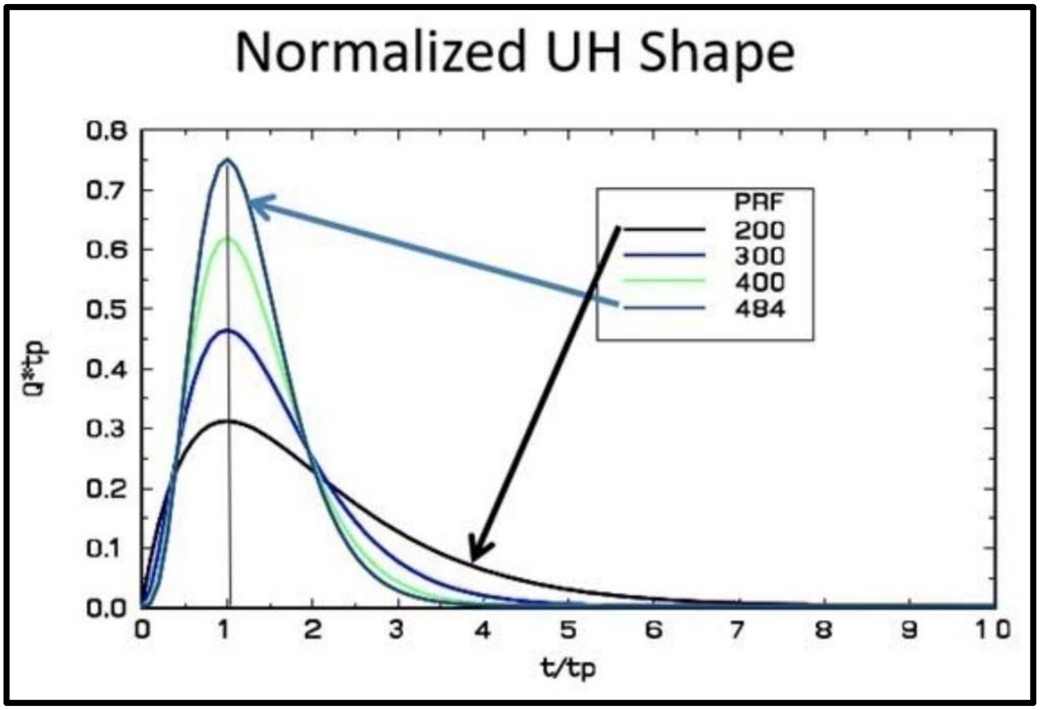



**Figure 3.13. Variation of UH Shape with Different PRF Values**

The two-parameter gamma distribution is now recommended by the NRCS as the way to describe UH shape for PRF values other than 484 as documented in the revised Chapter 16 Hydrographs of the National Engineering Handbook, Part 630 Hydrology, but no guidance is given about how to assign land use specific PRF values.

The SC UH resulted from the findings and results of multiple research and stormwater management studies. These studies verified the SC UH Method with an important outcome, i.e., PRF values vary with land use and, more importantly, each land use category has a unique PRF value.

### 3.7 Peak Rate Factors for South Carolina

Implementation of single PRF values for specific land uses (Table 3.14) followed recommendations by the NRCS to have a method that provided a single PRF value for each unique land use comparable with the NRCS curve number method. The values in Table 3.13 are an outcome from the results of stormwater management studies. These values were tested and confirmed during stormwater model verifications.

Because of changing land use conditions, pre- and post-development PRF values are different. For most applications, the post-development PRF is greater than the pre-development PRF. Development either increases the percentage imperviousness and/or the efficiency of the internal stormwater collection and conveyance system.

For watersheds with mixed land use conditions, use area weighting to determine average PRF.

The values in Table 3.14 probably will require updating if the recommended changes to the CN model are accepted by the NRCS. Initial tests by the author affirm simulated runoff hydrographs are different when using the current and revised CN models. Revisions probably can be determined using the results of multiple simulations for different watershed conditions.

**Table 3.14**
**Recommended PRF Values for South Carolina**

| Land Use | | Unit Hydrograph Peak Rate Factor (PRF) |
|---|---|---|
| Open Space | | 250 |
| Impervious Areas (paved parking lots, roofs, etc.) | | 550 |
| Streets and Roads | | |
| Paved with curbs and storm sewers | | 550 |
| Paved with open ditches | | 500 |
| Gravel | | 450 |
| Dirt | | 350 |
| Urban Land Use | % Imp | |
| Commercial and Business | 85 | 550 |
| Industrial | 72 | 550 |
| Residential | | |
| 1/8 Acre | 65 | 400 |
| 1/4 Acre | 38 | 375 |
| 1/3 Acre | 30 | 350 |
| 1/2 Acre | 25 | 350 |
| 1 Acre | 20 | 325 |
| 2 Acre | 12 | 300 |
| Developing urban areas, newly graded, no grass cover | | 400 |
| Pasture | | |
| Poor Condition | | 200 |
| Fair Condition | | 190 |
| Good Condition | | 180 |
| Woods | | |
| Poor Condition | | 200 |
| Fair Condition | | 190 |
| Good Condition | | 180 |
| Row Crop | | |
| Straight Row | | 300 |
| Contoured | | 275 |
| Contoured and Terraced | | 250 |

.

### 3.8. Critical Rainfall Duration

There are different interpretations of critical rainfall duration. One is the time for runoff from the watershed boundary to reach a specified downstream location. Another, which is used in this Manual, is the rainfall duration that produces the maximum peak flow or maximum runoff volume.

Tables 3.15 and 3.16 show D-hour rainfall event results for pre-land use change conditions at a Eutawville watershed using the NOAA B and NRCS Type II rainfall distributions. Note the star and arrow at the left side of each table. The star flags the duration with the maximum runoff volume which is 12 hours. The maximum runoff volume is the same in both tables since the values are based on the same rainfall depth and NRCS CN.

As illustrated in Table 3.15, for a given return period storm, one duration produces the maximum peak runoff and a different duration produces the maximum runoff volume. For some applications, the same duration may produce both maximum amounts. One vital point is that most critical durations are not 24-hours, particularly for peak flow prediction, which gives significant reason and justification to challenge regulations that prescribe a single design storm duration that is not a critical duration and could lead to a wrong and unsafe design.

A critical duration analysis should be performed to estimate peak discharges and runoff volumes to assure properly sized SCDOT flood control projects. An analysis can be performed to determine the critical duration storm for each watershed and each study reach across the watershed.

More significant results are the storm durations for peak runoff. The NOAA B peak occurs for the 6-hour event and the NRCS Type II peak occurs for the 3-hour event. The NOAA B peak is 120.5 cfs and the Type II peak is 125.5 cfs. The Type II resulted in a greater peak but not for the 24-hour storm for which the Type II peak is 86.3 cfs and the NOAA B 24-hour storm peak is 90.4 cfs. Bottomline, if design flows in South Carolina where the NOAA B curve applies are to be based on 24-hour storms, one should use the NOAA B curve and not the NRCS Type II. Peak flow comparisons should be performed at all locations.

Furthermore, if design flows are to be based on maximum peak flows for a specific return period the critical duration should be determined and the flow for that duration should be the design flow. As emphasized in this example, the maximum is not for the 24-hour event which underscores all designs should consider multiple storm durations and not just a 24-hour event.

**Table 3.15 Results for D-Hour Rainfall Events**
**Using NOAA B Rainfall Distribution**

| | Design Storm Return Period = | | | | 25 | Years | |
|---|---|---|---|---|---|---|---|
| | Design Rainfall Distribution = | | | | NOAA B | | |
| | Runoff Results for D-Hour Rainfall Events | | | | | | |
| | Storm Duration (hrs) | Rainfall Depth (in) | CN Adjusted for Rainfall Duration <24-hr | Runoff Volume $Q_{CN}$ (WS-in) | Peak Runoff, $Q_p$ (cfs) | Time of Peak Runoff (min) | |
| | 1 | 3.13 | 89.5 | 2.06 | 94.5 | 84 | |
| | 2 | 3.85 | 88.9 | 2.67 | 114.6 | 120 | |
| | 3 | 4.17 | 88.2 | 2.91 | 115.1 | 150 | |
| => | 6 | 4.94 | 86.2 | 3.43 | **120.5** | 240 | <= |
| * | 12 | 5.84 | 81.8 | **3.82** | 119.8 | 420 | |
| | 24 | 7.04 | 66.9 | 3.33 | **90.4** | 786 | |

**Table 3.16 Results for D-Hour Rainfall Events**
**Using NRCS Type II Rainfall Distribution**

| | Design Storm Return Period = | | | | 25 | Years | |
|---|---|---|---|---|---|---|---|
| | Design Rainfall Distribution = | | | | Type II | | |
| | Runoff Results for D-Hour Rainfall Events | | | | | | |
| | Storm Duration (hrs) | Rainfall Depth (in) | CN Adjusted for Rainfall Duration <24-hr | Runoff Volume $Q_{CN}$ (WS-in) | Peak Runoff, $Q_p$ (cfs) | Time of Peak Runoff (min) | |
| | 1 | 3.13 | 89.5 | 2.06 | 95.4 | 78 | |
| | 2 | 3.85 | 88.9 | 2.67 | 120.5 | 108 | |
| => | 3 | 4.17 | 88.2 | 2.91 | **125.5** | 138 | <= |
| | 6 | 4.94 | 86.2 | 3.43 | 121.8 | 234 | |
| * | 12 | 5.84 | 81.8 | **3.82** | 118.1 | 408 | |
| | 24 | 7.04 | 66.9 | 3.33 | **86.3** | 768 | |

# CHAPTER 4: CALIBRATION OF THE METHOD DURING STORMWATER MANAGEMENT STUDIES

## 4.1 Model Calibration and Verification

Model calibration is the fine-tuning of parameter values to achieve the best reproduction of observed events. When using the SC UH Method, one of the fundamental calibration parameters is the unit hydrograph PRF, which controls the magnitude of the hydrograph peak and shape.

Model verification is to compare predicted and observed stormwater hydrographs without adjusting parameter values. This represents a test of how good the model is and, if successful, instills confidence about using the model for design, forecasting, planning, and other decision-making efforts.

## 4.2 Discussion of Selected Calibration Studies and Parameter Modification

Stormwater management studies were conducted at multiple locations in South Carolina that verified the application of the SC UH Method, (i.e., different PRF values among watersheds). At each watershed, local rainfall and streamflow data were collected and used to calibrate and verify a stormwater model. The UH PRF and time to peak parameters were adjusted to obtain the closest match between simulated and measured hydrographs. Optimal PRF values for different land uses were evaluated statistically and, per recommendations by the NRCS, summarized as single values for unique land use categories. These values subsequently were tested and validated during model verification studies.

One notable study was Rawls Creek near Irmo, SC. The watershed was divided into 86 subwatersheds and included 130+ routing functions (i.e., road crossings, and streams). The land use included woods, pasture, single family residential (SFR), multifamily residential (MFR), and commercial. Three rain gages and seven streamflow gages were installed and monitored. Stormwater runoff was modeled with the Drain:Edge computer program. Model parameters were adjusted based on both internal and boundary validation tests.

In the period between Phases I and II, the study of USGS gaged watersheds was completed, resulting in equations to estimate PRF values. During Phase II, the PRF values were estimated with the equations with adjustments based on findings of the Aiken County Wise Hollow Creek study.

Two verification tests were performed. The first focused on Koon Branch, a tributary to Rawls Creek. Frequency flood peaks calculated using the equation predicted PRF values were compared to values obtained with the calibrated model. Results for the 2, 5, 10, 25, 50, and 100-year events at different road crossings showed good agreement that supported using the equation predicted PRF values. The second verification test compared flood peaks at several crossings along the main channel of Rawls Creek. The results also showed good agreement.

A second and very significant study that became a primary basis for the current PRF and Land Use Table was the Wise Hollow study in Aiken County, SC. While developing the conceptual

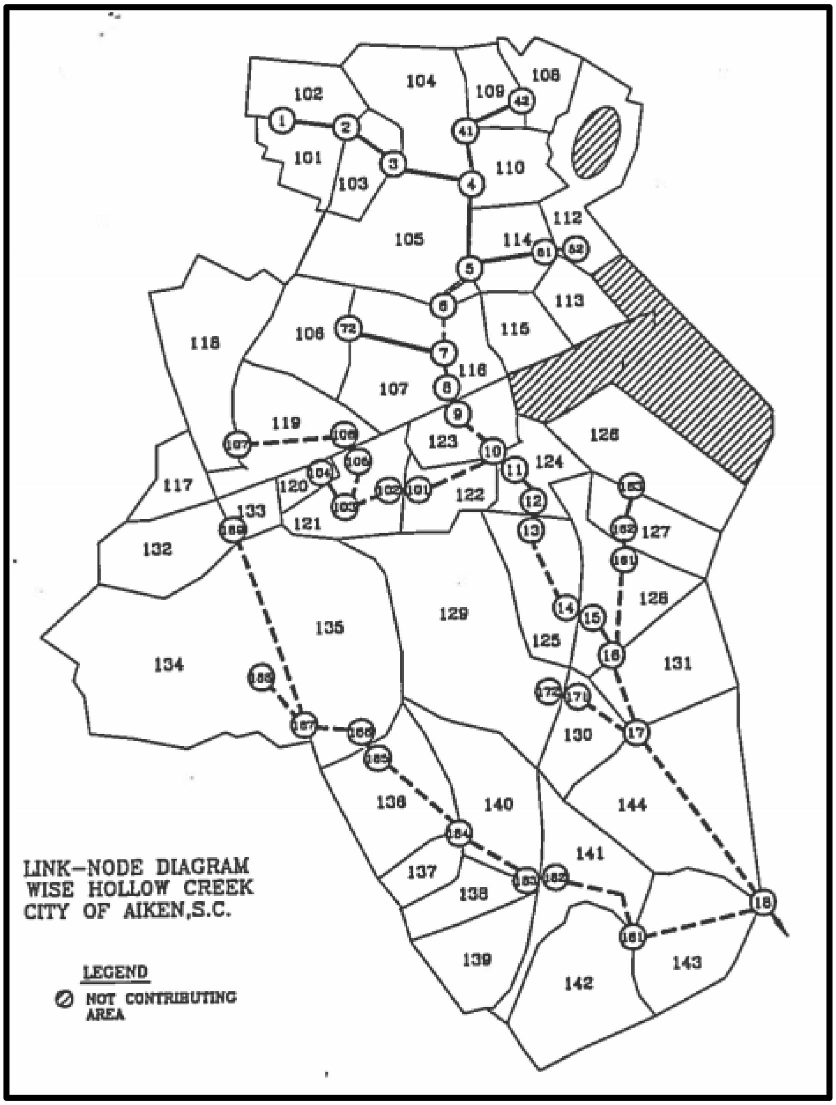



**Figure 4.1. Link-Node Diagram for Wise Hollow Creek Watershed**

model for the Wise Hollow Creek watershed, the study area was divided into 44 subwatersheds as shown in Figure 4.1. These subwatersheds were delineated with consideration for topographic features, land use, placement with respect to channel confluence, and the presence of hydraulic controls such as pond or roadway cross-drains that modify or attenuate the runoff hydrograph.

Figure 4.1 is a link-node diagram that represents the conceptual model for the watershed. A link-node diagram is a flowchart that shows the location of subwatershed outflows to the internal drainage network and the sequencing and connectivity of drainage features such as ponds, pipes, ditches, and road cross-drains. Nodes, indicated by numbered circles, represent the point of outflow from a subwatershed into the primary network, a point of confluence of two or more branches, and the upstream or downstream end of a pond, storm sewer, ditch, or road cross drain. Links connect nodes and represent ditches, ponds, storm sewers, or road cross drains.

The selection of PRF values was based on the results of stormwater management studies prior to the Wise Hollow Study during which researchers established ranges of PRF values for four general land uses: (1) commercial, (2) single family residential, (3) row crop, and (4) forest. These four groups were: (1) areas that are highly developed and paved with sewers; (2) areas with some development but with minor drainage system improvement; (3) areas with a high concentration of agricultural land; and (4) undeveloped and unimproved areas. PRF values (ranges of values) for these land uses are shown in Table 2.1 on Page 6 of this Manual and repeated below.

**Table 2.1**
**Results from South Carolina Stormwater Management Studies**
**Prior to Wise Hollow Study**

| Land Use | PRF |
|---|---|
| Commercial | 550 |
| Single Family Residential | 320-345 |
| Row Crop | 300-320 |
| Forest | 180 |

Based on data in Table 2.1, for commercial and industrial areas which usually have a high percent imperviousness and are well drained, PRF=550 was chosen. The value for forest was chosen as 180. Values for pasture and open space were determined using equations developed during the SC Unit Hydrograph study. The value for row crop was selected as the lower end of the range in Table 2.1. Values for single and multi-family residential were determined as weighted averages of open space and commercial land use based on percent imperviousness.

The final values of PRF values for specific land uses are shown in Table 2.2. Model calibration and verification results supported these values.

**Table 2.2**
**PRF Values for Specific Land Uses**

| Land Use | PRF |
|---|---|
| Urban | |
| Single Family Residential | 325 |
| Multi-family Residential | 375 |
| Commercial | 550 |
| Industrial | 550 |
| Open Spaces | 250 |
| Agricultural | |
| Forest | 180 |
| Pasture | 200 |
| Row Crop | 300 |

# CHAPTER 5: EXAMPLE APPLICATIONS

## 5.1. Rainfall Distributions for Storm Durations Less Than 24 Hours

Table 5.1 shows the development of the distribution for a 1-hour storm using the NOAA D rainfall distribution curve. The first two columns are the one-hour time and dimensionless rainfall ordinates from the 24-hour distribution. As explained in Section 3.2 of this Manual, the 1-hour interval is from 11.5 hours (690 minutes) to 12.5 hours (750 minutes). The dimensionless rainfall data are listed in the second column under the $P/P_{24\text{-hr}}$ heading. Using a burst duration of 6 minutes, the rescaled time ordinates are listed in the third column. At the start of the 1-hour rain, the dimensionless accumulated depth $P/P_{1\text{-hr}}$ is zero. After one hour, $P/P_{1\text{-hr}}$ is one. The values for the intermediate times are determined using the $P/P_{24\text{-hr}}$ data and linear interpolation with the following equation.

$$\frac{P(t)}{P_{1-hr}} = \frac{\frac{P(t)}{P_{24}} - 0.3170}{0.6830 - 0.3170} \tag{5.1}$$

Where 0.3170 and 0.6830 are $P/P_{24\text{-hr}}$ values at the beginning and end of the one-hour time interval that goes from 11.5 hours (690 minutes) to 12.5 hours (750 minutes).

**Table 5.1**
**Rescaling the 24-Hour NOAA D Rainfall Distribution Curve Ordinates to Form a 1-Hour Distribution**

| Maximum 1-Hour Portion of NOAA D Distribution Curve | | Rescaled Curve | |
|---|---|---|---|
| Time (min) | $P/P_{24\text{-hr}}$ | Time (min) | $P/P_{1\text{-hr}}$ |
| 690 | 0.3170 | 0 | 0.0000 |
| 696 | 0.3351 | 6 | 0.0495 |
| 702 | 0.3542 | 12 | 0.1015 |
| 708 | 0.3803 | 18 | 0.1729 |
| 714 | 0.4165 | 24 | 0.2720 |
| 720 | 0.4791 | 30 | 0.4429 |
| 726 | 0.5835 | 36 | 0.7280 |
| 732 | 0.6197 | 42 | 0.8271 |
| 738 | 0.6459 | 48 | 0.8985 |
| 744 | 0.6649 | 54 | 0.9505 |
| 750 | 0.6830 | 60 | 1.0000 |

Table 5.2 shows development of the distribution for a 2-hour storm using the NOAA B rainfall distribution curve. The first two columns are the two-hour time and dimensionless rainfall ordinates from the 24-hour distribution. The 2-hour interval is from 11 hours (660 minutes) to 13 hours (780 minutes). Use the following equation to determine the rescaled ordinates.

$$\frac{P(t)}{P_{2-hr}} = \frac{\frac{P(t)}{P_{24}} - 0.2156}{0.7844 - 0.2156} \tag{5.2}$$

**Table 5.2**
**Rescaling the 24-Hour NOAA D Rainfall Distribution Curve Ordinates to Form a 2-Hour Distribution**

| Maximum 2-Hour Portion of NOAA B Distribution Curve | | Rescaled Curve | |
|---|---|---|---|
| **Time (min)** | **$P/P_{24\text{-hr}}$** | **Time (min)** | **$P/P_{2\text{-hr}}$** |
| 660 | 0.2156 | 0 | 0.0000 |
| 666 | 0.2248 | 6 | 0.016 |
| 672 | 0.2352 | 12 | 0.034 |
| 478 | 0.2468 | 18 | 0.055 |
| 684 | 0.2596 | 24 | 0.077 |
| 690 | 0.2735 | 30 | 0.102 |
| 696 | 0.2955 | 36 | 0.140 |
| 702 | 0.3186 | 42 | 0.181 |
| 708 | 0.3504 | 48 | 0.237 |
| 714 | 0.3949 | 54 | 0.315 |
| 720 | 0.4729 | 60 | 0.452 |
| 726 | 0.6051 | 66 | 0.685 |
| 732 | 0.6496 | 72 | 0.763 |
| 738 | 0.6815 | 78 | 0.819 |
| 744 | 0.7046 | 84 | 0.860 |
| 750 | 0.7265 | 90 | 0.898 |
| 756 | 0.7404 | 96 | 0.923 |
| 762 | 0.7532 | 102 | 0.945 |
| 768 | 0.7649 | 108 | 0.966 |
| 774 | 0.7752 | 114 | 0.984 |
| 780 | 0.7844 | 120 | 1.0000 |

## 5.2. Runoff Weighted and Area Weighted NRCS Curve Number Values

Tables 5.3 and 5.4 show Runoff Weighted and Area Weighted CN values for the same watershed. As suggested in Section 3.4.4.4, Area Weighted CNs underpredict runoff amounts compared to Runoff Weighted CNs. For this watershed, the Area Weighted CN is 69 and the storm runoff is 0.67 inches. The Runoff Weighted CN is 70.67 and the runoff is 0.75 inches. The Area Weighted CN underpredicted the runoff volume by 10 percent compared to the Runoff Weighted CN. For other storms and watersheds, the percentage differences probably will be different from 10 percent, but greater than zero. For this example, given the watershed area of 100 acres, the Area Weighted CN underpredicted the Runoff Weighted CN runoff volume by more than 27,000 cubic feet which can have a significant impact on the analysis and design of stormwater storage and flow conveyance systems.

The data for this example is taken from Example 2 in Chapter 10 of the 16 October 2017 Updated Revision of the National Engineering Handbook.

**Table 5.3**
**Runoff Weighted Curve Number Calculator**

| Land Use | Condition | HSG | Standard CN | 24-hr $Q_{CN}$ | Area (ac) | Area*$Q_{CN}$ |
|---|---|---|---|---|---|---|
| Land Use 1 | Fair | B | 55 | 0.19 | 25.0 | 4.87 |
| Land Use 2 | Fair | C | 69 | 0.67 | 50.0 | 33.48 |
| Land Use 3 | Good | B | 83 | 1.45 | 25.0 | 36.16 |
| **Rainfall Depth (P) =** | **3.00** | | **Sum of Area*$Q_{CN}$ =** | | | 74.52 |
| **Runoff Depth ($Q_{CN}$) =** | **0.75** | | | | | |
| **Runoff Weighted CN =** | **70.67** | | | | | |
| **WS Retention S (in) =** | **4.15** | | | | | |
| **Initial Abstractions $I_a$=0.2S (in) =** | **0.83** | | | | | |

Insert P and $Q_{CN}$ into the following equation to compute CN:

$$CN = \frac{1000}{10+5P+10Q_{CN} - 10\left(Q_{CN}^2+1.25PQ_{CN}\right)^{0.5}} \tag{5.3}$$

**Table 5.4**
**Area Weighted Curve Number Calculator**

| Land Use | Condition | HSG | Standard CN | Area (ac) | Area*CN |
|---|---|---|---|---|---|
| Land Use 1 | Fair | B | 55 | 25.0 | 1,375 |
| Land Use 2 | Fair | C | 69 | 50.0 | 3,450 |
| Land Use 3 | Good | B | 83 | 25.0 | 2,075 |
| **Rainfall Depth (P) =** | **3.00** | | **Sum of Area*CN =** | | 6,900 |
| **Runoff Depth ($Q_{CN}$) =** | **0.67** | | | | |
| **Area Weighted WS CN =** | **69.00** | | | | |
| **WS Retention S (in) =** | **4.49** | | | | |
| **Initial Abstractions $I_a$=0.2S (in) =** | **0.90** | | | | |

## 5.3. Runoff Hydrograph Simulation Using the SC UH Method

Given: A 100-acre watershed near Eutawville, SC is scheduled to change from forest and pasture to a mix of forest, pasture, and urban land use. The pre-land use change conditions are 50% forest (good condition) and 50% row crop (straight row, good), hydraulic length is 0.50 miles, and average watershed slope is 1.6%. The watershed soils are HSG-B.

Post-land use change conditions will be 35% forest (good condition), 40% row crop (straight row, good), 15% SFR (30% impervious), 5% MFR, and 5% commercial. The sheet flow length will be 250 feet (pavement on 2.0% grade), shallow concentrated flow length will be 1,750 feet (pavement on 1.5% grade), and the channelized flow length will be 1,500 feet (30-inch RCP on 1% grade).

Find: Pre- and post-land use change runoff hydrographs for a 25-year 1-hour rainfall.

Solution: **Part I: Pre-land use change**

Use SC Synthetic Unit Hydrograph Method
Rainfall depth
Curve number runoff model
Watershed lag time
Unit hydrograph parameters
Rainfall and excess distribution curves
Convolution

*Step* 1: Determine design rainfall depth. Use an appropriate data source such as PFDS.

25-yr 1-hr P = 3.13 inches

*Step* 2: Determine the Runoff Weighted curve number to obtain the watershed retention factor. Runoff Weighted curve number is recommended as it is now supported by the NRCS. Use the Standard CN value (i.e. the 24-hour CN value). The next step will adjust it for the 1-hour storm.

| **Land Use** | **Condition** | **HSG** | **Standard CN** | **24-hr $Q_{CN}$** | **Area (ac)** | **Area*$Q_{CN}$** |
|---|---|---|---|---|---|---|
| Forest | Good | B | 55 | 0.23 | 50.0 | 11.53 |
| Row Crop | Good | B | 78 | 1.22 | 50.0 | 61.12 |
| **24-Hr Rainfall Depth (P) =** | **7.04** | | **Sum of Area*$Q_{CN}$ =** | | | 72.64 |
| **Runoff Depth ($Q_{CN}$) =** | **3.33** | | | | | |
| **Runoff Weighted CN =** | **66.92** | | | | | |
| **WS Retention S (in) =** | **4.94** | | | | | |
| **Initial Abstractions $I_a$=0.2S (in) =** | **0.99** | | | | | |

Runoff Weighted 24-hr CN = 66.92

Watershed retention: S = 4.94 inches

Initial abstractions: $I_a$= 0.2S = 0.99 inches

Adjust CN for rainfall duration of 1 hour: Since the CN is close to the limiting value of 65, use the McCuen method.

| | D-hr WS CN |
|---|---|
| | **24 Hour** |
| 24-hr CN = | 66.9 |
| S = | 4.94 |
| $I_a$ = | 0.99 |
| | **1 Hour** |
| **D =** | 1 |
| 1-hr P = | 3.13 |
| 1-hr Gamma = | 13.77 |
| 1-hr S = | 1.17 |
| 1-hr $I_a$ = | 0.23 |
| 1-hr $Q_{CN}$ = | 2.06 |
| **1-hr CN =** | **89.52** |

*Step 3*: Evaluate watershed lag time. You are given lumped parameter watershed data, so use the NRCS CN Lag Time equation.

Lag time: $t_l = \frac{L^{0.8}(S+1)^{0.7}}{1900Y^{0.5}}$= 0.79 hours = 47.5 minutes

L is the hydraulic length in feet, S is watershed retention computed in Step 2, and Y is the average watershed slope in percent. **As advised by NRCS, until further testing is complete use the S value computed with the standard NRCS 24-hr CN.**

*Step 4*: Evaluate unit hydrograph time and shape parameters. Use a burst duration of 6 minutes, i.e., D=6 min, which is consistent with NRCS methods.

Time to peak: $t_p = t_l + D/2$ = 50.5 min

Rule-of-thumb is to always round $t_p$ to the nearest integer multiple of burst duration.

Rounded $t_p$ = 48 min

Unit hydrograph PRF: Use values from Table 3.14 and the following worktable to evaluate the unit hydrograph PRF.

| Land Use | PRF | Acres | Product |
|---|---|---|---|
| Forest | 180 | 50 | 9,000 |
| Row Crop | 300 | 50 | 15,000 |
| **Sum of Acres and Products** | | **100** | **24,000** |

Area Weighted PRF = Σ Products /Σ Acres = 240

Interpolate on the following table to determine shape parameter, n, corresponding to the average PRF value. Interpolate linearly.

| **Shape Parameter (n)** | **Peak Rate Factor (PRF)** |
|---|---|
| 1.50 | 156 |
| 2.00 | 237 |
| 2.50 | 298 |
| 3.00 | 349 |
| 3.50 | 393 |
| 4.00 | 433 |
| 4.50 | 470 |
| 4.70 | 484 |
| 5.00 | 504 |
| 6.00 | 566 |

Shape parameter n = 2.02

Unit hydrograph peak: $Q_p = \dfrac{PRF \cdot A}{t_p} =$ 46.9 cfs

A is watershed area in square miles and $t_p$ is unit hydrograph time to peak in hours.

The gamma function UH is: $Q = Q_p \left[ \dfrac{t}{t_p} e^{1-\frac{t}{t_p}} \right]^{n-1}$

The unit hydrograph equation is: $Q = 46.9\left(\frac{t}{48}e^{1-\frac{t}{48}}\right)^{1.02}$ cfs

*Step* 5: Distribute rainfall in time using the maximum 1-hour portion of the appropriate NOAA dimensionless rainfall curve. Use the following table to list the NOAA distribution curve ordinates and to adjust them for a 1-hour rainfall. Use these data and the 1-hr NRCS CN to determine the rainfall excess accumulation curve. Subtract successive ordinates to determine the burst excess amounts, $\Delta Q_{CN}$, which is convoluted with the unit hydrograph to obtain the event runoff hydrograph. Label the NOAA Curve Ordinates column with NOAA A, B, C, or D.

| **Rainfall and Excess Accumulation Curves** | | | | | |
|---|---|---|---|---|---|
| Time, min | NOAA Curve (B) Ordinates | $P/P_1$ | P(t) | $Q_{CN}$ | $\Delta Q_{CN}$ |
| 0 | 0.2735 | 0.000 | 0.00 | 0.00 | 0.00 |
| 6 | 0.2955 | 0.048 | 0.15 | 0.00 | 0.00 |
| 12 | 0.3186 | 0.099 | 0.31 | 0.00 | 0.06 |
| 18 | 0.3504 | 0.170 | 0.53 | 0.06 | 0.15 |
| 24 | 0.3949 | 0.268 | 0.84 | 0.21 | 0.36 |
| 30 | 0.4729 | 0.440 | 1.38 | 0.56 | 0.75 |
| 36 | 0.6051 | 0.732 | 2.29 | 1.31 | 0.27 |
| 42 | 0.6496 | 0.830 | 2.60 | 1.58 | 0.20 |
| 48 | 0.6815 | 0.901 | 2.82 | 1.78 | 0.14 |
| 54 | 0.7046 | 0.952 | 2.98 | 1.92 | 0.14 |
| 60 | 0.7265 | 1.000 | 3.13 | 2.06 | |

Note: Burst 1 starts at time t=0 minutes and ends at t=6 minutes. Burst 2 starts at time t=6 minutes and ends at t=12 minutes.

*Step* 6: Convolute the rainfall excess with unit hydrograph to simulate the event runoff hydrograph.

| Time (min) | UH Ord (cfs) | Burst Hydrographs | | | | | | | | | | Summation Hydrograph (cfs) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Burst 1 $\Delta Q_{CN}=$ 0.00 | Burst 2 $\Delta Q_{CN}=$ 0.00 | Burst 3 $\Delta Q_{CN}=$ 0.06 | Burst 4 $\Delta Q_{CN}=$ 0.15 | Burst 5 $\Delta Q_{CN}=$ 0.36 | Burst 6 $\Delta Q_{CN}=$ 0.75 | Burst 7 $\Delta Q_{CN}=$ 0.27 | Burst 8 $\Delta Q_{CN}=$ 0.20 | Burst 9 $\Delta Q_{CN}=$ 0.14 | Burst 10 $\Delta Q_{CN}=$ 0.14 | |
| 0 | 0.00 | 0.00 | | | | | | | | | | 0.00 |
| 6 | 13.65 | 0.06 | 0.00 | | | | | | | | | 0.06 |
| 12 | 24.42 | 0.12 | 0.06 | 0.00 | | | | | | | | 0.18 |
| 18 | 32.55 | 0.15 | 0.12 | 0.75 | 0.00 | | | | | | | 1.03 |
| 24 | 38.46 | 0.18 | 0.15 | 1.35 | 1.99 | 0.00 | | | | | | 3.68 |
| 30 | 42.53 | 0.20 | 0.18 | 1.80 | 3.56 | 4.90 | 0.00 | | | | | 10.64 |
| 36 | 45.10 | 0.21 | 0.20 | 2.13 | 4.74 | 8.77 | 10.18 | 0.00 | | | | 26.23 |
| 42 | 46.47 | 0.22 | 0.21 | 2.35 | 5.60 | 11.69 | 18.22 | 3.69 | 0.00 | | | 41.99 |
| 48 | 46.88 | 0.22 | 0.22 | 2.49 | 6.20 | 13.81 | 24.28 | 6.61 | 2.69 | 0.00 | | 56.52 |
| 54 | 46.53 | 0.22 | 0.22 | 2.57 | 6.57 | 15.27 | 28.68 | 8.81 | 4.82 | 1.97 | 0.00 | 69.14 |
| 60 | 45.60 | 0.22 | 0.22 | 2.59 | 6.77 | 16.20 | 31.72 | 10.41 | 6.42 | 3.53 | 1.89 | 79.97 |
| 66 | 44.24 | 0.21 | 0.22 | 2.57 | 6.83 | 16.69 | 33.64 | 11.51 | 7.58 | 4.71 | 3.39 | 87.34 |
| 72 | 42.55 | 0.20 | 0.21 | 2.52 | 6.78 | 16.84 | 34.66 | 12.20 | 8.38 | 5.57 | 4.51 | 91.87 |
| 78 | 40.63 | 0.19 | 0.20 | 2.45 | 6.64 | 16.71 | 34.96 | 12.57 | 8.89 | 6.15 | 5.33 | 94.11 |
| 84 | 38.57 | 0.18 | 0.19 | 2.35 | 6.45 | 16.38 | 34.70 | 12.68 | 9.16 | 6.53 | 5.89 | 94.52 |
| 90 | 36.42 | 0.17 | 0.18 | 2.25 | 6.20 | 15.89 | 34.01 | 12.59 | 9.24 | 6.72 | 6.25 | 93.51 |
| 96 | 34.23 | 0.16 | 0.17 | 2.13 | 5.92 | 15.28 | 32.99 | 12.34 | 9.17 | 6.78 | 6.44 | 91.40 |
| 102 | 32.04 | 0.15 | 0.16 | 2.01 | 5.62 | 14.59 | 31.73 | 11.97 | 8.99 | 6.73 | 6.50 | 88.46 |
| 108 | 29.89 | 0.14 | 0.15 | 1.89 | 5.31 | 13.85 | 30.31 | 11.51 | 8.72 | 6.60 | 6.45 | 84.93 |
| 114 | 27.80 | 0.13 | 0.14 | 1.77 | 4.99 | 13.08 | 28.77 | 10.99 | 8.39 | 6.40 | 6.32 | 80.98 |
| 120 | 25.78 | 0.12 | 0.13 | 1.65 | 4.67 | 12.29 | 27.16 | 10.44 | 8.01 | 6.16 | 6.13 | 76.77 |
| 126 | 23.84 | 0.11 | 0.12 | 1.54 | 4.36 | 11.51 | 25.53 | 9.85 | 7.60 | 5.88 | 5.90 | 72.40 |
| 132 | 22.00 | 0.10 | 0.11 | 1.42 | 4.05 | 10.74 | 23.90 | 9.26 | 7.18 | 5.58 | 5.63 | 67.98 |
| 138 | 20.25 | 0.10 | 0.10 | 1.32 | 3.76 | 9.98 | 22.30 | 8.67 | 6.75 | 5.27 | 5.35 | 63.59 |
| 144 | 18.61 | 0.09 | 0.10 | 1.22 | 3.47 | 9.26 | 20.73 | 8.09 | 6.32 | 4.95 | 5.05 | 59.27 |
| 150 | 17.08 | 0.08 | 0.09 | 1.12 | 3.21 | 8.56 | 19.22 | 7.52 | 5.89 | 4.64 | 4.74 | 55.08 |
| 156 | 15.64 | 0.07 | 0.08 | 1.03 | 2.95 | 7.90 | 17.78 | 6.97 | 5.48 | 4.33 | 4.44 | 51.04 |
| 162 | 14.30 | 0.07 | 0.07 | 0.94 | 2.71 | 7.27 | 16.41 | 6.45 | 5.08 | 4.02 | 4.14 | 47.18 |
| 168 | 13.06 | 0.06 | 0.07 | 0.86 | 2.49 | 6.69 | 15.11 | 5.95 | 4.70 | 3.73 | 3.85 | 43.51 |
| 174 | 11.91 | 0.06 | 0.06 | 0.79 | 2.28 | 6.13 | 13.88 | 5.48 | 4.34 | 3.45 | 3.57 | 40.04 |
| 180 | 10.85 | 0.05 | 0.06 | 0.72 | 2.08 | 5.62 | 12.74 | 5.04 | 3.99 | 3.18 | 3.30 | 36.78 |
| 186 | 9.87 | 0.05 | 0.05 | 0.66 | 1.90 | 5.14 | 11.66 | 4.62 | 3.67 | 2.93 | 3.05 | 33.73 |
| 192 | 8.97 | 0.04 | 0.05 | 0.60 | 1.74 | 4.69 | 10.67 | 4.23 | 3.37 | 2.69 | 2.81 | 30.88 |
| 198 | 8.15 | 0.04 | 0.04 | 0.55 | 1.58 | 4.28 | 9.74 | 3.87 | 3.08 | 2.47 | 2.58 | 28.23 |
| 204 | 7.39 | 0.04 | 0.04 | 0.50 | 1.44 | 3.90 | 8.88 | 3.53 | 2.82 | 2.26 | 2.37 | 25.77 |
| 210 | 6.70 | 0.03 | 0.04 | 0.45 | 1.31 | 3.55 | 8.09 | 3.22 | 2.57 | 2.07 | 2.17 | 23.50 |
| 216 | 6.06 | 0.03 | 0.03 | 0.41 | 1.19 | 3.22 | 7.36 | 2.94 | 2.35 | 1.89 | 1.98 | 21.40 |
| 222 | 5.49 | 0.03 | 0.03 | 0.37 | 1.08 | 2.93 | 6.69 | 2.67 | 2.14 | 1.72 | 1.81 | 19.46 |
| 228 | 4.96 | 0.02 | 0.03 | 0.34 | 0.98 | 2.65 | 6.08 | 2.43 | 1.95 | 1.57 | 1.65 | 17.69 |
| 234 | 4.48 | 0.02 | 0.02 | 0.30 | 0.88 | 2.41 | 5.51 | 2.20 | 1.77 | 1.43 | 1.50 | 16.05 |
| 240 | 4.05 | 0.02 | 0.02 | 0.27 | 0.80 | 2.18 | 5.00 | 2.00 | 1.61 | 1.30 | 1.37 | 14.56 |
| 246 | 3.65 | 0.02 | 0.02 | 0.25 | 0.72 | 1.97 | 4.52 | 1.81 | 1.46 | 1.18 | 1.24 | 13.19 |
| 252 | 3.29 | 0.02 | 0.02 | 0.22 | 0.65 | 1.78 | 4.09 | 1.64 | 1.32 | 1.07 | 1.13 | 11.94 |
| 258 | 2.97 | 0.01 | 0.02 | 0.20 | 0.59 | 1.61 | 3.70 | 1.48 | 1.20 | 0.97 | 1.02 | 10.81 |

**Part II: Post-land use change**

*Step* 1: Determine design rainfall depth. Use an appropriate data source such as PFDS.

25-yr 1-hr P = 3.13 inches

*Step* 2: Determine the Runoff Weighted average watershed curve number to obtain the watershed retention factor. Use the Standard CN value, i.e. the 24-hour CN value. The next step will adjust it for the 1-hour storm.

| **Land Use** | **Condition** | **HSG** | **Standard CN** | **24-hr $Q_{CN}$** | **Area (ac)** | **Area*$Q_{CN}$** |
|---|---|---|---|---|---|---|
| Forest | Good | B | 55 | 2.15 | 35.0 | 75.23 |
| Row Crop | Good | B | 78 | 4.51 | 40.0 | 180.44 |
| SFR (30% Impervious) | Good | B | 64 | 3.03 | 15.0 | 45.48 |
| MFR | Good | B | 80 | 4.73 | 5.0 | 23.66 |
| Commercial | Good | B | 89 | 5.75 | 5.0 | 28.74 |
| **24-Hr Rainfall Depth (P) =** | **7.04** | | **Sum of Area*$Q_{CN}$ =** | | | 353.54 |
| **24-Hr Runoff Depth ($Q_{CN}$) =** | **3.54** | | | | | |
| **Runoff Weighted CN =** | **68.89** | | | | | |
| **WS Retention S (in) =** | **4.52** | | | | | |
| **Initial Abstractions $I_a$=0.2S (in) =** | **0.90** | | | | | |

Runoff Weighted 24-hr CN = 68.89

Watershed retention: S = 4.52 inches

Initial abstractions: $I_a$= 0.2S = 0.90 inches

Adjust CN for rainfall duration of 1 hour. Since the McCuen method was used for pre-land use change, use it for post-land use change.

| | D-hr WS CN |
|---|---|
| | **24 Hour** |
| 24-hr CN = | 68.9 |
| S = | 4.52 |
| $I_a$ = | 0.90 |
| | **1 Hour** |
| **D =** | 1 |
| 1-hr P = | 3.13 |
| 1-hr Gamma = | 13.38 |
| 1-hr S = | 1.13 |
| 1-hr $I_a$ = | 0.23 |
| 1-hr $Q_{CN}$ = | 2.09 |
| **1-hr CN =** | **89.82** |

*Step 3*: Evaluate watershed time of concentration. You are given data for three flow path segments: sheet flow (250 feet of pavement on 2.0% grade), shallow concentrated flow (1,750 feet pavement on 1.5% grade), and channelized flow (1,500 feet of 30-inch RCP storm sewer on 1% grade). The pavement is smooth asphalt.

Sheet flow travel time is estimated with Manning's kinematic time of concentration equation, Equation 3.28, that includes Manning's n-value, flow path length and slope, and the 2-year 24-hour rainfall depth.

Length = <u>250</u> ft Slope = 2% = <u>0.02</u> ft/ft Manning's n = <u>0.011</u>

Use the Precipitation Frequency Data Server to find the 2-year 24-hour rainfall depth for Eutawville, SC.

$P_{2,24}$ = <u>3.76</u> inches

Check the maximum allowable length with the McCuen-Spiess equation, Equation 3.29.

$$l = \frac{100\sqrt{S}}{n} \tag{3.29}$$

where l is length, S is slope and n is Manning's n-value.

The maximum allowable length is $l = \frac{100\sqrt{0.02}}{0.011}$ = 1,285 feet

This result confirms the length of 250 is allowable, so compute sheet flow travel time for a flow path length of 250 feet.

The sheet flow travel time (i.e. time of concentration) is

$$T_t = \frac{0.42}{P_{2,24}^{0.5}}\left(\frac{nL}{\sqrt{S_o}}\right)^{0.8} = \frac{0.42}{\sqrt{3.76}}\left[\frac{(0.011)(250)}{\sqrt{0.02}}\right]^{0.8} = 2.33\ minutes$$

Next, compute the shallow concentrated flow path travel time. The recommended NRCS method to estimate velocity for shallow concentrated flow uses the following empirical equations developed from experimental watershed data.

$$v = 16.13\ S_o^{0.5} \quad \text{unpaved} \tag{3.30a}$$

$$v = 20.33\ S_o^{0.5} \quad \text{paved} \tag{3.30b}$$

where v is the average velocity in feet per second (fps) and $S_o$ is the watercourse slope in feet per feet (ft/ft). For this application, use the equation for paved conditions to compute velocity. Determine travel time by dividing length by velocity. Remember to convert seconds to minutes.

$$T_t = \frac{L}{v} = \frac{L}{20.33\sqrt{S_o}} = \frac{1750}{20.33\sqrt{0.015}} = 703\ seconds = 11.72\ minutes$$

The last component of watershed time of concentration is channelized flow travel time, which for this application is travel time in the storm sewer. A common approach to estimate travel time in storm sewers is to divide the pipe length by the pipe-full flow velocity computed with this form of Manning's equation.

$$v = \frac{Q}{A} = \frac{1.49}{n} R^{2/3}\sqrt{S_o} \tag{5.4}$$

Where v is velocity in feet per second, Q is flowrate in units of cubic feet per second, A is cross-sectional area in square feet, n is Manning's n-value found in reference tables, R is hydraulic radius in units of feet computed by dividing area by wetted perimeter, and $S_o$ is the sewer slope in units of feet per feet.

For pipe-full flow

Area is: $A = \frac{\pi D^2}{4}$

Wetted perimeter is: $P = \pi D$

Hydraulic radius is: $R = \frac{A}{P} = \frac{D}{4}$

Manning's n: $n = 0.013$

Storm sewer travel time is:

$$T_t = \frac{L}{v} = \frac{L}{\frac{1.49}{n} R^{\frac{2}{3}} \sqrt{S_o}} = \frac{1500}{\frac{1.49}{0.013} \left(\frac{30}{(12)(4)}\right)^{\frac{2}{3}} \sqrt{0.01}} = 143.6\ seconds = 2.39\ minutes$$

Sum the flow path segment travel times to get the time of concentration:

Tc = 2.33 + 11.72 + 2.39 = 16.44 minutes

*Step 4*: Evaluate unit hydrograph time and shape parameters. Use a burst duration of 6 minutes, i.e., D=6 min, which is consistent with NRCS methods.

Time to peak: $t_p = t_l + D/2 =$ _14.1_ min

Rule-of-thumb is to always round $t_p$ to the nearest integer multiple of burst duration.

Rounded $t_p =$ _12_ min

Use values in Table 3.14 and the following worktable to evaluate the unit hydrograph PRF.

| Land Use | PRF | Acres | Product |
|---|---|---|---|
| Forest | 180 | 35.0 | 6,300 |
| Row Crop | 300 | 40.0 | 12,000 |
| SFR (30% Impervious) | 350 | 15.0 | 5,250 |
| MFR | 400 | 5.0 | 2,000 |
| Commercial | 550 | 5.0 | 2,750 |
| **Sum of Acres and Products** | | **100** | **28,300** |

Area Weighted PRF = Σ Products /Σ Acres = _283

Interpolate on the following table to determine shape parameter, n, corresponding to the average PRF value. Interpolate linearly.

| Shape Parameter (n) | Peak Rate Factor (PRF) |
|---|---|
| 1.50 | 156 |
| 2.00 | 237 |
| 2.50 | 298 |
| 3.00 | 349 |
| 3.50 | 393 |
| 4.00 | 433 |
| 4.50 | 470 |
| 4.70 | 484 |
| 5.00 | 504 |
| 6.00 | 566 |

Shape parameter n = 2.38

Unit hydrograph peak: $Q_p = \dfrac{PRF \cdot A}{t_p} = \underline{221.1}$ cfs

A is watershed area in square miles and $t_p$ is unit hydrograph time to peak in hours.

The gamma function UH is: $Q = Q_p \left[ \dfrac{t}{t_p} e^{1-\frac{t}{t_p}} \right]^{n-1}$

The unit hydrograph equation is: $Q = 221.1 \left( \frac{t}{12} e^{1-\frac{t}{12}} \right)^{1.38}$ cfs

*Step* 5: Distribute rainfall in time using the maximum 1-hour portion of the appropriate NOAA dimensionless rainfall curve.

| **Rainfall and Excess Accumulation Curves** | | | | | |
|---|---|---|---|---|---|
| **Time, min** | **NOAA Curve (B) Ordinates** | **$P/P_1$** | **P(t)** | **$Q_{CN}$** | **$\Delta Q_{CN}$** |
| 0 | 0.2735 | 0.000 | 0.00 | 0.00 | 0.000 |
| 6 | 0.2955 | 0.048 | 0.15 | 0.00 | 0.006 |
| 12 | 0.3186 | 0.099 | 0.31 | 0.01 | 0.059 |
| 18 | 0.3504 | 0.170 | 0.53 | 0.06 | 0.150 |
| 24 | 0.3949 | 0.268 | 0.84 | 0.21 | 0.365 |
| 30 | 0.4729 | 0.440 | 1.38 | 0.58 | 0.753 |
| 36 | 0.6051 | 0.732 | 2.29 | 1.33 | 0.272 |
| 42 | 0.6496 | 0.830 | 2.60 | 1.60 | 0.198 |
| 48 | 0.6815 | 0.901 | 2.82 | 1.80 | 0.145 |
| 54 | 0.7046 | 0.952 | 2.98 | 1.95 | 0.139 |
| 60 | 0.7265 | 1.000 | 3.13 | 2.09 | |

Note: Burst 1 starts at time t=0 minutes and ends at t=6 minutes. Burst 2 starts at time t=6 minutes and ends at t=12 minutes.

*Step* 6: Convolute the rainfall excess with unit hydrograph to simulate the event runoff hydrograph. For this example, convolution was terminated after 150 minutes.

| Time (min) | UH Ord (cfs) | Burst Hydrographs | | | | | | | | | | Summation Hydrograph (cfs) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Burst 1 $\Delta Q_{CN}=$ 0.000 | Burst 2 $\Delta Q_{CN}=$ 0.006 | Burst 3 $\Delta Q_{CN}=$ 0.059 | Burst 4 $\Delta Q_{CN}=$ 0.150 | Burst 5 $\Delta Q_{CN}=$ 0.365 | Burst 6 $\Delta Q_{CN}=$ 0.753 | Burst 7 $\Delta Q_{CN}=$ 0.272 | Burst 8 $\Delta Q_{CN}=$ 0.198 | Burst 9 $\Delta Q_{CN}=$ 0.145 | Burst 10 $\Delta Q_{CN}=$ 0.139 | |
| 0 | 0.00 | 0.00 | | | | | | | | | | 0.00 |
| 6 | 169.46 | 0.99 | 0.00 | | | | | | | | | 0.99 |
| 12 | 221.09 | 1.30 | 0.99 | 0.00 | | | | | | | | 2.29 |
| 18 | 194.11 | 1.14 | 1.30 | 9.92 | 0.00 | | | | | | | 12.35 |
| 24 | 144.90 | 0.85 | 1.14 | 12.94 | 25.43 | 0.00 | | | | | | 40.36 |
| 30 | 98.97 | 0.58 | 0.85 | 11.36 | 33.18 | 61.91 | 0.00 | | | | | 107.88 |
| 36 | 63.90 | 0.37 | 0.58 | 8.48 | 29.13 | 80.77 | 127.58 | 0.00 | | | | 246.91 |
| 42 | 39.69 | 0.23 | 0.37 | 5.79 | 21.74 | 70.91 | 166.45 | 46.14 | 0.00 | | | 311.65 |
| 48 | 23.96 | 0.14 | 0.23 | 3.74 | 14.85 | 52.94 | 146.13 | 60.20 | 33.59 | 0.00 | | 311.82 |
| 54 | 14.16 | 0.08 | 0.14 | 2.32 | 9.59 | 36.16 | 109.09 | 52.85 | 43.83 | 24.65 | 0.00 | 278.70 |
| 60 | 8.22 | 0.05 | 0.08 | 1.40 | 5.96 | 23.34 | 74.51 | 39.45 | 38.48 | 32.15 | 23.59 | 239.02 |
| 66 | 4.71 | 0.03 | 0.05 | 0.83 | 3.60 | 14.50 | 48.11 | 26.95 | 28.72 | 28.23 | 30.78 | 181.79 |
| 72 | 2.67 | 0.02 | 0.03 | 0.48 | 2.12 | 8.75 | 29.88 | 17.40 | 19.62 | 21.07 | 27.03 | 126.40 |
| 78 | 1.50 | 0.01 | 0.02 | 0.28 | 1.23 | 5.17 | 18.04 | 10.81 | 12.67 | 14.39 | 20.17 | 82.79 |
| 84 | 0.83 | 0.00 | 0.01 | 0.16 | 0.71 | 3.00 | 10.66 | 6.52 | 7.87 | 9.29 | 13.78 | 52.00 |
| 90 | 0.46 | 0.00 | 0.00 | 0.09 | 0.40 | 1.72 | 6.19 | 3.85 | 4.75 | 5.77 | 8.90 | 31.68 |
| 96 | 0.25 | 0.00 | 0.00 | 0.05 | 0.22 | 0.97 | 3.54 | 2.24 | 2.81 | 3.48 | 5.53 | 18.85 |
| 102 | 0.14 | 0.00 | 0.00 | 0.03 | 0.12 | 0.55 | 2.01 | 1.28 | 1.63 | 2.06 | 3.34 | 11.01 |
| 108 | 0.07 | 0.00 | 0.00 | 0.01 | 0.07 | 0.30 | 1.13 | 0.73 | 0.93 | 1.20 | 1.97 | 6.34 |
| 114 | 0.04 | 0.00 | 0.00 | 0.01 | 0.04 | 0.17 | 0.63 | 0.41 | 0.53 | 0.68 | 1.14 | 3.61 |
| 120 | 0.02 | 0.00 | 0.00 | 0.00 | 0.02 | 0.09 | 0.35 | 0.23 | 0.30 | 0.39 | 0.66 | 2.03 |
| 126 | 0.01 | 0.00 | 0.00 | 0.00 | 0.01 | 0.05 | 0.19 | 0.13 | 0.16 | 0.22 | 0.37 | 1.13 |
| 132 | 0.01 | 0.00 | 0.00 | 0.00 | 0.01 | 0.03 | 0.10 | 0.07 | 0.09 | 0.12 | 0.21 | 0.63 |
| 138 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.01 | 0.06 | 0.04 | 0.05 | 0.07 | 0.12 | 0.35 |
| 144 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.01 | 0.03 | 0.02 | 0.03 | 0.04 | 0.06 | 0.19 |
| 150 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.02 | 0.01 | 0.01 | 0.02 | 0.04 | 0.10 |

# CHAPTER 6: SPREADSHEET

## 6.1 Purpose of Spreadsheet

An Excel spreadsheet was developed to facilitate user application of the SC UH Method. The spreadsheet includes 88 worksheets, 62 of which are hidden and not visible to users. The hidden worksheets perform calculations the results of which are shown on active worksheets visible to users. The purpose and function of the active worksheets are listed in Table 6.1.

The first four worksheets listed in Table 6.1 are informational, self-explanatory, and are not discussed in this chapter. The other worksheets are for data entry, calculations, and showing results. The following sections discuss these worksheets.

**Table 6.1**
**Purpose and Function of Active Worksheets**

| Worksheet Label | Worksheet Purpose and Function |
|---|---|
| Title Sheet | Cites spreadsheet title and author |
| Introduction | Brief note to users about the spreadsheet and table of abbreviations |
| Watershed Information | SCDOT recommended form for site information |
| Spreadsheet Organization and Use | List and purpose of the active worksheets |
| Rainfall Data | User accesses on-line precipitation data and enters it into the spreadsheet. |
| SC Rainfall Distribution Map | Map of South Carolina showing regions where NOAA A, B, C, and D rainfall distributions apply |
| Data for CN Determination | User enters site land use and the NRCS curve number and area for each land use. |
| PRF Calculator | User enters the unit hydrograph Peak Rate Factor for each land use and this worksheet computes the watershed average PRF. |
| Tc Calculator | This worksheet has elements to compute watershed lag time with the lag time equation and time of concentration with the flow path travel time method. |
| WS & UH Data & Runoff Results | This worksheet reports summary watershed and unit hydrograph data, and the D-hour rainfall data, runoff depths, and peak runoff rates for storms with Annual Exceedance Probabilities of 10, 4, 2, and 1 percent. This worksheet includes drop down boxes for users to select CN, CN Modification Method, and Tc. One very important feature is this worksheet identifies the critical storm durations for runoff depth and peak flow for each AEP. |
| D-hr Runoff Ordinates | Tabular runoff hydrograph ordinates for 1, 2, 3, 6,12 and 24-hr storms with AEPs of 10%, 4%, 2%, and 1% |
| 10-yr D-hr Storm Hydrographs | Tabular and graphical runoff hydrographs for D-hr storms with 10-year return period (10% AEP) |
| 25-yr D-hr Storm Hydrographs | Tabular and graphical runoff hydrographs for D-hr storms with 25-year return period (4% AEP) |
| 50-yr D-hr Storm Hydrographs | Tabular and graphical runoff hydrographs for D-hr storms with 50-year return period (2% AEP) |
| 100-yr D-hr Storm Hydrographs | Tabular and graphical runoff hydrographs for D-hr storms with 100-year return period (1% AEP) |
| 24-hr Storm Hydrographs | Tabular and graphical runoff hydrographs for 24-hr storms with AEP values of 100%, 50%, 20%, 10%, 4%, 2%, and 1% |
| Pond Design Data and Results | User inputs the Design Storm AEP, data for calculation of pond stage-storage and stage-discharge ratings, and seepage rate through pond bottom. Pond performance results are shown for 1, 2, 3, 6, 12, and 24-hour design storms. |
| D-hr Storm Pond Routing Results | Tabular and graphical pond routing results for D-hour design storm events |
| 24-hr Storm Pond Routing Results | Tabular and graphical pond routing results for 24-hr storms with AEPS of 100%, 50%, 20%, 10%, 4%, 2%, and 1% |
| Pond Sediment Trap Efficiency | Computes sediment pond trapping efficiency computed with South Carolina Department of Health and Environmental Control (SCDHEC) Design Aid Curves and equations developed at UofSC. |
| USLE & MUSLE Results | Calculates annual erosion with the USLE and D-hour storm erosion with the MUSLE. |
| R factor | USLE Rainfall Factor: User selects value with Drop Down list of SC counties. |
| K factor | USLE Soil Erodibility Factor: User selects value with Drop Down list of SC soils. |
| LS factor | USLE Topography Factor: User enters erosion site length and slope and the value is calculated with an empirical equation. |
| C factor | USLE Cover Factor: User selects value by selecting cover conditions from Drop Down list. |
| P factor | USLE Conservation Practice Factor: User selects value by selecting conservation practice from Drop Down list. |

## 6.2 Discussion of Worksheets

### 6.2.1 Rainfall Data

Use the worksheet shown in Figure 6.1 to enter rainfall data for 1, 2, 3, 6, 12, and 24-hour storms with 1, 2, 4, and 10% Annual Exceedance Probabilities and for 24-hour storms with 1, 2, 4, 10, 20, 50, and 100% Annual Exceedance Probabilities. Follow the instructions in Section 3.1 about using the Precipitation Frequency Data Server to access NOAA rainfall data for a specific location. Basic steps include identifying the site location, which for this example is Eutawville, SC as shown in the unshaded cell in the first row of Figure 6.1. Next click on the cell showing https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=sc. This will access a map of South Carolina with a red crosshair you can move to the desired location or you can enter the location as shown in Figure 6.2. At the top of the map page, select the data type as precipitation depth which will provide the image shown in Figure 6.3. Note the rainfall data with AEP equal to 10% is outlined with a red border (done by the author of this Manual). These values were entered in the cells outlined with a red border in Figure 6.1. Enter the rainfall depths for the different durations and the 24-hour rainfalls for the different AEPs. Computed peak runoff rates are shown in the shaded cells beside the rainfall depths.

| Rain Gage Location | Eutawville |
|---|---|

| Click the cell below to access the Precipitation Frequency Data Server |
|---|
| https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=sc |

**Rainfall Data and Peak Runoff Rates for 10 Year D-Hour Storms and Seven 24-Hour Storms With AEP Ranging from 1 to 100 Percent**

| | D-Hour Rainfall Data | | | | 24-Hour Rainfall Data | | |
|---|---|---|---|---|---|---|---|
| | AEP (%) = | 10 | | | AEP (%) | Depth (in) | Peak Runoff, $Q_p$ (cfs) |
| | Return Period (yrs) = | 10 | Peak Runoff, $Q_p$ (cfs) | | | | |
| | Duration (hrs) | Depth (in) | | | 100 | 3.09 | 16.23 |
| Red Arrows flag rainfall durations with maximum peak runoff. | 1 | 2.66 | 85.14 | | 50 | 3.76 | 27.42 |
| | 2 | 3.22 | 100.30 | | 20 | 4.84 | 49.00 |
| | 3 | 3.45 | 99.03 | | 10 | 5.74 | 69.22 |
| | 6 | 4.07 | 102.01 | <= | 4 | 7.04 | 100.93 |
| | 12 | 4.78 | 97.07 | | 2 | 8.12 | 128.77 |
| | 24 | 5.74 | 69.22 | | 1 | 9.29 | 159.88 |

**25, 50 and 100 Year D-Hour Rainfall Data and Peak Runoff Rates**

| D-Hour Rainfall Data | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| AEP (%) = | 4 | | | 2 | | | 1 | | |
| Return Period (yrs) = | 25 | Peak Runoff, $Q_p$ (cfs) | | 50 | Peak Runoff, $Q_p$ (cfs) | | 100 | Peak Runoff, $Q_p$ (cfs) | |
| Duration (hrs) | Depth (in) | | | Depth (in) | | | Depth (in) | | |
| 1 | 3.13 | 107.17 | | 3.56 | 127.71 | | 3.98 | 148.02 | |
| 2 | 3.85 | 127.85 | | 4.43 | 153.65 | | 5.00 | 179.24 | |
| 3 | 4.17 | 128.33 | | 4.85 | 156.43 | | 5.54 | 185.21 | |
| 6 | 4.94 | 133.41 | <= | 5.77 | 163.78 | <= | 6.60 | 194.39 | |
| 12 | 5.84 | 130.45 | | 6.86 | 163.15 | | 7.90 | 196.82 | <= |
| 24 | 7.04 | 100.93 | | 8.12 | 128.77 | | 9.29 | 159.88 | |

**Figure 6.1 Rainfall Data Entry Worksheet**

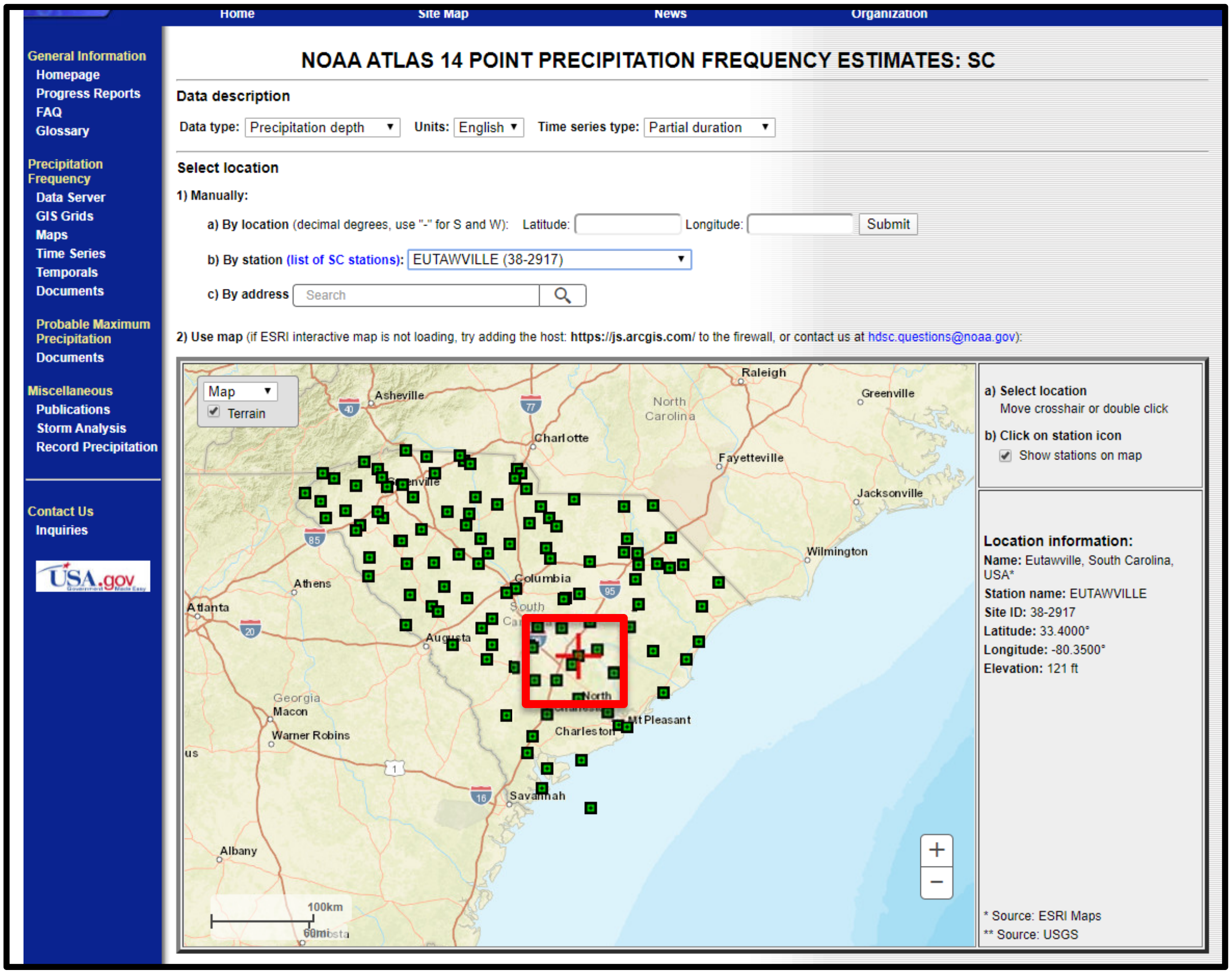



**Figure 6.2 Rain Gage Locations in South Carolina with Red Marker on Desired Location**

## POINT PRECIPITATION FREQUENCY (PF) ESTIMATES

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

PF tabular | PF graphical | Supplementary information | Print page

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

| Duration | Average recurrence interval (years) | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 5 | 10 | 25 | 50 | 100 | 200 | 500 | 1000 |
| 5-min | 0.468 (0.435-0.504) | 0.538 (0.500-0.579) | 0.610 (0.566-0.656) | 0.697 (0.645-0.748) | 0.785 (0.724-0.843) | 0.866 (0.795-0.929) | 0.940 (0.858-1.01) | 1.01 (0.917-1.09) | 1.10 (0.988-1.19) | 1.19 (1.06-1.28) |
| 10-min | 0.747 (0.694-0.804) | 0.860 (0.799-0.925) | 0.977 (0.907-1.05) | 1.11 (1.03-1.20) | 1.25 (1.15-1.34) | 1.38 (1.27-1.48) | 1.49 (1.36-1.60) | 1.60 (1.45-1.72) | 1.74 (1.56-1.87) | 1.87 (1.66-2.02) |
| 15-min | 0.934 (0.868-1.01) | 1.08 (1.00-1.16) | 1.24 (1.15-1.33) | 1.41 (1.31-1.51) | 1.59 (1.46-1.70) | 1.75 (1.60-1.87) | 1.89 (1.72-2.02) | 2.02 (1.83-2.17) | 2.19 (1.97-2.36) | 2.35 (2.09-2.53) |
| 30-min | 1.28 (1.19-1.38) | 1.49 (1.39-1.61) | 1.76 (1.63-1.89) | 2.04 [illegible] | 2.35 (2.17-2.52) | 2.63 (2.42-2.82) | 2.89 (2.64-3.10) | 3.15 (2.85-3.38) | 3.49 (3.13-3.75) | 3.80 (3.38-4.10) |
| 60-min | 1.60 (1.48-1.72) | 1.87 (1.74-2.02) | 2.25 (2.09-2.42) | 2.66 (2.46-2.86) | 3.13 (2.89-3.36) | 3.56 (3.27-3.82) | 3.98 (3.64-4.27) | 4.42 (4.00-4.75) | 5.00 (4.49-5.39) | 5.55 (4.93-5.99) |
| 2-hr | 1.85 (1.71-2.00) | 2.19 (2.03-2.37) | 2.68 (2.48-2.89) | 3.22 (2.97-3.46) | 3.85 (3.54-4.14) | 4.43 (4.05-4.76) | 5.00 (4.55-5.38) | 5.61 (5.06-6.03) | 6.39 (5.71-6.89) | 7.14 (6.30-7.72) |
| 3-hr | 1.96 (1.81-2.14) | 2.33 (2.15-2.54) | 2.86 (2.64-3.11) | 3.45 (3.18-3.75) | 4.17 (3.82-4.53) | 4.86 (4.41-5.26) | 5.55 (5.00-6.00) | 6.29 (5.61-6.80) | 7.29 (6.43-7.90) | 8.25 (7.19-8.97) |
| 6-hr | 2.31 (2.13-2.52) | 2.74 (2.54-2.98) | 3.37 (3.10-3.66) | 4.07 (3.74-4.42) | 4.94 (4.51-5.35) | 5.77 (5.24-6.25) | 6.61 (5.95-7.15) | 7.51 (6.70-8.13) | 8.75 (7.70-9.49) | 9.95 (8.64-10.8) |
| 12-hr | 2.69 (2.47-2.95) | 3.19 (2.94-3.50) | 3.93 (3.62-4.30) | 4.78 (4.38-5.22) | 5.84 (5.32-6.35) | 6.86 (6.20-7.45) | 7.91 (7.07-8.57) | 9.05 (8.01-9.81) | 10.6 (9.26-11.5) | 12.2 (10.4-13.2) |
| 24-hr | 3.09 (2.86-3.37) | 3.76 (3.47-4.09) | 4.84 (4.46-5.26) | 5.74 (5.28-6.24) | 7.04 (6.44-7.63) | 8.13 (7.39-8.81) | 9.30 (8.40-10.1) | 10.6 (9.47-11.5) | 12.4 (11.0-13.5) | 13.9 (12.2-15.2) |
| 2-day | 3.62 (3.33-3.97) | 4.38 (4.03-4.80) | 5.60 (5.13-6.13) | 6.61 (6.04-7.23) | 8.07 (7.32-8.82) | 9.28 (8.38-10.2) | 10.6 (9.49-11.6) | 12.0 (10.7-13.2) | 14.0 (12.3-15.5) | 15.7 (13.7-17.4) |
| 3-day | 3.90 (3.59-4.25) | 4.71 (4.34-5.13) | 5.97 (5.49-6.51) | 7.01 (6.42-7.64) | 8.50 (7.75-9.26) | 9.73 (8.83-10.6) | 11.0 (9.95-12.1) | 12.5 (11.1-13.6) | 14.5 (12.8-15.9) | 16.2 (14.2-17.8) |
| 4-day | 4.17 (3.85-4.54) | 5.03 (4.64-5.47) | 6.34 (5.84-6.89) | 7.41 (6.81-8.05) | 8.93 (8.17-9.69) | 10.2 (9.27-11.1) | 11.5 (10.4-12.5) | 12.9 (11.6-14.1) | 15.0 (13.3-16.4) | 16.6 (14.7-18.3) |
| 7-day | 4.86 (4.51-5.26) | 5.84 (5.42-6.32) | 7.28 (6.75-7.87) | 8.43 (7.80-9.11) | 10.0 (9.23-10.8) | 11.3 (10.4-12.3) | 12.7 (11.6-13.7) | 14.1 (12.8-15.3) | 16.1 (14.5-17.6) | 17.8 (15.9-19.5) |
| 10-day | 5.57 (5.18-5.98) | 6.66 (6.21-7.16) | 8.16 (7.60-8.78) | 9.34 (8.68-10.0) | 10.9 (10.1-11.8) | 12.2 (11.3-13.2) | 13.5 (12.4-14.6) | 14.9 (13.6-16.1) | 16.8 (15.2-18.2) | 18.3 (16.6-19.9) |
| 20-day | 7.45 (6.96-7.98) | 8.87 (8.29-9.50) | 10.7 (9.98-11.5) | 12.1 (11.3-13.0) | 14.1 (13.1-15.1) | 15.6 (14.5-16.7) | 17.2 (15.8-18.4) | 18.8 (17.3-20.2) | 21.0 (19.2-22.6) | 22.7 (20.6-24.5) |
| 30-day | 9.19 (8.64-9.77) | 10.9 (10.2-11.6) | 12.9 (12.1-13.7) | 14.4 (13.5-15.3) | 16.4 (15.4-17.5) | 18.0 (16.8-19.1) | 19.5 (18.2-20.8) | 21.1 (19.6-22.5) | 23.2 (21.4-24.8) | 24.8 (22.7-26.6) |
| 45-day | 11.6 (11.0-12.3) | 13.7 (12.9-14.5) | 16.0 (15.1-16.9) | 17.7 (16.7-18.8) | 20.0 (18.8-21.2) | 21.7 (20.3-23.1) | 23.4 (21.9-24.9) | 25.1 (23.4-26.7) | 27.3 (25.3-29.1) | 28.9 (26.8-31.0) |
| 60-day | 13.8 (13.0-14.6) | 16.2 (15.3-17.2) | 18.8 (17.7-19.9) | 20.7 (19.5-21.9) | 23.2 (21.8-24.5) | 25.0 (23.5-26.5) | 26.8 (25.2-28.5) | 28.6 (26.8-30.4) | 30.9 (28.9-32.9) | 32.6 (30.4-34.8) |

[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.

**Figure 6.3 Precipitation Frequency Rainfall Table at Selected Location**

Users also select the Rainfall Distribution Curve on the Rainfall Data Entry Worksheet. There is a map of South Carolina that shows the recommended distribution curve for different regions. The user selects the appropriate distribution with a drop-down box as seen in Figure 6.4. A larger scale map is shown on the following worksheet labelled SC Rainfall Distribution Map.

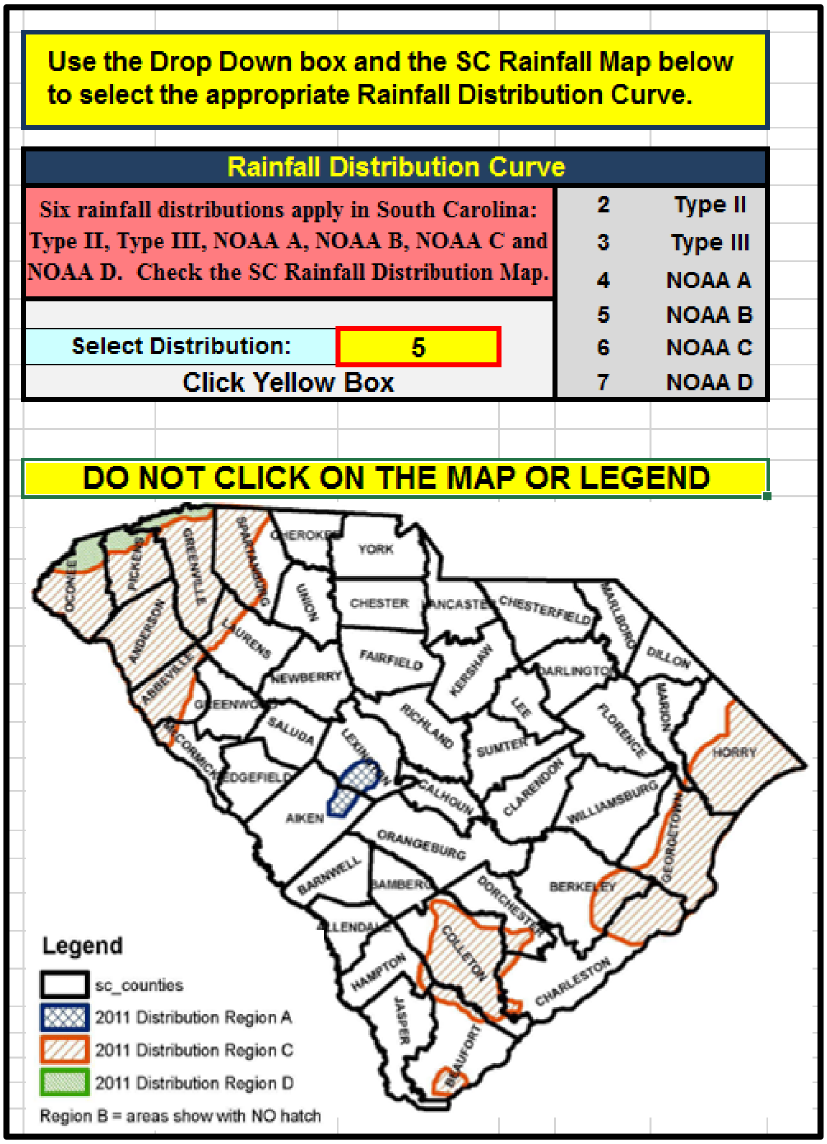
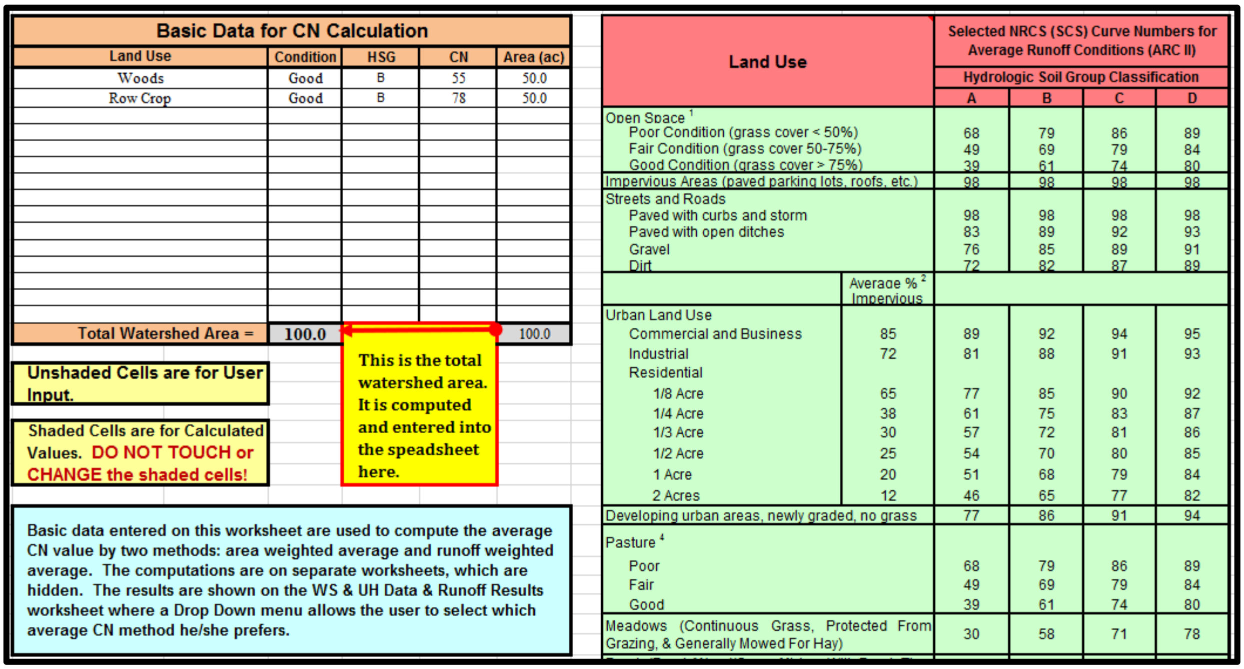



**Figure 6.4 Map of South Carolina Showing Regions for Rainfall Distribution Curves A, B, C, and D**

### 6.2.2 Data for CN Determination

Data entered on this worksheet, shown in Figure 6.5, are used to compute the average CN value with one of two methods: Area Weighted average and Runoff Weighted average. The user enters watershed data for land use, hydrologic condition, hydrologic soil group (HSG), curve number (CN), and area in acres. Values are entered for each land use category. The average CN computations are done on separate worksheets, which are hidden. Historically, average curve numbers were computed as Area Weighted which is still done in many locations. The NRCS now recommends using CN determined with the Runoff Weighted method.



Basic Data for CN Calculation

| Land Use | Condition | HSG | CN | Area (ac) |
|---|---|---|---|---|
| Woods | Good | B | 55 | 50.0 |
| Row Crop | Good | B | 78 | 50.0 |
| Total Watershed Area = | 100.0 | | | 100.0 |

Unshaded Cells are for User Input.

Shaded Cells are for Calculated Values. DO NOT TOUCH or CHANGE the shaded cells!

This is the total watershed area. It is computed and entered into the speadsheet here.

Basic data entered on this worksheet are used to compute the average CN value by two methods: area weighted average and runoff weighted average. The computations are on separate worksheets, which are hidden. The results are shown on the WS & UH Data & Runoff Results worksheet where a Drop Down menu allows the user to select which average CN method he/she prefers.

| Land Use | Average % [2] Impervious | Selected NRCS (SCS) Curve Numbers for Average Runoff Conditions (ARC II) — Hydrologic Soil Group Classification: A | B | C | D |
|---|---|---|---|---|---|
| Open Space [1] | | | | | |
| Poor Condition (grass cover < 50%) | | 68 | 79 | 86 | 89 |
| Fair Condition (grass cover 50-75%) | | 49 | 69 | 79 | 84 |
| Good Condition (grass cover > 75%) | | 39 | 61 | 74 | 80 |
| Impervious Areas (paved parking lots, roofs, etc.) | | 98 | 98 | 98 | 98 |
| Streets and Roads | | | | | |
| Paved with curbs and storm | | 98 | 98 | 98 | 98 |
| Paved with open ditches | | 83 | 89 | 92 | 93 |
| Gravel | | 76 | 85 | 89 | 91 |
| Dirt | | 72 | 82 | 87 | 89 |
| Urban Land Use | | | | | |
| Commercial and Business | 85 | 89 | 92 | 94 | 95 |
| Industrial | 72 | 81 | 88 | 91 | 93 |
| Residential | | | | | |
| 1/8 Acre | 65 | 77 | 85 | 90 | 92 |
| 1/4 Acre | 38 | 61 | 75 | 83 | 87 |
| 1/3 Acre | 30 | 57 | 72 | 81 | 86 |
| 1/2 Acre | 25 | 54 | 70 | 80 | 85 |
| 1 Acre | 20 | 51 | 68 | 79 | 84 |
| 2 Acres | 12 | 46 | 65 | 77 | 82 |
| Developing urban areas, newly graded, no grass | | 77 | 86 | 91 | 94 |
| Pasture [4] | | | | | |
| Poor | | 68 | 79 | 86 | 89 |
| Fair | | 49 | 69 | 79 | 84 |
| Good | | 39 | 61 | 74 | 80 |
| Meadows (Continuous Grass, Protected From Grazing, & Generally Mowed For Hay) | | 30 | 58 | 71 | 78 |

**Figure 6.5 Worksheet for Watershed Average Curve Number Calculation**

Results are shown on the WS & UH Data & Runoff Results worksheet (Figure 6.6) where a Drop-Down menu (Figure 6.7) allows the user to select the preferred method to determine the average CN.

There are two other Drop-Down menus on this worksheet as seen in Figure 6.7. One allows users to select either the McCuen or Merkel method to adjust CN for rainfall durations less than 24 hours. The other Drop-Down menu allows users to choose between the Travel Time and Lag Time methods to compute time of concentration.

| Summary Watershed and Unit Hydrograph Data | | | |
|---|---|---|---|
| **Watershed Data** | | **Unit Hydrograph Data** | |
| Area (acres) = | 100.0 | Burst Dur (min) = | 6 |
| Tc (minutes) = | 58.7 | Γ-fn n = | 2.02 |
| PRF = | 240 | Lag time (min) = | 35.2 |
| Standard CN = | 67.3 | Adj Tc (min) = | 60.0 |
| WS Retention S = | 4.86 | UH_Tp = | 42.0 |
| Initial Abstraction Ia = | 0.97 | UH_Qp = | 53.6 |

| Drop Down boxes to select CN, CN Modification Method, and Tc | | | |
|---|---|---|---|
| Select Standard CN Value | 2 | 1 | Area Weighted CN |
| Click Yellow Box | | 2 | Runoff Weighted CN |
| Select CN Modification Method | 1 | 1 | McCuen |
| Click Yellow Box | | 2 | Merkel |
| Select Time of Concentration | 1 | 1 | Travel Time Method |
| Click Yellow Box | | 2 | Lag Time Equation |

**For the Following Tables**

| Design Rainfall Distribution = | NOAA B | Curve Number Adjustment Method = | McCuen |
|---|---|---|---|

In each table, the D-hr design storm duration with the maximum runoff volume is flagged with a Blue Arrow in the left side column and the duration with maximum peak runoff is flagged with a Red Arrow in the right side column. These are *Critical Durations*.

**D-Hour Rainfall Data, Runoff Depths, and Peak Runoff Rates**

The following cells report runoff data for storms with Annual Exceedance Probabilities of 10, 4, 2 and 1 percent.

| Annual Exceedance Probablity (AEP) = | 10 | % | Annual Exceedance Probablity (AEP) = | 4 | % |
|---|---|---|---|---|---|
| Design Storm Return Period = | 10 | Years | Design Storm Return Period = | 25 | Years |

**Runoff Results for 10 Year D-Hour Rainfall Events**

| | Storm Duration (hrs) | Rainfall Depth (in) | CN Adjusted for Rainfall Duration <24-hr | Runoff Volume $Q_{CN}$ (WS-in) | Peak Runoff $Q_p$ (cfs) | Time of Peak Runoff (min) | |
|---|---|---|---|---|---|---|---|
| | 1 | 2.66 | 89.6 | 1.64 | 85.26 | 78 | |
| | 2 | 3.22 | 88.9 | 2.09 | 99.83 | 120 | |
| | 3 | 3.45 | 88.3 | 2.25 | 98.58 | 144 | |
| | 6 | 4.07 | 86.3 | 2.64 | 102.17 | 234 | <= |
| => | 12 | 4.78 | 82.0 | 2.89 | 99.29 | 414 | |
| | 24 | 5.74 | 67.3 | 2.36 | 69.22 | 780 | |

**Runoff Results for 25 Year D-Hour Rainfall Events**

| | Storm Duration (hrs) | Rainfall Depth (in) | CN Adjusted for Rainfall Duration <24-hr | Runoff Volume $Q_{CN}$ (WS-in) | Peak Runoff $Q_p$ (cfs) | Time of Peak Runoff (min) | |
|---|---|---|---|---|---|---|---|
| | 1 | 3.13 | 89.6 | 2.07 | 107.29 | 78 | |
| | 2 | 3.85 | 88.9 | 2.68 | 127.35 | 114 | |
| | 3 | 4.17 | 88.3 | 2.91 | 127.87 | 144 | |
| | 6 | 4.94 | 86.3 | 3.44 | 133.57 | 234 | <= |
| => | 12 | 5.84 | 82.0 | 3.84 | 132.68 | 414 | |
| | 24 | 7.04 | 67.3 | 3.37 | 100.93 | 774 | |

**Figure 6.6 Worksheet Showing WS & UH Data & Runoff Results**

| Drop Down boxes to select CN, CN Modification Method, and Tc | | | |
|---|---|---|---|
| Select Standard CN Value | 2 | 1 | Area Weighted CN |
| Click Yellow Box | | 2 | Runoff Weighted CN |
| Select CN Modification Method | 2 | 1 | McCuen |
| Click Yellow Box | | 2 | Merkel |
| Select Time of Concentration | 1 | 1 | Travel Time Method |
| Click Yellow Box | | 2 | Lag Time Equation |

**Figure 6.7 Drop Down Boxes to Select CN Averaging Method, CN Modification Method, and Time of Concentration Calculation Method**

### 6.2.3 PRF Calculator

Data entered on this worksheet (Figure 6.8) are used to compute an Area Weighted average PRF value for the contributing watershed. The land use categories and areas are the data entered on the curve number determination worksheet (Figure 6.5). The user selects and enters land use specific PRF values shown in the table on the right side of the worksheet. The average PRF and corresponding gamma function shape parameter value are calculated and shown in shaded cells immediately below the Land Use data cells. These values are used in other worksheets to develop the appropriate SC Unit Hydrograph that is convoluted with the excess distributions for different duration rainfalls to compute storm hydrographs.

**PRF Calculator**

| Land Use | PRF | Area (ac) | Product |
|---|---|---|---|
| Woods | 180 | 50.0 | 9,000 |
| Row Crop | 300 | 50.0 | 15,000 |
| 0 | | 0.0 | 0 |
| 0 | | 0.0 | 0 |
| 0 | | 0.0 | 0 |
| 0 | | 0.0 | 0 |
| 0 | | 0.0 | 0 |
| 0 | | 0.0 | 0 |
| 0 | | 0.0 | 0 |
| 0 | | 0.0 | 0 |
| 0 | | 0.0 | 0 |
| 0 | | 0.0 | 0 |
| 0 | | 0.0 | 0 |
| 0 | | 0.0 | 0 |
| 0 | | 0.0 | 0 |
| Total Watershed Area = 100.0 | ⟷ | 100.0 | 24,000 |
| WS PRF = 240 | | | |
| Shape Parameter, n = 2.02 | | | |

This is the total watershed area computed on the Data for CN Determination sheet.

Basic data entered on this worksheet are used to compute the average PRF value for the contributing watershed. The resulting unit hydrograph is shown in the graph to the right.

**Unshaded Cells are for User Input.**

**Shaded Cells are for Calculated Values.**
**DO NOT TOUCH the shaded cells!**

**PRF Values for South Carolina**

| Land Use | % Impervious | Unit Hydrograph Peak Rate Factor (PRF) |
|---|---|---|
| Open Space | | |
| Poor Condition (grass cover < 50%) | | 250 |
| Fair Condition (grass cover 50-75%) | | 250 |
| Good Condition (grass cover > 75%) | | 250 |
| Impervious Areas (paved parking lots, roofs, etc.) | | 550 |
| Streets and Roads | | |
| Paved with curbs and storm sewers | | 550 |
| Paved with open ditches | | 500 |
| Gravel | | 450 |
| Dirt | | 350 |
| Urban Land Use | | |
| Commercial and Business | 85 | 550 |
| Industrial | 72 | 550 |
| Residential | | |
| 1/8 Acre | 65 | 400 |
| 1/4 Acre | 38 | 375 |
| 1/3 Acre | 30 | 350 |
| 1/2 Acre | 25 | 350 |
| 1 Acre | 20 | 325 |
| 2 Acres | 12 | 300 |
| Developing urban areas, newly graded, no grass cover | | 400 |
| Pasture | | |
| Poor | | 200 |
| Fair | | 190 |
| Good | | 180 |
| Woods | | |
| Poor Condition | | 200 |
| Fair Condition | | 190 |
| Good Condition | | 180 |
| Row Crop | | |
| Straight Row | | 300 |
| Contoured | | 275 |
| Contoured and Terraced | | 250 |

**Figure 6.8 Worksheet for SCUH Peak Rate Factor Calculation**

### 6.2.4 Tc Calculator

Data entered on this worksheet are used to compute the watershed lag time and time of concentration.

#### 6.2.4.1 Lag Time

Lag time is interpreted as the time from the centroid of rainfall excess to the centroid of direct runoff. The NRCS method for watershed lag was developed by Mockus in 1961. It spans a broad set of conditions ranging from heavily forested watersheds with steep channels and a high percent of runoff resulting from subsurface flow, to grasslands providing a high retardance to surface runoff, to smooth land surfaces and large paved areas. The watershed lag equation was developed using data from 24 watersheds ranging in size from 1.3 acres to 9.2 square miles, with most watersheds being less than 2,000 acres (Mockus 1961).

Lag time is determined with the lumped parameter equation, Equation 3.33, that uses single values for watershed length, slope, and retention as shown in Figure 6.9. Watershed retention is computed with the CN Model using the standard 24-hour curve number determined with the land use and soil HSG data entered on the worksheet shown in Figure 6.5.

| Lag Time Equation [3] | |
|---|---|
| Length (feet) = | 1320 |
| Slope (%) = | 0.5 |
| Curve Number = | 67 |
| Watershed Retention S = | 4.86 |
| **Lag Time (min) =** | **48.3** |
| **Time of Concentration (min) =** | **80.7** |

**Figure 6.9 Worksheet Portion That Computes Watershed Lag Time**

The NRCS lag time equation is mostly applied to watersheds in natural conditions, which most often is a single land use or associated land uses, such as forest and pasture or grassland. To get the time of concentration, multiply $t_l$ by 1.67.

### 6.2.4.2 Time of Concentration

Tc is the time for runoff to travel from the hydraulically most distant point of the watershed to a point of interest within the watershed. Tc is computed by the travel times for consecutive components of the drainage conveyance system, i.e., it is determined as the sum of travel times along sheet flow, shallow concentrated flow, and channelized flow path segments. This worksheet computes travel time for each segment as shown in Figures 6.10 and 6.11. For the sake of clarity, the image of the full worksheet was split yielding these two figures.

Figure 6.10 shows typical user input and calculated results of travel times for sheet flow, excess sheet flow, and shallow concentrated flow. The arrow at the bottom of this figure highlights the calculated Travel Time Method Time of Concentration and the Lag Time Equation Time of Concentration in units of minutes.

There are green cells for each flow path segment. To change the surface and flow path type, the user must click on a green cell. A Drop-Down box will appear in the upper right-hand corner. Click on the down arrow in the Drop-Down box. A list of surface and flow path types will appear. Click on the name of your choice. That name will appear in the green cell and appropriate (default or computed) values will appear in shaded cells under the green cell.

With respect to sheet flow, McCuen and Spiess (1995) indicated using one flow length as the limiting variable could lead to bad designs, and proposed the length limitation should instead be based on:

$$l = \frac{100\sqrt{S}}{n} \tag{3.29}$$

where l is length, S is slope and n is Manning's n-value. This limit calculated is calculated for each sheet flow segment and is listed in the cell under the cell where the user enters the sheet flow segment length. If the length limit is less than the user entered value, that value is corrected to the limit value. Extra length is added to the excess sheet flow length.

Figure 6.11shows user input and calculated results for two channelized flow path segments: open channel flow and storm sewers. For open channel flow, the channel geometry is assumed to be trapezoidal or can be approximated as trapezoidal. The stream flow (discharge) calculations use Manning's equation. Similar calculations are done for storm sewer flow. The pipe material cells are green cells. The user clicks on the green cell and selects the appropriate pipe material from a list in the Drop-Down box. For each pipe material there is an assigned Manning's n-value that is entered into the worksheet.

There are cells at the bottom of this figure with the words User Input Velocity in the title box. This was added at the request of SCDOT.

| Travel Time Method | | | | | |
|---|---|---|---|---|---|
| **Sheet Flow** | | | | **Excess Sheet Flow** | |
| Surface = | Short grass prairie | Natural Range | Dense underbrush | Short-grass pasture | |
| Length (ft) = | 300 | | | Length (ft) [2] = | 205.72 |
| Limit (ft) = [1] | 94.28 | 0.08 | 0.01 | Slope (%) = | 2 |
| Corrected Length (ft) = | 94.28 | 0.00 | 0.00 | Velocity constant = | 6.96 |
| Manning's n-value = | 0.150 | 0.130 | 0.800 | Velocity (fps) = | 0.98 |
| Overland Slope (%) = | 2.00 | 0.00 | 0.00 | Travel time (min) = | 3.48 |
| 2-yr 24-hr P (inches) = | 3.76 | 3.76 | 3.76 | | |
| Travel time (min) = | **8.62** | **0.00** | **0.00** | | |
| | | Sum = | **8.62** | | |

| | |
|---|---|
| Cells shaded with this color involve a Drop Down menu Click on the light green shaded cells. | |
| Use Drop Down to select Sheet Flow Surface, Shallow Concentrated Flow Type and Storm Sewer Pipe Material. Click on an appropriate cell and the Drop Down symbol will appear next to the upper right hand corner. | |

| Shallow Concentrated Flow | | | | | |
|---|---|---|---|---|---|
| Shallow Flow Type = | Forest with heavy ground litter and hay meadows | Cultivated straight row crops | Short-grass pasture | Minimum cultivation, contour or strip-cropped, and woodlands | Pavement and small upland gullies |
| Length (ft) = [2] | 100 | 110 | 130 | 120 | 140 |
| Slope (%) = | 0.5 | 1 | 2 | 2 | 2 |
| Velocity constant = | 2.52 | 8.76 | 6.96 | 5.03 | 20.33 |
| Velocity (fps) = | 0.18 | 0.88 | 0.98 | 0.71 | 2.87 |
| Travel time (min) = | **9.37** | **2.09** | **2.20** | **2.81** | **0.81** |
| | | | | Sum = | **17.28** |

| Method | Time of Concentration |
|---|---|
| **Travel Time Method Time of Concentration (min) =** | **58.72** |
| **Lag Time Equation Time of Concentration (min) =** | **80.67** |

| |
|---|
| **User enters preferred $t_c$ value on Watershed & UH Data & Runoff Results worksheet with a Drop Down Menu.** |

**Figure 6.10 Worksheet Portion That Computes Basin Travel Time for Sheet Flow and Shallow Concentrated Flow Path Segments**

| Channelized Flow - Open Channel[56] | | | | |
|---|---|---|---|---|
| Base Width (ft) = | 3 | 3 | 3 | 3 |
| Front Slope (Z hor:1 vert) = | 2 | 2 | 2 | 2 |
| Back Slope (Z hor:1 vert) = | 3 | 3 | 3 | 3 |
| Channel Depth (ft) = | 2 | 2 | 2 | 2 |
| Length (ft) = | 1,500 | 1,500 | 0 | 0 |
| Channel Bed Slope (%) = | 0.25 | 0.25 | 0.25 | 0.25 |
| Manning n-value = | 0.035 | 0.035 | 0.035 | 0.035 |
| Stream Flow (cfs) = | 38.0 | 38.0 | 38.0 | 38.0 |
| Wetted Perimeter (ft) = | 13.80 | 13.80 | 13.80 | 13.80 |
| Cross-sectional Area (sq ft) = | 16 | 16.00 | 16 | 16 |
| Velocity (fps) = | 2.37 | 2.37 | 2.37 | 2.37 |
| Travel time (min) = | **10.53** | **10.53** | **0.00** | **0.00** |
| | | | **Sum =** | **21.06** |

| Channelized Flow - Storm Sewer | | | | |
|---|---|---|---|---|
| Pipe Material = | **CMP** | **PVC** | **Concrete** | **Steel** |
| Diameter (in) = | 36 | 24 | 30 | 36 |
| Length (ft) = | 300 | 300 | 300 | 300 |
| Slope (%) = | 0.50 | 0.50 | 0.50 | 0.50 |
| Manning n-value = | 0.024 | 0.01 | 0.013 | 0.013 |
| Pipe Flow (cfs) = | 25.55 | 20.80 | 29.00 | 47.16 |
| Cross-sectional Area (sq ft) = | 4.71 | 3.14 | 3.93 | 4.71 |
| Velocity (fps) = | 5.4 | 6.6 | 7.4 | 10.0 |
| Travel time (min) = | **0.92** | **0.76** | **0.68** | **0.50** |
| | | | **Sum =** | **2.85** |

| Channelized Flow (Storm Sewer and/or Open Channel) - User Input Velocity | | | | |
|---|---|---|---|---|
| Length (ft) = | 0 | 300 | 300 | 300 |
| Velocity (fps) = | 0.0 | 2.0 | 3.0 | 4.0 |
| Travel time (min) = | **0.00** | **2.50** | **1.67** | **1.25** |
| | | | **Sum =** | **5.42** |

**Figure 6.11 Worksheet Portion That Computes Travel Time for Channelized Flow Path Segments**

### 6.2.5 WS & UH Data & Runoff Results

The WS & UH Data & Runoff Results worksheet shows Summary Watershed and Unit Hydrograph Data, the Drop-Down menus discussed in Section 6.2.2, and D-Hour Rainfall Data, Runoff Depths, and Peak Runoff Rates for storms with Annual Exceedance Probabilities of 10, 4, 2, and 1 percent. A portion of the worksheet with data for storms with Annual Exceedance Probabilities of 10 and 4 percent is shown in Figure 6.12. The portion with data for storms with AEP of 2 and 1 percent was omitted to achieve better size on the page and clarity of the figure. For greater clarity, Figure 6.12 was split yielding Figures 6.13 and 6.14.

| Summary Watershed and Unit Hydrograph Data | | | |
|---|---|---|---|
| **Watershed Data** | | **Unit Hydrograph Data** | |
| Area (acres) = | 100.0 | Burst Dur (min) = | 6 |
| Tc (minutes) = | 58.7 | Γ-fn n = | 2.02 |
| PRF = | 240 | Lag time (min) = | 35.2 |
| Standard CN = | 67.3 | Adj Tc (min) = | 60.0 |
| WS Retention S = | 4.86 | UH_Tp = | 42.0 |
| Initial Abstraction Ia = | 0.97 | UH_Qp = | 53.6 |

| Drop Down boxes to select CN, CN Modification Method, and Tc | | | |
|---|---|---|---|
| Select Standard CN Value | 2 | 1 | Area Weighted CN |
| Click Yellow Box | | 2 | Runoff Weighted CN |
| Select CN Modification Method | 2 | 1 | McCuen |
| Click Yellow Box | | 2 | Merkel |
| Select Time of Concentration | 1 | 1 | Travel Time Method |
| Click Yellow Box | | 2 | Lag Time Equation |

**For the Following Tables**

| Design Rainfall Distribution = | NOAA B | Curve Number Adjustment Method = | Merkel |
|---|---|---|---|

In each table, the D-hr design storm duration with the maximum runoff volume is flagged with a Blue Arrow in the left side column and the duration with maximum peak runoff is flagged with a Red Arrow in the right side column. **These are *Critical Durations*.**

**D-Hour Rainfall Data, Runoff Depths, and Peak Runoff Rates**

**The following cells report runoff data for storms with Annual Exceedance Probabilities of 10, 4, 2 and 1 percent.**

Annual Exceedance Probablity (AEP) = 10 % ; Design Storm Return Period = 10 Years

| | Runoff Results for 10 Year D-Hour Rainfall Events | | | | | | |
|---|---|---|---|---|---|---|---|
| | Storm Duration (hrs) | Rainfall Depth (in) | CN Adjusted for Rainfall Duration <24-hr | Runoff Volume $Q_{CN}$ (WS-in) | Peak Runoff $Q_p$ (cfs) | Time of Peak Runoff (min) | |
| | 1 | 2.66 | 89.5 | 1.64 | 85.14 | 78 | |
| | 2 | 3.22 | 89.2 | 2.10 | 100.30 | 120 | |
| | 3 | 3.45 | 88.6 | 2.26 | 99.03 | 144 | |
| | 6 | 4.07 | 86.2 | 2.63 | 102.01 | 234 | <= |
| => | 12 | 4.78 | 80.4 | 2.82 | 97.07 | 414 | |
| | 24 | 5.74 | 67.3 | 2.36 | 69.22 | 780 | |

Annual Exceedance Probablity (AEP) = 4 % ; Design Storm Return Period = 25 Years

| | Runoff Results for 25 Year D-Hour Rainfall Events | | | | | | |
|---|---|---|---|---|---|---|---|
| | Storm Duration (hrs) | Rainfall Depth (in) | CN Adjusted for Rainfall Duration <24-hr | Runoff Volume $Q_{CN}$ (WS-in) | Peak Runoff $Q_p$ (cfs) | Time of Peak Runoff (min) | |
| | 1 | 3.13 | 89.9 | 2.06 | 107.17 | 78 | |
| | 2 | 3.85 | 89.5 | 2.69 | 127.85 | 114 | |
| | 3 | 4.17 | 88.7 | 2.92 | 128.33 | 144 | |
| | 6 | 4.94 | 86.1 | 3.43 | 133.41 | 234 | <= |
| => | 12 | 5.84 | 80.1 | 3.77 | 130.45 | 414 | |
| | 24 | 7.04 | 67.3 | 3.37 | 100.93 | 774 | |

**Figure 6.12 Summary Watershed and Unit Hydrograph Results and Simulated Runoff Results**

Figure 6.13 shows summary watershed and unit hydrograph data and runoff results for 1, 2, 3, 6, 12, and 24-hour storms with 10% AEP. The runoff results include rainfall depths, storm duration adjusted CN values, runoff volumes, peak flow rates, and times to peak for the different duration storms. A key result is the identification of critical duration events which are storm durations with the maximum runoff volume and maximum peak runoff. A recurring outcome among different simulations is the critical durations are not 24 hours. It is important and ethically correct that designs must consider the critical durations and not only regulatory 24-hour results.

The critical durations in Figures 6.13 and 6.14 are 12 hours for maximum runoff volume and 6 hours for peak runoff rate. The critical duration runoff volumes are 19% (10% AEP) and 59% (4% AEP) greater than the 24-hour runoff volumes. The critical duration peak runoff rates are 43% (10% AEP) and 32% (4% AEP) greater than the 24-hour peak runoff rates. These are typical results and affirm SCDOT designs must consider the critical duration storms.

As noted by SCDOT in the Requirements for Hydraulic Design Studies manual, for design and check events, the 24-hour, runoff volume critical duration, and peak discharge critical duration rainfalls shall be used. With respect to stormwater ponds, use the 24-hour, critical peak flowrate, and critical runoff volume rainfall durations during the design process. Select the safest design as **The Design**. Outflow from the pond shall not increase peak flowrate for the 10, 4, 2, and 1% AEP events.

| Summary Watershed and Unit Hydrograph Data | | | |
|---|---|---|---|
| **Watershed Data** | | **Unit Hydrograph Data** | |
| Area (acres) = | 100.0 | Burst Dur (min) = | 6 |
| Tc (minutes) = | 58.7 | Γ-fn n = | 2.02 |
| PRF = | 240 | Lag time (min) = | 35.2 |
| Standard CN = | 67.3 | Adj Tc (min) = | 60.0 |
| WS Retention S = | 4.86 | UH_Tp = | 42.0 |
| Initial Abstraction Ia = | 0.97 | UH_Qp = | 53.6 |

For the F

| | |
|---|---|
| Design Rainfall Distribution = | NOAA B |

In each table, the D-hr design storm duration with the maximum runoff vo

maximum peak runoff is flagged with a Red Arrow in the right side colum

D-Hour Rainfall Data, Runof

The following cells report runoff data for storms with Annu

| | | |
|---|---|---|
| Annual Exceedance Probablity (AEP) = | 10 | % |
| Design Storm Return Period = | 10 | Years |

**Runoff Results for 10 Year D-Hour Rainfall Events**

| | Storm Duration (hrs) | Rainfall Depth (in) | CN Adjusted for Rainfall Duration <24-hr | Runoff Volume $Q_{CN}$ (WS-in) | Peak Runoff $Q_p$ (cfs) | Time of Peak Runoff (min) | |
|---|---|---|---|---|---|---|---|
| | 1 | 2.66 | 89.5 | 1.64 | 85.14 | 78 | |
| | 2 | 3.22 | 89.2 | 2.10 | 100.30 | 120 | |
| | 3 | 3.45 | 88.6 | 2.26 | 99.03 | 144 | |
| | 6 | 4.07 | 86.2 | 2.63 | 102.01 | 234 | <= |
| => | 12 | 4.78 | 80.4 | 2.82 | 97.07 | 414 | |
| | 24 | 5.74 | 67.3 | 2.36 | 69.22 | 780 | |

**Figure 6.13 Summary Watershed and Unit Hydrograph Results and Simulated Runoff Results for a Storm with Annual Exceedance Probability of 10%**

Figure 6.14 shows summary watershed and unit hydrograph data and runoff results for 1, 2, 3, 6, 12, and 24-hour storms with 4% AEP and the Drop-Down menus discussed in Section 6.2.2. The Drop-Down menus allow users to select the preferred method to determine the average CN, to choose either the McCuen or Merkel method to adjust CN for rainfall durations less than 24 hours, and to choose between the Travel Time and Lag Time methods to compute watershed time of concentration.

| Drop Down boxes to select CN, CN Modification Method, and Tc | | | |
|---|---|---|---|
| Select Standard CN Value | 2 | 1 | Area Weighted CN |
| Click Yellow Box | | 2 | Runoff Weighted CN |
| Select CN Modification Method | 2 | 1 | McCuen |
| Click Yellow Box | | 2 | Merkel |
| Select Time of Concentration | 1 | 1 | Travel Time Method |
| Click Yellow Box | | 2 | Lag Time Equation |

ollowing Tables

| Curve Number Adjustment Method = | Merkel |
|---|---|

olume is flagged with a Blue Arrow in the left side column and the duration with

n. These are *Critical Durations*.

ff Depths, and Peak Runoff Rates

ual Exceedance Probabilities of 10, 4, 2 and 1 percent.

| | | |
|---|---|---|
| Annual Exceedance Probablity (AEP) = | 4 | % |
| Design Storm Return Period = | 25 | Years |

**Runoff Results for 25 Year D-Hour Rainfall Events**

| | Storm Duration (hrs) | Rainfall Depth (in) | CN Adjusted for Rainfall Duration <24-hr | Runoff Volume $Q_{CN}$ (WS-in) | Peak Runoff $Q_p$ (cfs) | Time of Peak Runoff (min) | |
|---|---|---|---|---|---|---|---|
| | 1 | 3.13 | 89.9 | 2.06 | 107.17 | 78 | |
| | 2 | 3.85 | 89.5 | 2.69 | 127.85 | 114 | |
| | 3 | 4.17 | 88.7 | 2.92 | 128.33 | 144 | |
| | 6 | 4.94 | 86.1 | 3.43 | 133.41 | 234 | <= |
| => | 12 | 5.84 | 80.1 | 3.77 | 130.45 | 414 | |
| | 24 | 7.04 | 67.3 | 3.37 | 100.93 | 774 | |

**Figure 6.14 Drop-Down Menus to Select CN, CN Modification Method, and Tc and Simulated Runoff Results for a Storm with Annual Exceedance Probability of 4%**

### 6.2.6 D-hr Runoff Ordinates

Figure 6.15 lists runoff hydrograph ordinates from time 0 to 174 minutes for 1, 2, 3, 6, 12, and 24-hour storms with 10% AEP. This time frame does not include non-zero runoff ordinates for the 12 and 24-hour events. As noted in Section 6.2.5, only a portion of the worksheet was included to achieve better size and clarity of the figure. The worksheet lists ordinates from start to end of runoff. The worksheets for D-hour storms with 4, 2, and 1% AEP are similar.

One aspect of this worksheet is the peak flow rate and time of peak flow for each duration storm are listed on the second and third lines under the headings labeled 1 Hr Q, 2 Hr Q, etc.

| | 1 Hr Q | | 2 Hr Q | | 3 Hr Q | | 6 Hr Q | | 12 Hr Q | | 24 Hr Q | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Peak = | 85.14 | | 100.30 | | 99.03 | | 102.01 | | 97.07 | | 69.22 | |
| Time = | 78 | | 120 | | 144 | | 234 | | 414 | | 780 | |
| | D-Hour Storm Hydrograph Ordinates | | | | | | | | | | | |
| | Annual Exceedance Probablity (AEP) = | | | | | 10 | % | Return Period = | | | 10 | Years |
| | 1 Hr Q | Time | 2 Hr Q | Time | 3 Hr Q | Time | 6 Hr Q | Time | 12 Hr Q | Time | 24 Hr Q | Time |
| | 0.00 | 0 | 0.00 | 0 | 0.00 | 0 | 0.00 | 0 | 0.00 | 0 | 0.00 | 0 |
| | 0.00 | 6 | 0.00 | 6 | 0.00 | 6 | 0.00 | 6 | 0.00 | 6 | 0.00 | 6 |
| | 0.01 | 12 | 0.00 | 12 | 0.00 | 12 | 0.00 | 12 | 0.00 | 12 | 0.00 | 12 |
| | 0.61 | 18 | 0.00 | 18 | 0.00 | 18 | 0.00 | 18 | 0.00 | 18 | 0.00 | 18 |
| | 2.90 | 24 | 0.00 | 24 | 0.00 | 24 | 0.00 | 24 | 0.00 | 24 | 0.00 | 24 |
| | 9.51 | 30 | 0.10 | 30 | 0.00 | 30 | 0.00 | 30 | 0.00 | 30 | 0.00 | 30 |
| | 24.99 | 36 | 0.61 | 36 | 0.00 | 36 | 0.00 | 36 | 0.00 | 36 | 0.00 | 36 |
| | 40.36 | 42 | 1.76 | 42 | 0.01 | 42 | 0.00 | 42 | 0.00 | 42 | 0.00 | 42 |
| | 54.20 | 48 | 4.03 | 48 | 0.12 | 48 | 0.00 | 48 | 0.00 | 48 | 0.00 | 48 |
| | 65.82 | 54 | 8.20 | 54 | 0.40 | 54 | 0.00 | 54 | 0.00 | 54 | 0.00 | 54 |
| | 75.46 | 60 | 16.61 | 60 | 0.94 | 60 | 0.00 | 60 | 0.00 | 60 | 0.00 | 60 |
| | 81.40 | 66 | 33.35 | 66 | 2.03 | 66 | 0.00 | 66 | 0.00 | 66 | 0.00 | 66 |
| | 84.41 | 72 | 49.43 | 72 | 3.76 | 72 | 0.00 | 72 | 0.00 | 72 | 0.00 | 72 |
| | 85.14 | 78 | 63.55 | 78 | 6.57 | 78 | 0.00 | 78 | 0.00 | 78 | 0.00 | 78 |
| | 84.15 | 84 | 75.15 | 84 | 11.16 | 84 | 0.00 | 84 | 0.00 | 84 | 0.00 | 84 |
| | 81.88 | 90 | 84.55 | 90 | 19.69 | 90 | 0.00 | 90 | 0.00 | 90 | 0.00 | 90 |
| | 78.69 | 96 | 91.38 | 96 | 36.01 | 96 | 0.00 | 96 | 0.00 | 96 | 0.00 | 96 |
| | 74.87 | 102 | 96.06 | 102 | 51.52 | 102 | 0.03 | 102 | 0.00 | 102 | 0.00 | 102 |
| | 70.64 | 108 | 98.92 | 108 | 65.03 | 108 | 0.10 | 108 | 0.00 | 108 | 0.00 | 108 |
| | 66.19 | 114 | 100.25 | 114 | 76.06 | 114 | 0.24 | 114 | 0.00 | 114 | 0.00 | 114 |
| | 61.65 | 120 | 100.30 | 120 | 84.89 | 120 | 0.46 | 120 | 0.00 | 120 | 0.00 | 120 |
| | 57.12 | 126 | 98.54 | 126 | 91.24 | 126 | 0.80 | 126 | 0.00 | 126 | 0.00 | 126 |
| | 52.69 | 132 | 95.45 | 132 | 95.50 | 132 | 1.30 | 132 | 0.00 | 132 | 0.00 | 132 |
| | 48.41 | 138 | 91.40 | 138 | 98.00 | 138 | 1.99 | 138 | 0.00 | 138 | 0.00 | 138 |
| | 44.33 | 144 | 86.70 | 144 | 99.03 | 144 | 2.90 | 144 | 0.00 | 144 | 0.00 | 144 |
| | 40.46 | 150 | 81.61 | 150 | 98.83 | 150 | 4.05 | 150 | 0.00 | 150 | 0.00 | 150 |
| | 36.82 | 156 | 76.30 | 156 | 97.60 | 156 | 5.80 | 156 | 0.00 | 156 | 0.00 | 156 |
| | 33.43 | 162 | 70.94 | 162 | 95.57 | 162 | 8.13 | 162 | 0.00 | 162 | 0.00 | 162 |
| | 30.28 | 168 | 65.63 | 168 | 92.92 | 168 | 11.48 | 168 | 0.00 | 168 | 0.00 | 168 |
| | 27.37 | 174 | 60.46 | 174 | 89.79 | 174 | 16.50 | 174 | 0.00 | 174 | 0.00 | 174 |

**Figure 6.15 D-Hour Storm Hydrograph Ordinates**

### 6.2.7 N-yr D-hr Storm Hydrographs

Figure 6.16 shows the N-yr D-hr Storm Hydrographs worksheet for a storm with 4% AEP. Rainfall data for the 1, 2, 3, 6, 12, and 24-hour events are entered on the Rainfall Data worksheet shown in Figure 6.1. The rainfall data are used to simulate runoff hydrographs for which the runoff depths, peak flow rate, and time of peak runoff are shown on this worksheet. Other results include ordinates and plots of the runoff hydrographs. There are equivalent worksheets for storms with 10, 4, 2, and 1% AEP.

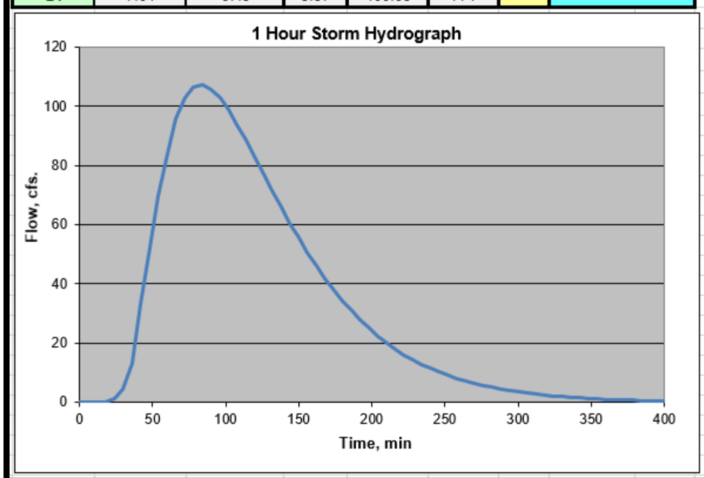
Note the red arrow flags the critical storm duration with maximum peak runoff. For this application, the critical duration is 6 hours and the peak flow rate is 32% greater than the 24-hour peak runoff rate. Although it is not flagged, the maximum runoff volume critical duration is 12 hours and the runoff volume is 12% greater than the 24-hour volume.

**Storms with Annual Exceedance Probability = 4 %**

**Runoff Results for 25 Year D-Hour Rainfall Events**

| Storm Duration (hrs) | Rainfall Depth (in) | CN Adjusted for Rainfall Duration <24-hr | Runoff Volume $Q_{CN}$ (WS-in) | Peak Runoff $Q_p$ (cfs) | Time of Peak Runoff (min) | |
|---|---|---|---|---|---|---|
| 1 | 3.13 | 89.9 | 2.06 | 107.17 | 78 | |
| 2 | 3.85 | 89.5 | 2.69 | 127.85 | 114 | |
| 3 | 4.17 | 88.7 | 2.92 | 128.33 | 144 | |
| 6 | 4.94 | 86.1 | 3.43 | 133.41 | 234 | <= |
| 12 | 5.84 | 80.1 | 3.77 | 130.45 | 414 | |
| 24 | 7.04 | 67.3 | 3.37 | 100.93 | 774 | |

The Red Arrow flags the critical storm duration with maximum peak runoff.




| Time, min | D-Hour Storm Hydrograph Ordinates 1 Hr | 2 Hr | 3 Hr | 6 Hr | 12 Hr | 24 Hr |
|---|---|---|---|---|---|---|
| 0 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 6 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 12 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 18 | 0.08 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 24 | 1.12 | 0.04 | 0.00 | 0.00 | 0.00 | 0.00 |
| 30 | 4.47 | 0.32 | 0.00 | 0.00 | 0.00 | 0.00 |
| 36 | 13.29 | 1.26 | 0.01 | 0.00 | 0.00 | 0.00 |
| 42 | 32.96 | 3.10 | 0.11 | 0.00 | 0.00 | 0.00 |
| 48 | 52.25 | 6.44 | 0.41 | 0.00 | 0.00 | 0.00 |
| 54 | 69.46 | 12.26 | 1.01 | 0.00 | 0.00 | 0.00 |
| 60 | 83.81 | 23.43 | 1.98 | 0.00 | 0.00 | 0.00 |
| 66 | 95.61 | 44.88 | 3.75 | 0.00 | 0.00 | 0.00 |
| 72 | 102.83 | 65.27 | 6.39 | 0.00 | 0.00 | 0.00 |
| 78 | 106.41 | 83.04 | 10.45 | 0.00 | 0.00 | 0.00 |
| 84 | 107.17 | 97.53 | 16.82 | 0.00 | 0.00 | 0.00 |
| 90 | 105.80 | 109.15 | 28.24 | 0.03 | 0.00 | 0.00 |
| 96 | 102.86 | 117.51 | 49.38 | 0.11 | 0.00 | 0.00 |
| 102 | 98.78 | 123.13 | 69.29 | 0.25 | 0.00 | 0.00 |
| 108 | 93.93 | 126.45 | 86.50 | 0.49 | 0.00 | 0.00 |
| 114 | 88.58 | 127.85 | 100.41 | 0.83 | 0.00 | 0.00 |
| 120 | 82.97 | 127.64 | 111.46 | 1.31 | 0.00 | 0.00 |
| 126 | 77.25 | 125.21 | 119.29 | 1.97 | 0.00 | 0.00 |
| 132 | 71.55 | 121.12 | 124.43 | 2.85 | 0.00 | 0.00 |
| 138 | 65.99 | 115.86 | 127.32 | 3.99 | 0.00 | 0.00 |
| 144 | 60.61 | 109.81 | 128.33 | 5.43 | 0.00 | 0.00 |
| 150 | 55.49 | 103.28 | 127.79 | 7.20 | 0.00 | 0.00 |
| 156 | 50.63 | 96.51 | 125.95 | 9.77 | 0.00 | 0.00 |
| 162 | 46.08 | 89.68 | 123.10 | 13.12 | 0.00 | 0.00 |
| 168 | 41.83 | 82.93 | 119.48 | 17.79 | 0.00 | 0.00 |
| 174 | 37.88 | 76.37 | 115.28 | 24.64 | 0.00 | 0.00 |
| 180 | 34.24 | 70.06 | 110.66 | 36.37 | 0.00 | 0.00 |

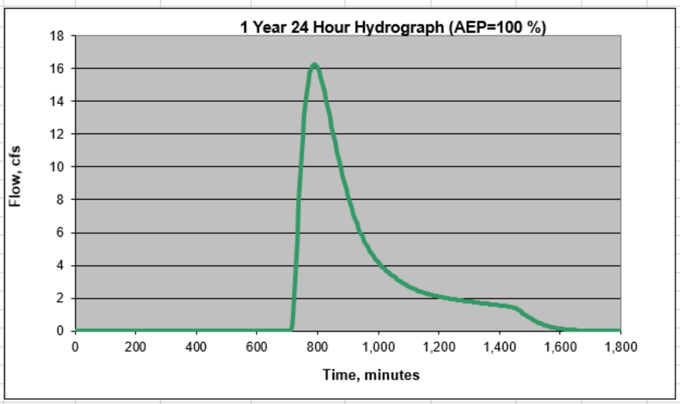
**Figure 6.16 Runoff Results for Storms with Annual Exceedance Probability of 4%**

### 6.2.8 24-hr Storm Hydrographs

Figure 6.17 shows the Runoff Results for 24-Hour Rainfall Events worksheet with hydrograph ordinates listed through 180 minutes. Rainfall data for the seven 24-hour events are entered on the Rainfall Data worksheet shown in Figure 6.1. The rainfall data are used to simulate runoff hydrographs for which the runoff depths, peak flow rate, and time of peak runoff are shown on this worksheet. Other results include hydrograph ordinates and plots of runoff hydrographs for the seven events.

**Runoff Results for 24-Hour Rainfall Events**

| Runoff Results for 24-Hour Rainfall Events | | | | | |
|---|---|---|---|---|---|
| AEP (%) | Return Period (yrs) | Rainfall Depth (in) | Runoff Volume $Q_{CN}$ (WS-in) | Peak Runoff, $Q_p$ (cfs) | Time of Peak Runoff (min) |
| 100 | 1 | 3.09 | 0.64 | 16.23 | 792 |
| 50 | 2 | 3.76 | 1.02 | 27.42 | 786 |
| 20 | 5 | 4.84 | 1.71 | 49.00 | 780 |
| 10 | 10 | 5.74 | 2.36 | 69.22 | 780 |
| 4 | 25 | 7.04 | 3.37 | 100.93 | 774 |
| 2 | 50 | 8.12 | 4.25 | 128.77 | 774 |
| 1 | 100 | 9.29 | 5.25 | 159.88 | 774 |




| Time, min | 24-Hour Storm Hydrograph Ordinates | | | | | | |
|---|---|---|---|---|---|---|---|
| | Annual Exceedance Probability | | | | | | |
| | 100% | 50% | 20% | 10% | 4% | 2% | 1% |
| 0 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 6 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 12 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 18 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 24 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 30 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 36 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 42 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 48 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 54 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 60 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 66 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 72 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 78 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 84 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 90 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 96 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 102 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 108 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 114 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 120 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 126 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 132 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 138 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 144 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 150 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 156 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 162 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 168 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 174 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 180 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |

**Figure 6.17 Runoff Results for 24-Hour Rainfall Events**

### 6.2.9 Design Storm Pond Results

This worksheet (Figure 6.18) is where users enter data to design a stormwater pond. There are two options. The first (Proposed Pond 1) is to size an inverted quadrilateral frustum and the second (Proposed Pond 2) is to use pond surface areas at different elevations. The second option will allow users to design a pond in an existing swale, valley, or graded area where a pond can be formed by constructing a dam. To select the pond option, click on the Select Pond Option cell. A Drop-Down menu will allow the user to select the desired option.

Both options require the design of the outflow structure. Candidate outflow devices are first and/or second stage orifices, a third stage rectangular weir, and an overflow spillway. The orifices and weir would be used with detention or retention-detention ponds. Additional outflow to be considered is seepage through the pond bottom, particularly for retention ponds, and seepage through the pond side slopes.

What is an inverted quadrilateral frustum?  Think of a four-sided pyramid. Remove the top and invert it. That geometry if an inverted quadrilateral frustum.

This worksheet shows pond results for the 1, 2, 3, 6, 12, and 24-hour events with 10% AEP. The user inputs the design storm AEP is an unshaded cell highlighted by a red arrow not shown in Figure 6.18. Some of the results for each duration storm include rainfall and runoff depths, magnitude and time of pond peak inflow and outflow, times to drain the first one-half inch, first inch and first two inches of inflow and the maximum ponding depth. Data about the pond peak inflow and outflow, and maximum ponding depth, relate to watershed hydrology and pond routing results. The times to drain different depths also relate to pond routing but are primary parameters related to pond water quality performance.

The red arrows flag the critical storm duration with maximum peak outflow and ponding depth. The results shown in Figure 6.18 highlight the important point that stormwater pond performances should be checked for multiple duration storms and not just the regulatory 24-hour design event. For this application, the pond peak outflow for all shorter duration events is greater than the peak outflow for the 24-hour event. The 1-hour peak outflow from the pond is 8.07% greater than the 24-hour peak outflow. For the other events (2, 3, 6, and 12-hour), the peak outflow ranges from 60.62% to 75.31% greater than the peak outflow for the 24-hour event. The maximum difference is for the 6-hour event that was flagged with red arrows.

The trial pond that is the basis for the results in Figure 6.18 probably should be redesigned. Why?  The maximum ponding depth is above the crest of the overflow spillway meaning the pond is overflowing, which is not acceptable pond performance for a design event.

For any design and check, the 24-hour, runoff volume critical duration, and peak discharge critical duration rainfalls shall be used. Ethically, one should select the safest design.

\

## Data for Pond Stage-Storage and Stage-Outflow Ratings

### Data for Pond Stage-Storage Ratings

User Has Two Options and **Must Select One.** Follow the Arrow.

| Proposed Pond 1 | | Proposed Pond 2 | |
|---|---|---|---|
| Inverted Quadrilateral Frustum | | Ellevation (ft-MSL) | Surface Area (sq ft) |
| Length (feet ) = | 200 | 100.00 | 2,000 |
| $W_1$ (feet) = | 200 | 101.00 | 2,100 |
| $W_2$ (feet) = | 200 | 102.00 | 2,200 |
| Side Slope Z = | 3.0 | 103.00 | 2,400 |
| Bottom Slope $S_o$ (%) = | 0.5 | 104.00 | 2,900 |
| | | 105.00 | 3,300 |
| | | 106.00 | 3,700 |
| Data Common to Both Options | | 107.00 | 4,000 |
| Bottom Elev (ft) = | 100 | 108.00 | 4,400 |
| Max Depth (ft) = | 10 | 109.00 | 4,800 |

| Select Pond Option |
|---|
| Proposed Pond 1 |

**Yellow data cells are default values.**

### Outflow Devices

| Circular Orifices | 1st Stage | 2nd Stage |
|---|---|---|
| Orifice coefficent = | 0.60 | 0.60 |
| Diameter (inches) = | 6 | 6 |
| Centerline elevation above bottom of pond (feet) = | 0.50 | 2.00 |
| Number of openings = | 1 | 1 |
| Retention Depth (feet) = | 0.25 | |

| 3rd Stage Rectangular Weir | |
|---|---|
| Weir coefficent = | 3.30 |
| Weir exponent = | 1.5 |
| Length (feet) = | 2.0 |
| Crest elev above pond bottom (ft) = | 4.00 |
| Number of weirs = | 1 |

| Overflow Spillway | |
|---|---|
| Broad-crested weir coefficent = | 3.00 |
| Weir exponent = | 1.5 |
| Length (feet) = | 20.0 |
| Crest elevation above pond bottom (ft) = | 6.0 |
| Seepage through pond bottom (in/hr) = | 2 |
| Seepage through pond side slopes (in/hr) = | 4 |

**User specifies Design Storm by entering the event AEP in the WHITE cell below.**
**This spreadsheet was developed for storms with AEP values of 10, 4, 2 and 1.**

| Annual Exceedance Probablity (AEP) = | 10 | % | Design Storm Return Period = | 10 | Years |
|---|---|---|---|---|---|

**Pond Results for 1, 2, 3, 6, 12 and 24 Hour Design Storms with AEP = 10 %**

| | Storm Duration (hrs) | Rainfall Depth (in) | Runoff Volume $Q_{CN}$ (WS-in) | Peak Runoff, $Q_p$ (cfs) | Pond Peak Inflow (cfs) | Time of Peak Inflow (min) | Pond Peak Outflow (cfs) | Time of Peak Outflow (min) | Time to Drain First One-Half Inch (hrs) | Time to Drain First Inch (hrs) | Time to Drain First Two Inches (hrs) | Maximum Ponding Depth (ft) | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2.66 | 1.64 | 85.14 | 85.14 | 78 | 53.62 | 132 | 1.40 | 3.00 | NA | 6.36 | |
| | 2 | 3.22 | 2.10 | 100.30 | 100.30 | 120 | 79.69 | 150 | 1.00 | 1.40 | 13.10 | 6.70 | |
| | 3 | 3.45 | 2.26 | 99.03 | 99.03 | 144 | 83.13 | 186 | 1.00 | 1.20 | 8.80 | 6.75 | |
| => | 6 | 4.07 | 2.63 | 102.01 | 102.01 | 234 | 86.98 | 270 | 1.00 | 1.10 | 3.20 | 6.80 | <= |
| | 12 | 4.78 | 2.82 | 97.07 | 97.07 | 414 | 81.99 | 450 | 1.00 | 1.10 | 3.20 | 6.73 | |
| | 24 | 5.74 | 2.36 | 69.22 | 69.22 | 780 | 49.61 | 840 | 1.50 | 2.30 | 8.90 | 6.31 | |

**Figure 6.18 Input Data and Summary Results for Stormwater Pond Performance**

### 6.2.10 D-hr Storm Pond Routing Results

The D-hr Storm Pond Routing Results worksheet shown in Figure 6.19 lists Runoff and Pond Routing Results for the 1, 2, 3, 6, 12, and 24-hour events with 10% AEP. Some of the listed data for each AEP storm include rainfall and runoff depths, magnitude and time of pond peak inflow and outflow, times to drain the first one-half inch, first inch and first two inches of inflow, and the maximum ponding depth. Data about the pond peak inflow and outflow, and maximum ponding depth, relate to watershed hydrology and pond routing hydraulics. The times to drain different depths also relate to pond routing but are primary parameters related to pond water quality performance.

The worksheet includes a table that lists the inflow and outflow hydrograph ordinates and plots of the inflow and outflow hydrographs. Figure 6.20 shows a sample table for storms with 10% AEP.

| Pond Design Storm | | | | | |
|---|---|---|---|---|---|
| Annual Exceedance Probablity (AEP) = | 10 | % | Design Storm Return Period = | 10 | Years |

**Runoff and Pond Routing Results for D-Hour Design Storm Rainfall Events**

A result of #VALUE! indicates non-convergence of the pond routing routine and is not a problem with the spreadsheet. User needs to upsize the pond or outflow structure. This also is the reason for red ##### symbols in column J.

| Storm Duration (hrs) | Rainfall Depth (in) | CN Adjusted for Rainfall Duration <24-hr | Runoff Volume $Q_{cn}$ (WS-in) | Pond Peak Inflow (cfs) | Time of Pond Peak Inflow (min) | Pond Peak Outflow (cfs) | Time of Pond Peak Outflow (min) |
|---|---|---|---|---|---|---|---|
| 1 | 2.66 | 89.5 | 1.64 | 85.14 | 78 | 53.62 | 132 |
| 2 | 3.22 | 89.2 | 2.10 | 100.30 | 120 | 79.69 | 150 |
| 3 | 3.45 | 88.6 | 2.26 | 99.03 | 144 | 83.13 | 186 |
| 6 | 4.07 | 86.2 | 2.63 | 102.01 | 234 | 86.98 | 270 |
| 12 | 4.78 | 80.4 | 2.82 | 97.07 | 414 | 81.99 | 450 |
| 24 | 5.74 | 67.3 | 2.36 | 69.22 | 780 | 49.61 | 840 |

<== The Red Arrow flags the critical storm duration that produces the maximum peak outflow from the pond.

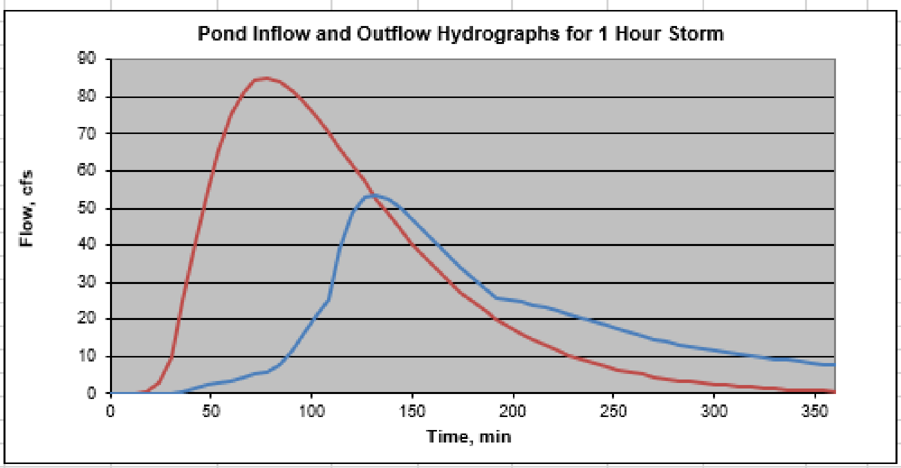



**Figure 6.19 Summary Results for Stormwater Pond Performance for a Storm with AEP of 10%**

| Time, min | D-Hour Storm Pond Routing Inflow and Outflow Hydrograph Ordinates | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1 Hr | | 2 Hr | | 3 Hr | | 6 Hr | | 12 Hr | | 24 Hr | |
| | Inflow | Outflow | Inflow | Outflow | Inflow | Outflow | Inflow | Outflow | Inflow | Outflow | Inflow | Outflow |
| 0 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 6 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 12 | 0.01 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 18 | 0.61 | 0.01 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 24 | 2.90 | 0.05 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 30 | 9.51 | 0.19 | 0.10 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 36 | 24.99 | 0.60 | 0.61 | 0.01 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 42 | 40.36 | 1.36 | 1.76 | 0.04 | 0.01 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 48 | 54.20 | 2.45 | 4.03 | 0.11 | 0.12 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 54 | 65.82 | 3.01 | 8.20 | 0.25 | 0.40 | 0.01 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 60 | 75.46 | 3.41 | 16.61 | 0.54 | 0.94 | 0.02 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 66 | 81.40 | 4.24 | 33.35 | 1.12 | 2.03 | 0.06 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 72 | 84.41 | 5.09 | 49.43 | 2.07 | 3.76 | 0.13 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 78 | 85.14 | 5.62 | 63.55 | 2.90 | 6.57 | 0.24 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 84 | 84.15 | 7.81 | 75.15 | 3.29 | 11.16 | 0.45 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 90 | 81.88 | 11.59 | 84.55 | 4.02 | 19.69 | 0.80 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 96 | 78.69 | 16.13 | 91.38 | 5.00 | 36.01 | 1.44 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 102 | 74.87 | 21.07 | 96.06 | 5.60 | 51.52 | 2.45 | 0.03 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 108 | 70.64 | 25.27 | 98.92 | 8.23 | 65.03 | 3.00 | 0.10 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 114 | 66.19 | 38.95 | 100.25 | 12.83 | 76.06 | 3.40 | 0.24 | 0.01 | 0.00 | 0.00 | 0.00 | 0.00 |
| 120 | 61.65 | 48.63 | 100.30 | 19.66 | 84.89 | 4.25 | 0.46 | 0.01 | 0.00 | 0.00 | 0.00 | 0.00 |
| 126 | 57.12 | 52.80 | 98.54 | 26.41 | 91.24 | 5.13 | 0.80 | 0.03 | 0.00 | 0.00 | 0.00 | 0.00 |
| 132 | 52.69 | 53.62 | 95.45 | 53.77 | 95.50 | 5.72 | 1.30 | 0.05 | 0.00 | 0.00 | 0.00 | 0.00 |
| 138 | 48.41 | 52.43 | 91.40 | 69.14 | 98.00 | 9.11 | 1.99 | 0.09 | 0.00 | 0.00 | 0.00 | 0.00 |
| 144 | 44.33 | 50.08 | 86.70 | 76.86 | 99.03 | 13.81 | 2.90 | 0.15 | 0.00 | 0.00 | 0.00 | 0.00 |
| 150 | 40.46 | 47.10 | 81.61 | 79.69 | 98.83 | 20.74 | 4.05 | 0.23 | 0.00 | 0.00 | 0.00 | 0.00 |
| 156 | 36.82 | 43.82 | 76.30 | 79.40 | 97.60 | 30.65 | 5.80 | 0.34 | 0.00 | 0.00 | 0.00 | 0.00 |
| 162 | 33.43 | 40.45 | 70.94 | 77.16 | 95.57 | 56.21 | 8.13 | 0.49 | 0.00 | 0.00 | 0.00 | 0.00 |
| 168 | 30.28 | 37.12 | 65.63 | 73.72 | 92.92 | 70.95 | 11.48 | 0.71 | 0.00 | 0.00 | 0.00 | 0.00 |
| 174 | 27.37 | 33.90 | 60.46 | 69.58 | 89.79 | 78.86 | 16.50 | 1.03 | 0.00 | 0.00 | 0.00 | 0.00 |
| 180 | 24.69 | 30.85 | 55.49 | 65.08 | 86.32 | 82.43 | 25.30 | 1.50 | 0.00 | 0.00 | 0.00 | 0.00 |
| 186 | 22.24 | 27.99 | 50.75 | 60.45 | 82.16 | 83.13 | 41.60 | 2.26 | 0.00 | 0.00 | 0.00 | 0.00 |
| 192 | 19.99 | 25.92 | 46.28 | 55.82 | 77.56 | 81.86 | 56.93 | 2.90 | 0.00 | 0.00 | 0.00 | 0.00 |
| 198 | 17.95 | 25.36 | 42.09 | 51.31 | 72.71 | 79.25 | 70.17 | 3.26 | 0.00 | 0.00 | 0.00 | 0.00 |
| 204 | 16.09 | 24.68 | 38.18 | 46.98 | 67.75 | 75.75 | 80.86 | 3.91 | 0.00 | 0.00 | 0.00 | 0.00 |

**Figure 6.20 D-Hour Stormwater Pond Routing Inflow and Outflow Hydrograph Ordinates for Storms with AEP of 10%**

### 6.2.11 24-hr Storm Pond Routing Results

The 24-hr Storm Pond Routing Results worksheet lists Runoff and Pond Routing Results for Seven 24-Hour Rainfall Events. The subject events have 100, 50, 20, 10, 4, 2, and 1 % AEP. Some of the listed data for each AEP storm include rainfall and runoff depths, magnitude and time of pond peak inflow and outflow, times to drain the first one-half inch, first inch and first two inches of inflow, and the maximum ponding depth. Data about the pond peak inflow and outflow, and maximum ponding depth, relate to watershed hydrology and pond routing results. The times to drain different depths also relate to pond routing but are primary parameters about pond water quality performance. There is a table that lists the inflow and outflow hydrograph ordinates and plots of the inflow and outflow hydrographs.

### 6.2.12 Pond Sediment Trap Efficiency

Sediment Basins are a Best Management Practice (BMP) used to control stormwater runoff from disturbed areas to restrict sediment and other pollutants from being discharged off-site. South Carolina regulations require that when stormwater drains to a single outlet from land disturbing activities of 10 acres or more, a sediment basin must be designed to meet a removal efficiency of 80% for suspended solids or 0.5 ml/l peak settable solids concentration, whichever is the more restrictive. The efficiency shall be calculated for disturbed conditions for the 10-year 24-hour design (rainfall) event.

This worksheet includes two methods for determining sediment pond trapping efficiency. One is based on the SCDHEC Sediment Basin Design Aids and the other on trapping efficiency curves developed at UofSC.

The Design Aids were developed by John Hayes and Billy Barfield (1994) during a study commissioned by SCDHEC. The objective was to develop a simple alternative to computer programs such as SEDIMOT (Warner et al, 1982). The intent was to develop a design method that gives reasonable assurance sediment ponds meet performance standards and to provide a straightforward and quick method that will benefit regulatory agencies, design engineers, and developers. The intended application was for single sites discharging to single ponds.

During the study, the performance of a wide range of pond and outflow structure combinations was based on results for almost 500,000 simulations. Pond trapping efficiencies were plotted versus the dimensionless pond peak outflow parameter that is the ratio of pond peak outflow divided by the product of pond surface area at the top of the riser and the settling velocity of the characteristic $D_{15}$ eroded particle. This diameter represents the point on the eroded particle size distribution curve where 15 percent of the particles (by weight) are equal to or smaller than this size.

The Design Aids were designed for soils classified as either coarse (sandy loam), medium (silt loam), or fine (clay loam). The Design Aids were developed for the following two separate conditions: (1) basins not located in low lying areas and/or not having a high-water table; and (2) basins located in low lying areas and/or having a high-water table.

The UofSC pond sediment trapping efficiency (TE) equation is the result of a study to develop trapping efficiency curves for the analysis of sediment ponds in South Carolina. An extensive pond performance database was simulated using a modified version of the SEDIMOT II computer program. Over 40 different soils were selected to characterize eight textural groups: clay loam (CL), silty clay loam (SiCL), sandy clay loam (SCL) loam (L), sand (S), loamy sand (LS), sandy loam (SL), and silty loam (SiL). Eroded grain size distributions were generated for all soils using equations developed for the CREAMS program. These data were input to the modified SEDIMOT II program and more than 500 simulations were performed for a range of watershed, storm, and pond characteristics. The simulated pond trapping efficiencies were correlated with various sediment, hydrograph, and pond parameters, and regression equations developed to predict trapping efficiency in terms of dimensionless parameters for pond retention storage, particle size gradation, pond overflow rate, and peak flow reduction (pond detention storage).

The worksheet includes equations developed by Bryan Smith during graduate research at UofSC to develop a User-Friendly Hydrologic Stormwater and Sediment Pond Computational Program. The equations for the Design Aids were determined with data for the curves, which were used in a regression analysis to provide an equation for each curve. For tidal soils, the equation is TE=107.89-7.77x with $r^2$=1.00, where TE is Trapping Efficiency and x is the log base 10 value of F* = ($Q_{po}/A_oV_{15}$) where $V_{15}$ is the settling velocity of the D15 particle. For other soil conditions found in the Piedmont, Sand Hills, Tidal, and Coastal Plain areas the equation is TE=75.91+13.64x-2.45$x^2$ with $r^2$=0.99.

The equation for UofSC trapping efficiency is TE=a+bS*+cD*+dQ* where a, b, c, and d are coefficients and $D^*=D_{85}/D_{15}$ characterizes eroded particle size distribution, $F^*=Q_{po}/A_oV_{15}$ is the dimensionless pond overflow rate equation, $q^*=Q_{po}/i_p$ characterizes pond detention routing effects, and $S^*=V_{ret}/V_{runoff}$ characterizes pond retention storage effects. The terms $Q_{po}$ and $i_p$ are pond peak outflow and inflow rates, and $V_{ret}$ and $V_{runoff}$ are pond retention and watershed runoff volumes. Computed values for these parameters are shown in a table at the bottom of Figure 6.21.

The results in Table 6.21 indicate the sediment pond satisfies the SCDHEC regulation that a sediment basin must be designed to meet a removal efficiency of 80% for the 10-year 24-hour rainfall event. Both the SCDHEC and UofSC trapping efficiency equations show results for the 24-hour rainfall greater than 80%. However, the SCDHEC results for 1, 2, 3, 6, and 12-hour storms are all less than 80% and the UofSC results for 3, 6, and 12-hour storms are less than 80%. The 6 and 12-hour events are critical duration storms and the 6-hour event is the critical duration pond design storm. These results are a red flag warning the proposed pond design is not satisfactory and should be upgraded.

**Pond Sediment Trapping Efficiency for D-Hour Storm Events**

Design Storm AEP = 10 %

| Soil Type = | TUSQUITEE-SUB | UofSC TE = a + bS* + cD* + dQ* | | | | | |
|---|---|---|---|---|---|---|---|
| Texture = | L | 24 hr | 12 hr | 6 hr | 3 hr | 2 hr | 1 hr |

| | Soil Texture | a | b | c | d | Trapping Efficiency | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | 24 hr | 12 hr | 6 hr | 3 hr | 2 hr | 1 hr | |
| | CL | 88.60 | 23.78 | -0.12 | -0.40 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | |
| | SiCL | 54.23 | 23.64 | 0.14 | -0.28 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | |
| | SCL | 89.56 | 13.93 | -0.08 | -0.70 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | |
| ==> | L | 88.53 | 19.99 | -0.11 | -0.74 | 80.27 | 79.32 | 79.40 | 79.77 | 80.12 | 81.51 | <== |
| | S | 98.75 | 2.56 | -0.16 | -0.70 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | |
| | LS | 94.63 | 6.22 | -0.04 | -2.15 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | |
| | SL | 91.84 | 11.52 | -0.08 | -1.29 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | |
| | SiL | 79.34 | 23.54 | -0.01 | -0.77 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | |

The **Red Arrows** in Columns D and P flag the soil texture and UofSC Trappng Efficiency results.

| Equation | | 24 hr | 12 hr | 6 hr | 3 hr | 2 hr | 1 hr |
|---|---|---|---|---|---|---|---|
| Coastal TE = 107.89 - 7.77F* | Coastal TE = | 69.17 | 67.47 | 67.27 | 67.43 | 67.57 | 68.91 |
| DHEC TE = 75.91 + 13.04F* - 2.45F*$^2$ | DHEC TE = | 80.05 | 77.45 | 77.13 | 77.37 | 77.60 | 79.66 |
| UofSC TE = a + bS* + cD* + dQ* | UofSC TE = | 80.27 | 79.32 | 79.40 | 79.77 | 80.12 | 81.51 |

**A result of ###### indicates non-convergence of the pond routing routine and is not a problem with the spreadsheet. User must change pond design.**

| | 24 hr | 12 hr | 6 hr | 3 hr | 2 hr | 1 hr |
|---|---|---|---|---|---|---|
| S* = | 0.106 | 0.089 | 0.095 | 0.111 | 0.119 | 0.153 |
| D* = | 70.31 | 70.31 | 70.31 | 70.31 | 70.31 | 70.31 |
| Q* = | 3.57 | 4.39 | 4.46 | 4.37 | 4.12 | 3.16 |
| F* = | 4.98 | 5.20 | 5.23 | 5.21 | 5.19 | 5.02 |
| q* = | 0.72 | 0.84 | 0.85 | 0.84 | 0.79 | 0.63 |

**Figure 6.21 Example Sediment Pond Trapping Efficiencies Computed with SDHEC and UofSC Equations**

### 6.2.13 USLE & MUSLE Results

USLE is the acronym for Universal Soil Loss Equation developed by USDA Agricultural Research Service (ARS) scientists. USLE has been the most widely accepted and utilized soil loss equation. USLE was developed to predict average annual soil loss caused by sheet and rill erosion. While it can estimate long-term annual soil loss and guide proper cropping and management and conservation practices, it cannot be applied to a specific year or a specific storm. USLE is mature technology and enhancements are limited by the simple equation structure.

The USLE for estimating average annual soil erosion is

$$A = RKLSCP \tag{6.1}$$

where

- A = average annual soil loss in tons per acre
- R = rainfall erosivity index
- K = soil erodibility factor
- LS = topographic factor - L is for slope length and S is for slope
- C = cropping or soil disturbance factor
- P = conservation practice factor

D-hour storm erosion is computed with the Modified Universal Soil Loss Equation (MUSLE). MUSLE was developed by replacing the rainfall energy factor in USLE with a runoff energy factor that is a function of the runoff volume and peak flow rate for individual storms. MUSLE has advantages over USLE that include application to individual storms and elimination of sediment delivery ratios because the runoff factor reflects energy used for sediment detachment and transport. MUSLE is more accurate because runoff accounts for greater variation in sediment yield than does rainfall.

MUSLE is given by the following equation:

$$S = 95\left(QQ_p\right)^{0.56} KLSCP \tag{6.2}$$

where S is the single storm event soil erosion in tons, Q is the event runoff volume in cubic feet, $Q_p$ is the peak runoff in cfs, and K, LS, C, P are USLE parameters identified above.

USLE has been tested and found to perform satisfactorily on grassland and mixed land use watersheds. As expected, the performance of MUSLE greatly depends on the accuracy of the hydrologic inputs.

The USLE and MUSLE Results worksheet shows the results of both methods as illustrated in Figure 6.22. Note the event gross erosion for the 2, 3, 6, and 12-hour events exceeds the erosion for a 24-hour event.

## USLE and MUSLE RESULTS

**USLE: A = R *K LS C P***

| Predicted Soil Loss | | |
|---|---|---|
| A = | 4.9 | tons/acre/year |
| **Gross Erosion** | | |
| A*Area = | 492.9 | tons/year |

| Factors: | |
|---|---|
| R = | 275 |
| K = | 0.20 |
| LS = | 0.25 |
| C = | 0.40 |
| P = | 0.90 |

**MUSLE: $S = 95(Q^{*}Qp)^{0.56}$ K LS C P**

Erosion for Single Event D-Hour Storms

AEP = 10 %

| | Storm Duration (hours) | Runoff Volume (ac-ft) | Peak Runoff (cfs) | Gross Erosion (tons) | |
|---|---|---|---|---|---|
| | 1 | 13.65 | 85.14 | 88.7 | |
| | 2 | 17.54 | 100.30 | 111.8 | |
| | 3 | 18.81 | 99.03 | 115.5 | |
| | 6 | 21.92 | 102.01 | 127.9 | |
| ==> | 12 | 23.51 | 97.07 | 129.4 | <== |
| | 24 | 19.67 | 69.22 | 96.9 | |

| | |
|---|---|
| Soil Type: | TUSQUITEE-SUB |
| Texture: | L |
| Slope (%): | 2 |
| Length (feet): | 210 |
| Cover: | Jute mesh |
| Practice: | Track-walked up and down slope |

The **Red Arrows** flag the critical storm duration with the maximum amount of erosion.

**Figure 6.22 USLE and MUSLE Results for 100 Acre Site Near Eutawville, SC with Tusquitee Topsoil**

#### 6.2.13.1 R Factor

The R factor is based on the erosive power of rainfall events common to the area. Sometimes called the "erosive index," R values were developed using weather records for maximum rainfall intensity and kinetic energy. R is a statistic calculated from the annual summation of rainfall energy in every storm (correlates with raindrop size) times its maximum 30-minute intensity. As expected, it varies geographically.

The annual R values for South Carolina are included in the SCDHEC document for Design Rainfall Data. Values range from 250 to 400 and are provided for each county in the state. Counties, such as Spartanburg, are divided into different regions, based largely on differences in hydrology, and have different 24-hour rainfall totals and R factors.

The user selects the R factor by clicking on the South Carolina county name in the list on this worksheet as shown in Figure 6.23. The R factor is used in the USLE equation to compute annual gross erosion.

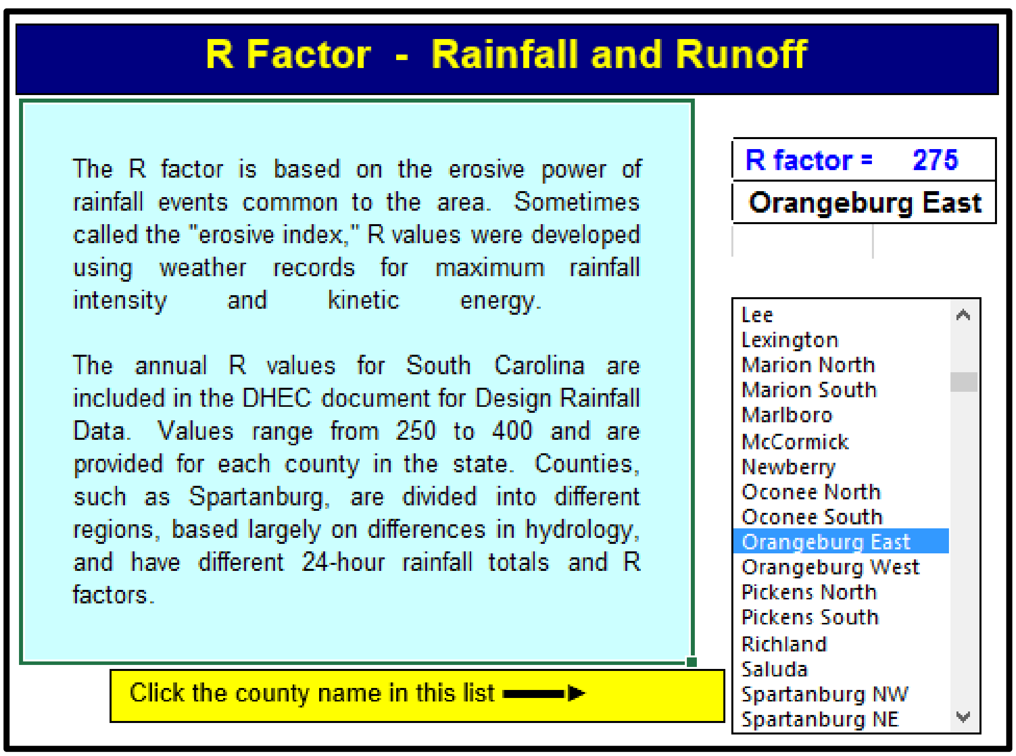



**Figure 6.23 Worksheet to Select South Carolina County Where the Study Site is Located and Thereby Enter the Rainfall Factor**

### 6.2.13.2 K Factor

The soil erodibility factor, K, indicates the susceptibility of a soil to erode. Two soil properties, infiltration capacity and structural stability, exert the greatest influence on erosion. These features, in turn, are related to a soil's organic matter and clay content, clay type, depth to an impervious layer, and tendency to crust. Selecting one of the soils in South Carolina assigns the K factor. Soils that do not erode readily have K values less than 0.2, while values greater than 0.3 indicate high erodibility. This factor quantifies the cohesive or bonding character of a soil type and its resistance to dislodging and transport due to raindrop impact and overland flow.

The user selects the K factor by clicking on the soil name in the list on this worksheet as shown in Figure 6.24. The K factor is used in the USLE equation to compute annual gross erosion.

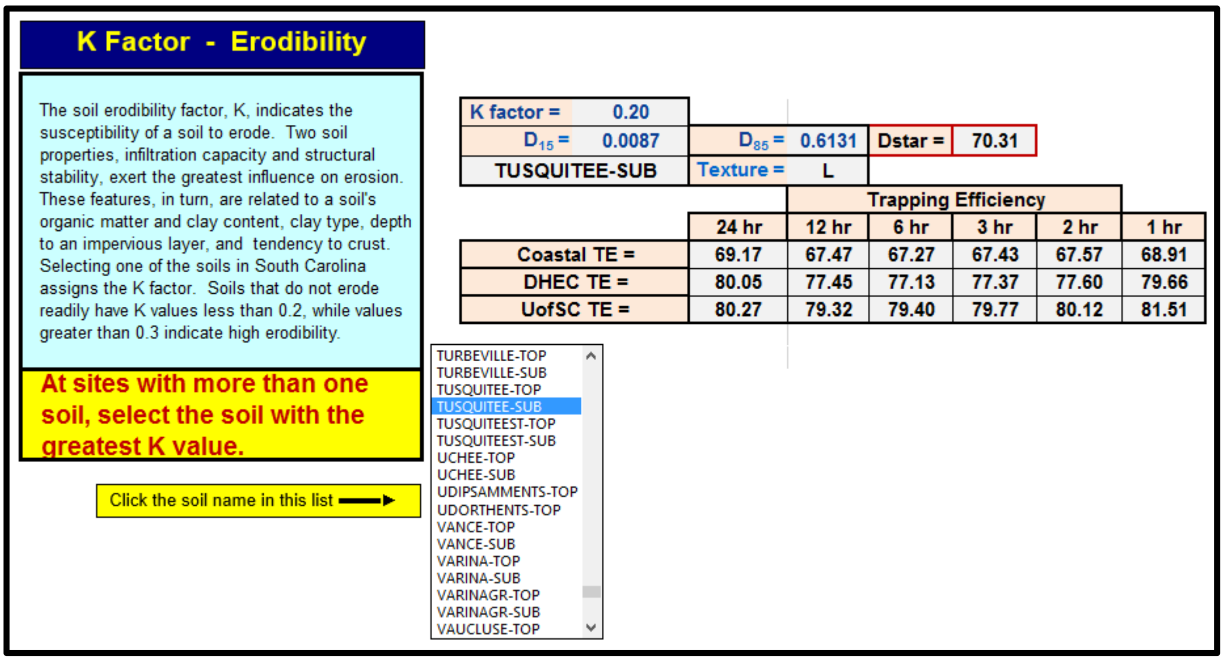



**Figure 6.24 Worksheet to Select Site Soil and Thereby Enter the Soil Erodibility Factor**

### 6.2.13.3 LS Factor

The effect of slope length and steepness on water erosion appears in the LS, or topography, factor. Erosion increases when either the length of the slope increases, the steepness of the slope increases, or both. Steeper slopes produce higher overland flow velocities. Longer slopes accumulate runoff from larger areas and result in higher flow velocities. Thus, both result in increased erosion potential. For convenience L and S are lumped into a single term. The LS factor value is computed with an empirical equation developed with data measured at experimental erosion plots.

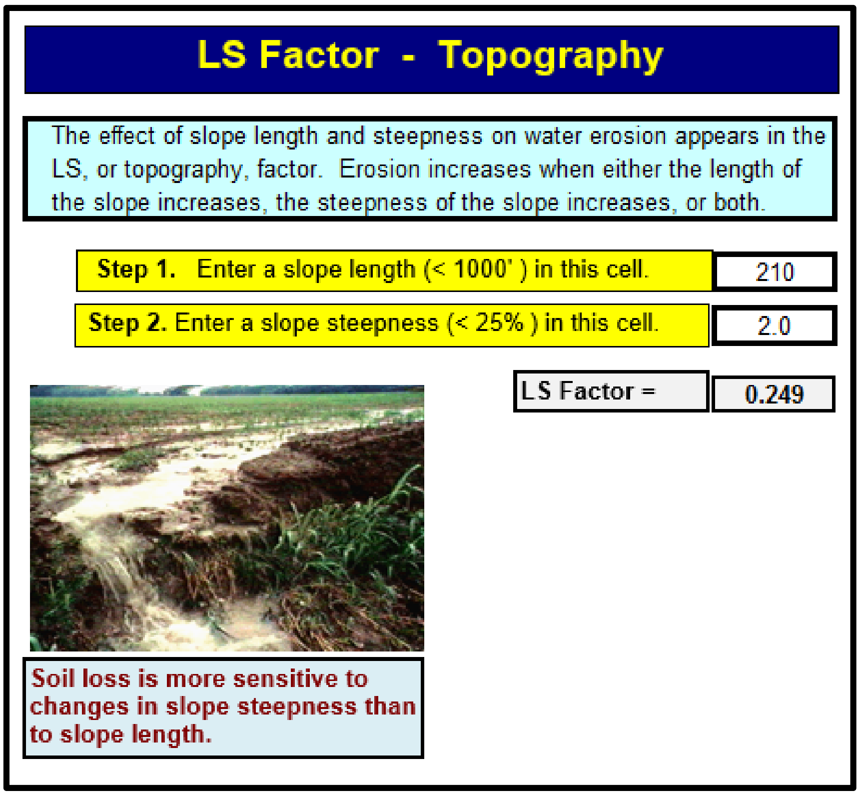



**Figure 6.25 Worksheet to Input Erodible Site Length and Slope**

#### 6.2.13.4 C Factor

The cover-management factor C represents the effect of plants, ground cover, soil biomass, and soil disturbing activities on erosion. This factor is the ratio of soil loss from land cropped under specified conditions to corresponding loss under tilled, continuous fallow conditions. The most computationally complicated of USLE factors, it incorporates effects of tillage management (dates and types), crops, seasonal erosivity index distribution, cropping history (rotation), and crop yield level (organic matter production potential).

Surface Covers are included in a list shown on this worksheet. The user selects the desired cover by clicking on the cover name in the list on this worksheet as shown in Figure 6.26. The C factor is used in the USLE equation to compute annual gross erosion.

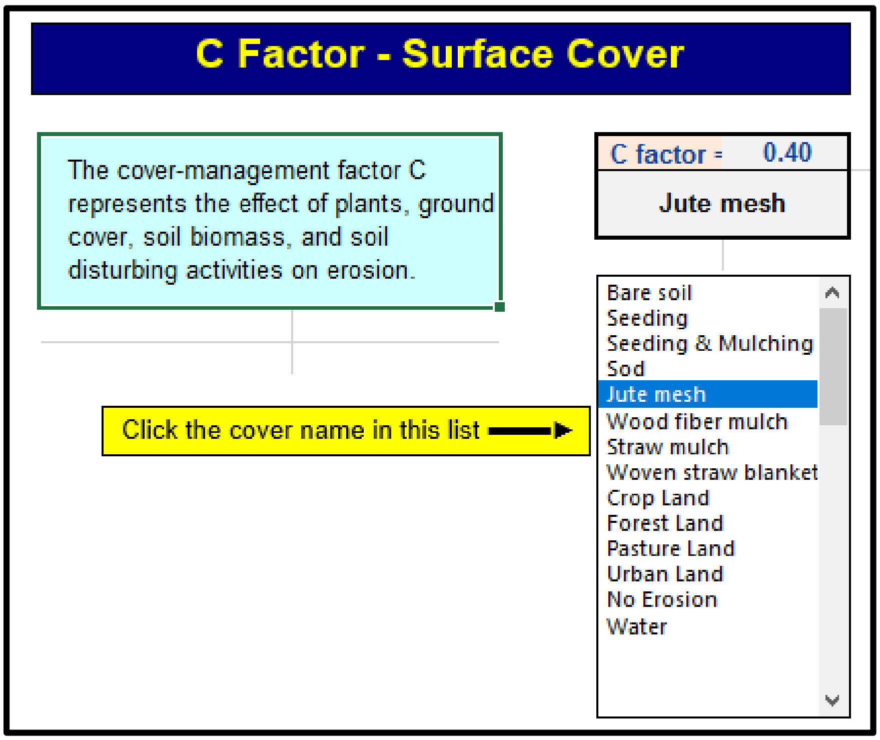



**Figure 6.26 Worksheet to Select Surface Cover Management Factor**

### 6.2.13.5 P Factor

The erosion control management factor, P, represents what people do to slow down the rate of erosion using control practices such as terraces, silt fences, check dams, and pasture. These and other practices are included in a list shown on this worksheet. The user selects the desired erosion control practice by clicking on the practice name in the list on this worksheet. As seen in Figure 6.27, the selected practice and its P factor value are displayed on the worksheet. The P factor is used in the USLE equation to compute annual gross erosion.

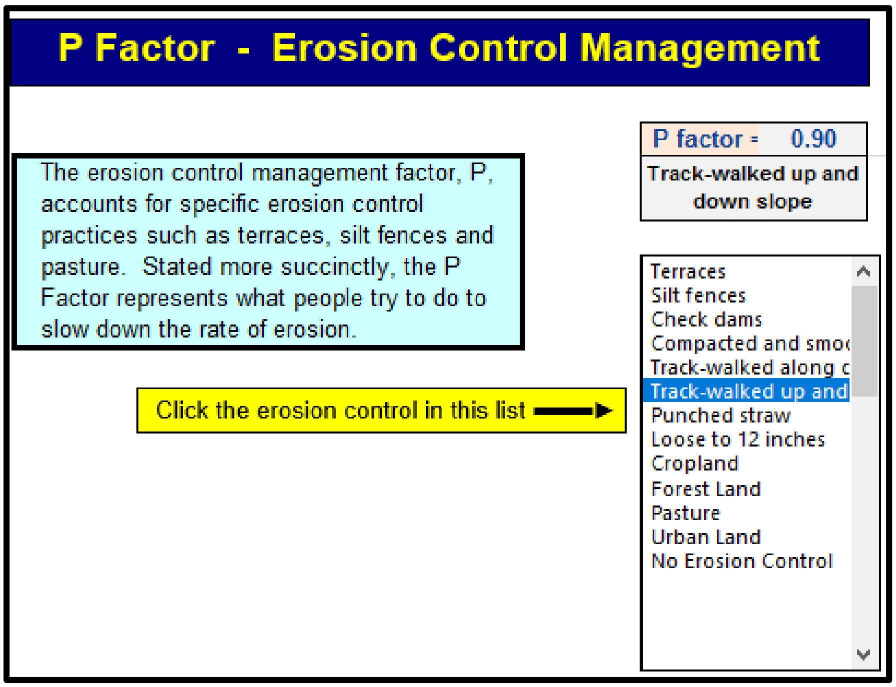



**Figure 6.27 Worksheet to Select Erosion Control Management Factor**

# CHAPTER 7: SUMMARY

As noted in Chapter 1, the purpose of this manual is to present and illustrate the South Carolina Synthetic Unit Hydrograph Method (identified throughout as the "SC UH Method"). Chapters 1 through 5 provide discussions of the origin and evolution of the method, an explanation of the components, notably the unit hydrograph peak rate factor (PRF), discussion of stormwater management studies that verified the method, and example applications. Chapter 6 discusses the spreadsheet developed to facilitate user application of the SC UH Method.

Chapter 3 includes discussion of two new concepts and procedures that are part of the SC UH Method and are incorporated into the Spreadsheet. One is modifying the NRCS Curve Number for rainfall durations less than 24-hours and the other is identifying critical storm durations that produce the maximum peak flow and/or maximum runoff volume. A significant outcome of CN modification is most critical durations are not 24-hours, particularly for peak flow prediction, which gives reason and justification to challenge regulations that prescribe a single design storm duration that is not a critical duration and could lead to an unsafe design.

The SC UH Method uses the two-parameter gamma distribution to describe and enumerate the unit hydrograph. A key parameter is PRF that relates to UH shape and proportion of runoff volume under the rising limb. PRF is defined as an index of watershed hydraulic efficiency as illustrated by the fact unit hydrographs with high PRF values have greater volume under the rising limb than unit hydrographs with low PRF values. One positive about the SC UH Method is that its application was demonstrated and verified during stormwater management studies at multiple locations in South Carolina. A significant outcome of those studies, and one that upholds the purpose and intent of the SC UH Method, is each watershed has its own unique PRF and thereby, its own unique UH. This led to the development of a table of land use specific PRF values.

A very important fact learned during the development of the SC UH Method is that watersheds are like people. As the author tells students in his Engineering Hydrology class: watersheds are just like each of us. Every person has a different height, weight, and complexion. Every watershed has a different area, hydraulic length, slope, land use, and soils. People have different personalities and so do watersheds (i.e., different watersheds have different CNs, PRFs, Tc) which are watershed personality parameters. The SC UH method has been tested and proven at multiple watersheds.

# BIBLIOGRAPHY

Blandford, G.E., Peters, N., and Meadows, M.E. (1983). "Development of Models for Simulating Stormwater Runoff from Surface Coal Mined Lands: Finite Element Simulation of Overland Flow," Report No. G5115213, Vol. II, U.S. Dept. of the Interior, Office of Surface Mining.

Chow, V.T. (1959). Open-channel hydraulics: New York, McGraw-Hill, 680 p.

Hawkins, R. H., and T.J. Ward. (2012) “Expected Value of Event Runoff with Curve Number Theory”. Proceedings (CD) ASCE Environmental and World Water Congress, Albuquerque NM, May 20-24, 2012.

Hayes, John C. and Barfield, Billy J. (1994). “Engineering Aids and Design Guidelines for Control of Sediment in South Carolina,” Research Report Submitted to South Carolina Department of Health and Environmental Control, Columbia, SC.

Meadows, Michael (2016) "Adjusting NRCS Curve Number for Rainfall Durations Less Than 24 Hours," Journal of South Carolina Water Resources: Vol. 3: Issue 1, Article 6.

Meadows, M.E. (1986). "Drain:Edge - Stormwater Simulation and Design Model," C D Software, Inc., Columbia, SC.

Meadows, M.E. (1987). "Results of a Regional Synthetic Unit Hydrograph Study," Proc., ASCE Engineering Hydrology Symposium, Williamsburg, VA, pp. 622-626.

Meadows, M.E. (2011). “Assessment of Flooding at the Aiken County Horse Farms and Applicability of NRCS Curve Numbers,” Report Submitted to the City of Aiken Public Works and USDA Natural Resources Conservation Service.

Meadows, M.E. (2016). "Adjusting NRCS Curve Number for Rainfall Durations Less Than 24 Hours," Journal of South Carolina Water Resources: Vol. 3: Issue 1, Article 6.

Meadows, M.E. and G.E. Blandford (1983). "Improved Methods and Guidelines for Modeling Stormwater Runoff from Surface Mined Lands," Research Report No. 147, Water Resources Institute, Univ. of Kentucky, Lexington, KY.

Meadows, M.E. and D.S. Eliatamby (1990). "Stormwater Management Study for Rawls Creek, Phase I: Model Development and Preliminary Results," Interim Project Report, Lexington County Dept. of Planning and Development, Lexington, SC.

Meadows, M.E. and K.M. Howard (1981). "Evaluation of Synthetic Hydrograph Techniques for Application at U.S. Army Military Installations," U.S. Army CERL, Champaign, IL.

Meadows, M.E. and E.W. Ramsey (1991). "South Carolina Regional Synthetic Unit Hydrograph Study: Methodology and Results," Project Report, Volume II, U.S. Geological Survey, Columbia, SC

Meadows, M E., Morris, K.B., and W.E. Spearman (1991). "Stormwater Management Study for the City of Aiken: Wise Hollow Drainage Basin," Project Report, Department of Engineering and Public Works, City of Aiken, SC.

Meadows, M.E., Morris, K.B., and W.E. Spearman (1992). "Stormwater Management Study for the City of Aiken: Sand River Drainage Basin," Project Report, Department of Engineering and Public Works, City of Aiken, SC.

Meadows, M.E., Morris, K.B., and W.E. Spearman (1992). "Improved Runoff Estimation Methods and Single Outlet Detention Pond Performance Curves Applicable to Aiken, SC," Supplemental Report, Department of Engineering and Public Works, City of Aiken, SC.

Meadows, M.E., Morris, K.B., and W.E. Spearman (1992). "Improved Runoff Estimation Methods and Single Outlet Detention Pond Performance Curves Applicable to Urban Watersheds in the Midlands of South Carolina," Supplemental Report: Kinley Creek Stormwater Study, Lexington County Dept. of Planning and Development, Lexington, SC.

Meadows, M.E., Morris, K.B., and W.E. Spearman (1992). "Stormwater Management Study for Koon Branch, Phase II: Strategies," Project Report, Lexington County Dept. of Planning and Development, Lexington, SC.

Meadows, M.E., Morris, K.B., and W.E. Spearman (1992). "Stormwater Management Study for Kinley Creek," Project Report, Lexington County Dept. of Planning and Development, Lexington, SC.

Meadows, M.E., Morris, K.B., and W.E. Spearman, (1992). "Stormwater Management Study for Rawls Creek," Project Report, Lexington County Dept. of Planning and Development, Lexington, SC.

Meadows, M.E. and F.M. Nevils, Jr. (1987). "Stormwater Management Study for Senn Branch Watershed," Project Report, Lexington County Dept. of Public Works, Lexington, SC.

Meadows, M.E. and E.W. Ramsey, (1989). "OPT-UH - Unit Hydrograph Optimization Program," Dept. of Civil Engineering, Univ. of South Carolina.

Meadows, M.E. and W.E. Spearman (1988). "Impact of Stormwater Detention Ponds on Downstream Flooding," Project Report, Richland County Public Works, Columbia, SC.

Meadows, M.E., Howard, K.M., and A.L. Chestnut (1983). "Development of Models for Simulating Stormwater Runoff from Surface Coal Mined Lands: Unit Hydrograph Models," Report No. G5115213, Vol. I, U.S. Dept. of the Interior, Office of Surface Mining.

Mockus, V. (1961). Watershed lag. U.S. Dept. of Agriculture, Soil Conservation Service, ES–1015, Washington, DC.

Nevils, F.M. and M.E. Meadows, (1984). "Preliminary Study to Develop a Method for Applying the Gamma Distribution Unit Hydrograph Model to Ungaged Watersheds," Dept. of Civil

Engineering, Univ. of South Carolina, Columbia, SC.

NRCS (Natural Resources Conservation Service). (2010). “Chapter 15 – Time of Concentration,” Part 630 Hydrology, National Engineering Handbook, Washington, DC.

Overton, D.E. and Meadows, M.E. (1976). Stormwater Modeling, Academic Press, New York, 358 p.

Rodriques-Iturbe, I. and J.B. Valdes, (1979). “The Geomorphic Structure of the Hydrologic Response,” Water Resources Research, Vol. 18, No. 4.

Rosso, R. (1984). “Nash Model Relation to Horton Order Ratios,” Water Resources Research, Vol. 20, No. 7.

Sauer, V.B, Thomas, W.O., Stricker, V.A., and K.V. Wilson, (1981). “Magnitude and Frequency of Floods in the United States,” U.S. Geological Society, Reston, VA.

Sauer, V.B., W.O. Thomas, V.A. Stricker, and K. Wilson (1983). Flood Characteristics of Urban Watersheds in the United States. Water Supply Paper 2207, U.S. Geological Survey, Reston, VA.

Smith, W.B. (1996). “HYSSOP: A User-Friendly Hydrologic Stormwater and Sediment Pond Computational Program,” M.S. Thesis, Department of Civil and Environmental Engineering, University of South Carolina, Columbia, SC.

Warner, R. C., Wilson, B. N., Barfield, B. J., Lostdon, D.S., and Nebgen, P.J. (1982). A Hydrology and Sedimentology Watershed Model. Part II: Users’ Manual, University of Kentucky, Department of Agricultural Engineering, Special Publication, Lexington, USA, 460 pp.

Williams, J.R. and R.W. Hann, (1973). "HYMO: Problem-Oriented Computer Language for Hydrologic Modeling User’s Manual," U.S. Department of Agriculture, Agricultural Research Service, ARS-S-9.

# APPENDICES

# APPENDIX A. TERMINOLOGY, DEFINITIONS AND ABBREVIATIONS

## A.1 Terminology and Definitions

| | |
|---|---|
| Watershed - | The surface area that contributes runoff to a common point (known as the watershed outlet or outfall). |
| Precipitation - | Water in solid or liquid form falling from the atmosphere to earth. Forms of precipitation include snow (crystalline), sleet (grains of ice), hail (balls of ice), and rain (liquid form). Types of precipitation include convective, orographic, frontal, and cyclonic. |
| Rainfall Depth - | The total amount of rainfall that accumulates at-a-point (i.e., rain gage) during a storm event. |
| Rainfall Intensity - | The rate (velocity) of rainfall. |
| Interception - | Rainfall (precipitation) captured and stored on leaves, rooftops, etc. |
| Depression - Storage | Rain that becomes trapped in depressions (holes) in the land surface and subsequently evaporates or infiltrates. |
| Evaporation - | A diffusive process whereby water undergoes a change in state from liquid to vapor and rises from the land surface to the atmosphere. Evaporation occurs from water transpired by plants (evapotranspiration or, more simply, ET), bare soil, and open water bodies. |
| Infiltration - | Water movement across the air-soil interface (ground surface) into the underlying soil. Represents the major loss or abstraction of rainfall. |
| Runoff - | Runoff is rainfall in excess of losses that retain the water within the watershed. Runoff is water that flows overland and through shallow and/or deep groundwater routes to exit the watershed at its outlet. |
| Direct Runoff - | Runoff that represents an immediate response of the watershed to a rainfall event. |
| Runoff Philosophies - | Different views about the dominant processes controlling direct runoff generation: *Hortonian runoff* is runoff due to rainfall in excess of infiltration, i.e., surface, or overland flow. Methods based on this philosophy generally assume the entire watershed contributes runoff. Hortonian runoff occurs from impervious and clayey topsoil watersheds. *Saturation overland flow* occurs where the soil saturated hydraulic conductivity exceeds the rainfall intensity, depth to the water table is shallow, and after a period of rain, the groundwater table mounds to the air-soil interface. Continuing rain occurs on saturated soil, cannot infiltrate into the soil, and goes directly to surface runoff. Saturation |

overland flow differs from Hortonian overland flow in that with Hortonian overland flow the soil is saturated from above by infiltration, while in saturation overland flow it is saturated from below by subsurface flow. Saturation overland flow occurs at the bottom of hillslopes, near streams, and in areas with shallow groundwater, such as along the coast. *Variable source area or partial area* denotes the area of a watershed contributing flow to the stream at any time. This philosophy recognizes that not all portions of a watershed contribute subsurface flow or saturation overland flow to a stream during a storm. The contributing area expands during a storm and contracts between storms, or during lulls within a storm. This also is known as dynamic watershed response.

Rainfall Excess - Rainfall that becomes direct storm runoff.

Stormwater - Runoff generated by a precipitation (rainfall) event.

Hydrograph - A hydrograph is a time series plot of discharge passing a point along the drainage network.

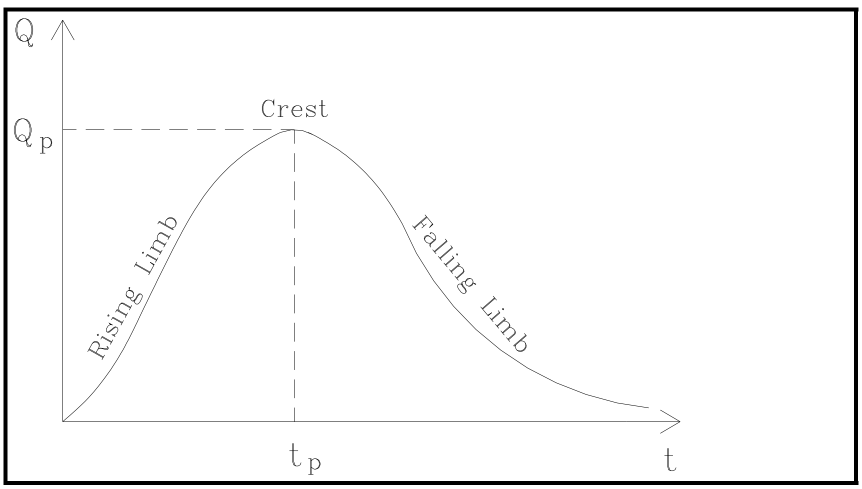



Figure A.1. Stormwater Hydrograph

Streamflow - Flow in a stream. The streamflow hydrograph is composed of the stormwater hydrograph and baseflow (recharge from groundwater).

Runoff Volume - The volume of water, typically measured in watershed inches or millimeters, that occurs as direct runoff.

Runoff Peak - The maximum, or peak, flowrate of a stormwater hydrograph. This is the value typically selected as the design flowrate for sizing open channels, culverts, and storm sewers.

Design Flow - The flowrate that we use in hydraulic calculations to size sewers, channels, etc.

Return Period - The *average* number of years between events of magnitude equal to or greater than a specified amount.

Risk - The probability of an event (rainfall depth or flood discharge) being equaled or exceeded in a specified number of years.

NRCS Methods - Procedures developed by the U.S. Department of Agriculture, Natural Resources Conservation Service (formerly Soil Conservation Service or SCS), to estimate volumes and rates of runoff.

Unit Hydrograph - Runoff hydrograph resulting from 1-inch of rainfall excess occurring uniformly over a watershed in a specified duration (D-hours). It represents the watershed hydrologic response function and is used mathematically to transform the estimate of event rainfall excess into the stormflow hydrograph. The mathematical procedure is convolution.

Time of Concentration ($t_c$) - A fundamental measure of the timing of watershed response to rainfall input. A historical definition is that it is the time required for a drop of water to travel from the watershed boundary to its outlet. For the condition of steady excess intensity, it is the time required to reach equilibrium conditions, at which time the runoff rate will be a maximum (peak) for the given excess intensity.

Hydrologic Soil Group (HSG) - NRCS method for classifying soils according to their runoff potential. There are four hydrologic soil groups identified by the letters A, B, C, and D. HSG-A soils are typified by sands, which have high infiltration rates and low runoff potential. HSG-D soils are typified by clays, which have low infiltration rates and high runoff potential.

NRCS Curve Number (CN) - An index of watershed runoff potential. CN values range between 0 (no runoff) and 100 (complete runoff). CN values are based on empirical data collected at experimental watersheds and are found in published tables as a function of soil type (HSG), land use, and surface cover conditions. **CN is not the percentage of rain that goes to runoff**.

NRCS CN Runoff Model - An equation to compute the runoff volume (expressed in watershed inches or mm) for a given rainfall depth.

Rational Method - A widely used method to estimate the peak runoff from a small watershed. The rational method is a rainfall intensity-based method.

Peak Rate Factor - Peak Rate Factor (PRF) is an index of watershed drainage efficiency. Well-drained watersheds have high PRF values (up to 550) while poorly drained watersheds have low PRF values (as low as 180). PRF is a parameter used to compute the amplitude (peak) of a unit hydrograph. High PRF means the unit hydrograph has a higher peak, shorter recession limb, and more of the runoff volume occurring under the rising limb than does a unit hydrograph with a lower PRF value. The standard NRCS unit hydrograph has PRF=484 (English units).

### A.2 Abbreviations

| | |
|---|---|
| A | Watershed Area |
| AEP | Annual Exceedance Probability |
| ASCII | American Standard Code for Information Interchange |
| BDF | Basin Development Factor |
| BMP | Best Management Practice |
| CN | Curve Number |
| DTM | Double Triangle |
| EFH-2 | NRCS Engineering Field Handbook Chapter 2 |
| FHWA | Federal Highway Administration |
| HSG | Hydrologic Soil Group |
| HYMO | USDA-ARS Hydrology Model |
| $I_a$ | Initial Abstractions |
| MUSLE | Modified Universal Soil Loss Equation |
| NEH | National Engineering Handbook |
| NEH-4 | National Engineering Handbook Section 4: Hydrology |
| NOAA | National Oceanic Atmospheric Administration |
| NOAA B | NOAA Type B Rainfall Distribution |
| NRCS | Natural Resources Conservation Service |
| NRCS II | NRCS Type II Rainfall Distribution |
| $P/P_{24}$ | Dimensionless Rainfall Depth During 24-Hour Storm |
| PFDS | Precipitation Frequency Data Server |
| PRF | Peak Rate Factor |
| Q | Volumetric Flow Rate |
| $Q_{CN}$ | NRCS Curve Number Runoff Depth |
| $Q_p$ | Peak Flow |
| RP | Return Period |
| S | Watershed Retention |

| | |
|---|---|
| SC UH | South Carolina Synthetic Unit Hydrograph |
| SCDOT | South Carolina Department of Transportation |
| SCS | Soil Conservation Service |
| Tc | Time of Concentration |
| $T_l$ | Lag Time |
| Tp | Time to Peak |
| TP-40 | Technical Paper 40 |
| TR-55 | Technical Release 55 |
| TVA | Tennessee Valley Authority |
| UH | Unit Hydrograph |
| UH PRF | Unit Hydrograph Peak Rate Factor |
| UofSC | University of South Carolina |
| URF | Unit Response Function |
| USDA | United States Department of Agriculture |
| USDA-ARS | United States Department of Agriculture-Agricultural Research Service |
| USGS | United States Geological Survey |
| USLE | Universal Soil Loss Equation |

## APPENDIX B: 24-HOUR RAINFALL DISTRIBUTIONS APPLICABLE IN SOUTH CAROLINA

| Time (min) | 24 Hour Rainfall Distributions | | | | | |
|---|---|---|---|---|---|---|
| | Type II | Type III | NOAA A | NOAA B | NOAA C | NOAA D |
| 0 | 0.0000 | 0.0000 | 0.0000 | 0.0000 | 0.0000 | 0.0000 |
| 6 | 0.0010 | 0.0010 | 0.0008 | 0.0010 | 0.0013 | 0.0011 |
| 12 | 0.0020 | 0.0020 | 0.0014 | 0.0018 | 0.0023 | 0.0022 |
| 18 | 0.0031 | 0.0030 | 0.0021 | 0.0027 | 0.0034 | 0.0033 |
| 24 | 0.0041 | 0.0040 | 0.0027 | 0.0035 | 0.0044 | 0.0045 |
| 30 | 0.0051 | 0.0050 | 0.0034 | 0.0043 | 0.0055 | 0.0056 |
| 36 | 0.0062 | 0.0060 | 0.0040 | 0.0052 | 0.0065 | 0.0067 |
| 42 | 0.0073 | 0.0070 | 0.0047 | 0.0061 | 0.0076 | 0.0079 |
| 48 | 0.0083 | 0.0080 | 0.0054 | 0.0069 | 0.0087 | 0.0091 |
| 54 | 0.0094 | 0.0090 | 0.0061 | 0.0078 | 0.0098 | 0.0103 |
| 60 | 0.0105 | 0.0100 | 0.0068 | 0.0087 | 0.0109 | 0.0115 |
| 66 | 0.0116 | 0.0110 | 0.0075 | 0.0096 | 0.0121 | 0.0127 |
| ]72 | 0.0127 | 0.0120 | 0.0082 | 0.0105 | 0.0132 | 0.0139 |
| 78 | 0.0139 | 0.0130 | 0.0090 | 0.0114 | 0.0143 | 0.0151 |
| 84 | 0.0150 | 0.0140 | 0.0097 | 0.0124 | 0.0155 | 0.0163 |
| 90 | 0.0161 | 0.0150 | 0.0105 | 0.0133 | 0.0167 | 0.0176 |
| 96 | 0.0173 | 0.0160 | 0.0112 | 0.0143 | 0.0178 | 0.0189 |
| 102 | 0.0185 | 0.0170 | 0.0120 | 0.0152 | 0.0190 | 0.0201 |
| 108 | 0.0196 | 0.0180 | 0.0128 | 0.0162 | 0.0202 | 0.0214 |
| 114 | 0.0208 | 0.0190 | 0.0136 | 0.0172 | 0.0214 | 0.0227 |
| 120 | 0.0220 | 0.0200 | 0.0144 | 0.0182 | 0.0226 | 0.0240 |
| 126 | 0.0232 | 0.0210 | 0.0152 | 0.0192 | 0.0238 | 0.0253 |
| 132 | 0.0244 | 0.0220 | 0.0160 | 0.0202 | 0.0251 | 0.0267 |
| 138 | 0.0257 | 0.0231 | 0.0168 | 0.0212 | 0.0263 | 0.0280 |
| 144 | 0.0269 | 0.0241 | 0.0176 | 0.0222 | 0.0276 | 0.0293 |
| 150 | 0.0281 | 0.0252 | 0.0185 | 0.0233 | 0.0288 | 0.0307 |
| 156 | 0.0294 | 0.0263 | 0.0193 | 0.0243 | 0.0301 | 0.0321 |
| 162 | 0.0307 | 0.0274 | 0.0202 | 0.0254 | 0.0314 | 0.0335 |
| 168 | 0.0319 | 0.0285 | 0.0211 | 0.0264 | 0.0327 | 0.0349 |
| 174 | 0.0332 | 0.0296 | 0.0219 | 0.0275 | 0.0340 | 0.0363 |
| 180 | 0.0345 | 0.0308 | 0.0228 | 0.0286 | 0.0353 | 0.0377 |
| 186 | 0.0358 | 0.0319 | 0.0237 | 0.0297 | 0.0366 | 0.0391 |
| 192 | 0.0371 | 0.0331 | 0.0246 | 0.0308 | 0.0379 | 0.0405 |
| 198 | 0.0385 | 0.0343 | 0.0256 | 0.0319 | 0.0393 | 0.0420 |
| 204 | 0.0398 | 0.0355 | 0.0265 | 0.0331 | 0.0406 | 0.0435 |
| 210 | 0.0411 | 0.0367 | 0.0274 | 0.0342 | 0.0420 | 0.0449 |
| 216 | 0.0425 | 0.0379 | 0.0284 | 0.0353 | 0.0434 | 0.0464 |
| 222 | 0.0439 | 0.0392 | 0.0293 | 0.0365 | 0.0447 | 0.0479 |
| 228 | 0.0452 | 0.0404 | 0.0303 | 0.0377 | 0.0461 | 0.0494 |
| 234 | 0.0466 | 0.0417 | 0.0313 | 0.0388 | 0.0475 | 0.0509 |
| 240 | 0.0480 | 0.0430 | 0.0322 | 0.0400 | 0.0489 | 0.0525 |

| Time (min) | 24 Hour Rainfall Distributions | | | | | |
|---|---|---|---|---|---|---|
| | Type II | Type III | NOAA A | NOAA B | NOAA C | NOAA D |
| 246 | 0.0494 | 0.0443 | 0.0332 | 0.0412 | 0.0504 | 0.0540 |
| 252 | 0.0508 | 0.0456 | 0.0342 | 0.0424 | 0.0518 | 0.0556 |
| 258 | 0.0523 | 0.0470 | 0.0352 | 0.0436 | 0.0532 | 0.0571 |
| 264 | 0.0538 | 0.0483 | 0.0363 | 0.0449 | 0.0547 | 0.0587 |
| 270 | 0.0553 | 0.0497 | 0.0373 | 0.0461 | 0.0562 | 0.0603 |
| 276 | 0.0568 | 0.0511 | 0.0383 | 0.0474 | 0.0576 | 0.0619 |
| 282 | 0.0583 | 0.0525 | 0.0394 | 0.0486 | 0.0591 | 0.0635 |
| 288 | 0.0598 | 0.0539 | 0.0404 | 0.0499 | 0.0606 | 0.0651 |
| 294 | 0.0614 | 0.0553 | 0.0415 | 0.0511 | 0.0621 | 0.0667 |
| 300 | 0.0630 | 0.0568 | 0.0426 | 0.0524 | 0.0636 | 0.0684 |
| 306 | 0.0646 | 0.0582 | 0.0436 | 0.0537 | 0.0651 | 0.0700 |
| 312 | 0.0662 | 0.0597 | 0.0447 | 0.0550 | 0.0667 | 0.0717 |
| 318 | 0.0679 | 0.0612 | 0.0458 | 0.0563 | 0.0682 | 0.0734 |
| 324 | 0.0696 | 0.0627 | 0.0469 | 0.0577 | 0.0697 | 0.0750 |
| 330 | 0.0713 | 0.0642 | 0.0481 | 0.0590 | 0.0713 | 0.0767 |
| 336 | 0.0730 | 0.0657 | 0.0492 | 0.0603 | 0.0729 | 0.0784 |
| 342 | 0.0747 | 0.0673 | 0.0503 | 0.0617 | 0.0745 | 0.0802 |
| 348 | 0.0764 | 0.0688 | 0.0515 | 0.0630 | 0.0760 | 0.0819 |
| 354 | 0.0782 | 0.0704 | 0.0526 | 0.0644 | 0.0776 | 0.0836 |
| 360 | 0.0800 | 0.0720 | 0.0538 | 0.0658 | 0.0793 | 0.0854 |
| 366 | 0.0818 | 0.0736 | 0.0550 | 0.0672 | 0.0809 | 0.0872 |
| 372 | 0.0836 | 0.0753 | 0.0562 | 0.0687 | 0.0826 | 0.0890 |
| 378 | 0.0855 | 0.0770 | 0.0575 | 0.0702 | 0.0843 | 0.0909 |
| 384 | 0.0874 | 0.0788 | 0.0588 | 0.0717 | 0.0861 | 0.0928 |
| 390 | 0.0893 | 0.0806 | 0.0602 | 0.0733 | 0.0879 | 0.0948 |
| 396 | 0.0912 | 0.0825 | 0.0615 | 0.0749 | 0.0898 | 0.0968 |
| 402 | 0.0931 | 0.0844 | 0.0630 | 0.0765 | 0.0916 | 0.0989 |
| 408 | 0.0950 | 0.0864 | 0.0644 | 0.0782 | 0.0936 | 0.1010 |
| 414 | 0.0970 | 0.0884 | 0.0659 | 0.0800 | 0.0955 | 0.1031 |
| 420 | 0.0990 | 0.0905 | 0.0675 | 0.0817 | 0.0975 | 0.1053 |
| 426 | 0.1010 | 0.0926 | 0.0690 | 0.0835 | 0.0996 | 0.1076 |
| 432 | 0.1030 | 0.0948 | 0.0706 | 0.0854 | 0.1017 | 0.1099 |
| 438 | 0.1051 | 0.0970 | 0.0723 | 0.0873 | 0.1038 | 0.1122 |
| 444 | 0.1072 | 0.0993 | 0.0740 | 0.0892 | 0.1060 | 0.1146 |
| 450 | 0.1093 | 0.1016 | 0.0757 | 0.0912 | 0.1082 | 0.1170 |
| 456 | 0.1114 | 0.1040 | 0.0774 | 0.0932 | 0.1104 | 0.1195 |
| 462 | 0.1135 | 0.1064 | 0.0792 | 0.0952 | 0.1127 | 0.1220 |
| 468 | 0.1156 | 0.1089 | 0.0810 | 0.0973 | 0.1150 | 0.1245 |
| 474 | 0.1178 | 0.1114 | 0.0829 | 0.0994 | 0.1174 | 0.1271 |
| 480 | 0.1200 | 0.1140 | 0.0848 | 0.1016 | 0.1198 | 0.1298 |

| Time (min) | 24 Hour Rainfall Distributions | | | | | |
|---|---|---|---|---|---|---|
| | Type II | Type III | NOAA A | NOAA B | NOAA C | NOAA D |
| 486 | 0.1223 | 0.1167 | 0.0867 | 0.1038 | 0.1223 | 0.1325 |
| 492 | 0.1246 | 0.1194 | 0.0887 | 0.1060 | 0.1247 | 0.1352 |
| 498 | 0.1271 | 0.1223 | 0.0907 | 0.1083 | 0.1273 | 0.1380 |
| 504 | 0.1296 | 0.1253 | 0.0928 | 0.1106 | 0.1298 | 0.1408 |
| 510 | 0.1323 | 0.1284 | 0.0949 | 0.1129 | 0.1324 | 0.1437 |
| 516 | 0.1350 | 0.1317 | 0.0970 | 0.1153 | 0.1351 | 0.1466 |
| 522 | 0.1379 | 0.1350 | 0.0991 | 0.1178 | 0.1378 | 0.1495 |
| 528 | 0.1408 | 0.1385 | 0.1013 | 0.1202 | 0.1405 | 0.1525 |
| 534 | 0.1439 | 0.1421 | 0.1036 | 0.1227 | 0.1432 | 0.1556 |
| 540 | 0.1470 | 0.1458 | 0.1058 | 0.1253 | 0.1461 | 0.1587 |
| 546 | 0.1502 | 0.1496 | 0.1082 | 0.1280 | 0.1490 | 0.1619 |
| 552 | 0.1534 | 0.1535 | 0.1108 | 0.1309 | 0.1521 | 0.1653 |
| 558 | 0.1566 | 0.1575 | 0.1135 | 0.1339 | 0.1554 | 0.1689 |
| 564 | 0.1598 | 0.1617 | 0.1164 | 0.1370 | 0.1588 | 0.1726 |
| 570 | 0.1630 | 0.1659 | 0.1195 | 0.1404 | 0.1623 | 0.1765 |
| 576 | 0.1663 | 0.1703 | 0.1227 | 0.1438 | 0.1660 | 0.1805 |
| 582 | 0.1697 | 0.1748 | 0.1260 | 0.1475 | 0.1699 | 0.1847 |
| 588 | 0.1733 | 0.1794 | 0.1295 | 0.1512 | 0.1739 | 0.1891 |
| 594 | 0.1771 | 0.1842 | 0.1331 | 0.1552 | 0.1780 | 0.1936 |
| 600 | 0.1810 | 0.1890 | 0.1370 | 0.1593 | 0.1823 | 0.1982 |
| 606 | 0.1851 | 0.1940 | 0.1409 | 0.1635 | 0.1868 | 0.2031 |
| 612 | 0.1895 | 0.1993 | 0.1450 | 0.1679 | 0.1914 | 0.2081 |
| 618 | 0.1941 | 0.2048 | 0.1493 | 0.1725 | 0.1961 | 0.2132 |
| 624 | 0.1989 | 0.2105 | 0.1537 | 0.1772 | 0.2010 | 0.2185 |
| 630 | 0.2040 | 0.2165 | 0.1583 | 0.1820 | 0.2061 | 0.2240 |
| 636 | 0.2094 | 0.2227 | 0.1635 | 0.1875 | 0.2117 | 0.2300 |
| 642 | 0.2152 | 0.2292 | 0.1693 | 0.1936 | 0.2179 | 0.2366 |
| 648 | 0.2214 | 0.2359 | 0.1758 | 0.2003 | 0.2247 | 0.2438 |
| 654 | 0.2280 | 0.2428 | 0.1829 | 0.2077 | 0.2321 | 0.2516 |
| 660 | 0.2350 | 0.2500 | 0.1907 | 0.2156 | 0.2400 | 0.2600 |
| 666 | 0.2427 | 0.2578 | 0.1997 | 0.2248 | 0.2490 | 0.2693 |
| 672 | 0.2513 | 0.2664 | 0.2101 | 0.2352 | 0.2591 | 0.2797 |
| 678 | 0.2609 | 0.2760 | 0.2218 | 0.2468 | 0.2702 | 0.2911 |
| 684 | 0.2715 | 0.2866 | 0.2348 | 0.2596 | 0.2825 | 0.3036 |
| 690 | 0.2830 | 0.2980 | 0.2490 | 0.2735 | 0.2955 | 0.3170 |
| 696 | 0.3068 | 0.3143 | 0.2718 | 0.2955 | 0.3157 | 0.3351 |
| 702 | 0.3544 | 0.3394 | 0.2957 | 0.3186 | 0.3370 | 0.3542 |
| 708 | 0.4308 | 0.3733 | 0.3295 | 0.3504 | 0.3662 | 0.3803 |
| 714 | 0.5679 | 0.4160 | 0.3782 | 0.3949 | 0.4067 | 0.4165 |
| 720 | 0.6630 | 0.5000 | 0.4666 | 0.4729 | 0.4766 | 0.4791 |

| Time (min) | 24 Hour Rainfall Distributions | | | | | |
|---|---|---|---|---|---|---|
| | Type II | Type III | NOAA A | NOAA B | NOAA C | NOAA D |
| 726 | 0.6820 | 0.5840 | 0.6218 | 0.6051 | 0.5933 | 0.5835 |
| 732 | 0.6986 | 0.6267 | 0.6705 | 0.6496 | 0.6338 | 0.6197 |
| 738 | 0.7130 | 0.6606 | 0.7043 | 0.6815 | 0.6630 | 0.6459 |
| 744 | 0.7252 | 0.6857 | 0.7282 | 0.7046 | 0.6843 | 0.6649 |
| 750 | 0.7350 | 0.7020 | 0.7510 | 0.7265 | 0.7045 | 0.6830 |
| 756 | 0.7434 | 0.7134 | 0.7652 | 0.7404 | 0.7176 | 0.6964 |
| 762 | 0.7514 | 0.7240 | 0.7782 | 0.7532 | 0.7298 | 0.7089 |
| 768 | 0.7588 | 0.7336 | 0.7899 | 0.7649 | 0.7409 | 0.7203 |
| 774 | 0.7656 | 0.7422 | 0.8003 | 0.7752 | 0.7510 | 0.7307 |
| 780 | 0.7720 | 0.7500 | 0.8094 | 0.7844 | 0.7600 | 0.7401 |
| 786 | 0.7780 | 0.7572 | 0.8171 | 0.7924 | 0.7679 | 0.7484 |
| 792 | 0.7836 | 0.7641 | 0.8242 | 0.7997 | 0.7753 | 0.7562 |
| 798 | 0.7890 | 0.7708 | 0.8307 | 0.8064 | 0.7821 | 0.7634 |
| 804 | 0.7942 | 0.7773 | 0.8365 | 0.8125 | 0.7883 | 0.7700 |
| 810 | 0.7990 | 0.7835 | 0.8417 | 0.8180 | 0.7939 | 0.7760 |
| 816 | 0.8036 | 0.7895 | 0.8463 | 0.8228 | 0.7990 | 0.7815 |
| 822 | 0.8080 | 0.7952 | 0.8507 | 0.8275 | 0.8039 | 0.7868 |
| 828 | 0.8122 | 0.8007 | 0.8550 | 0.8321 | 0.8086 | 0.7920 |
| 834 | 0.8162 | 0.8060 | 0.8591 | 0.8365 | 0.8132 | 0.7969 |
| 840 | 0.8200 | 0.8110 | 0.8631 | 0.8407 | 0.8177 | 0.8018 |
| 846 | 0.8237 | 0.8158 | 0.8669 | 0.8448 | 0.8220 | 0.8064 |
| 852 | 0.8273 | 0.8206 | 0.8705 | 0.8488 | 0.8261 | 0.8109 |
| 858 | 0.8308 | 0.8252 | 0.8740 | 0.8526 | 0.8301 | 0.8153 |
| 864 | 0.8342 | 0.8297 | 0.8774 | 0.8562 | 0.8340 | 0.8195 |
| 870 | 0.8376 | 0.8341 | 0.8805 | 0.8597 | 0.8377 | 0.8235 |
| 876 | 0.8409 | 0.8383 | 0.8836 | 0.8630 | 0.8412 | 0.8274 |
| 882 | 0.8442 | 0.8425 | 0.8865 | 0.8661 | 0.8446 | 0.8311 |
| 888 | 0.8474 | 0.8465 | 0.8892 | 0.8692 | 0.8479 | 0.8347 |
| 894 | 0.8505 | 0.8504 | 0.8918 | 0.8720 | 0.8510 | 0.8381 |
| 900 | 0.8535 | 0.8543 | 0.8942 | 0.8747 | 0.8540 | 0.8414 |
| 906 | 0.8565 | 0.8579 | 0.8965 | 0.8773 | 0.8568 | 0.8444 |
| 912 | 0.8594 | 0.8615 | 0.8987 | 0.8798 | 0.8595 | 0.8475 |
| 918 | 0.8622 | 0.8650 | 0.9009 | 0.8822 | 0.8622 | 0.8505 |
| 924 | 0.8649 | 0.8683 | 0.9030 | 0.8847 | 0.8649 | 0.8534 |
| 930 | 0.8676 | 0.8716 | 0.9052 | 0.8871 | 0.8676 | 0.8564 |
| 936 | 0.8702 | 0.8747 | 0.9072 | 0.8894 | 0.8702 | 0.8592 |
| 942 | 0.8728 | 0.8777 | 0.9093 | 0.8917 | 0.8727 | 0.8620 |
| 948 | 0.8753 | 0.8806 | 0.9113 | 0.8940 | 0.8753 | 0.8648 |
| 954 | 0.8777 | 0.8833 | 0.9133 | 0.8962 | 0.8778 | 0.8675 |
| 960 | 0.8800 | 0.8860 | 0.9152 | 0.8984 | 0.8802 | 0.8702 |

| Time (min) | 24 Hour Rainfall Distributions | | | | | |
|---|---|---|---|---|---|---|
| | Type II | Type III | NOAA A | NOAA B | NOAA C | NOAA D |
| 966 | 0.8823 | 0.8886 | 0.9171 | 0.9006 | 0.8826 | 0.8729 |
| 972 | 0.8846 | 0.8911 | 0.9190 | 0.9027 | 0.8850 | 0.8755 |
| 978 | 0.8868 | 0.8936 | 0.9208 | 0.9048 | 0.8873 | 0.8780 |
| 984 | 0.8890 | 0.8960 | 0.9226 | 0.9068 | 0.8896 | 0.8805 |
| 990 | 0.8912 | 0.8984 | 0.9243 | 0.9088 | 0.8918 | 0.8830 |
| 996 | 0.8934 | 0.9007 | 0.9261 | 0.9108 | 0.8940 | 0.8854 |
| 1002 | 0.8955 | 0.9030 | 0.9277 | 0.9127 | 0.8962 | 0.8878 |
| 1008 | 0.8976 | 0.9052 | 0.9294 | 0.9146 | 0.8983 | 0.8901 |
| 1014 | 0.8997 | 0.9074 | 0.9310 | 0.9165 | 0.9004 | 0.8924 |
| 1020 | 0.9018 | 0.9095 | 0.9326 | 0.9183 | 0.9025 | 0.8947 |
| 1026 | 0.9038 | 0.9116 | 0.9341 | 0.9200 | 0.9045 | 0.8969 |
| 1032 | 0.9058 | 0.9136 | 0.9356 | 0.9218 | 0.9064 | 0.8990 |
| 1038 | 0.9078 | 0.9156 | 0.9370 | 0.9235 | 0.9084 | 0.9011 |
| 1044 | 0.9098 | 0.9175 | 0.9385 | 0.9251 | 0.9103 | 0.9032 |
| 1050 | 0.9117 | 0.9194 | 0.9398 | 0.9267 | 0.9121 | 0.9052 |
| 1056 | 0.9136 | 0.9212 | 0.9412 | 0.9283 | 0.9139 | 0.9072 |
| 1062 | 0.9155 | 0.9230 | 0.9425 | 0.9298 | 0.9157 | 0.9091 |
| 1068 | 0.9174 | 0.9247 | 0.9438 | 0.9313 | 0.9174 | 0.9110 |
| 1074 | 0.9192 | 0.9264 | 0.9450 | 0.9328 | 0.9191 | 0.9128 |
| 1080 | 0.9210 | 0.9280 | 0.9462 | 0.9342 | 0.9208 | 0.9146 |
| 1086 | 0.9228 | 0.9296 | 0.9474 | 0.9356 | 0.9224 | 0.9164 |
| 1092 | 0.9246 | 0.9312 | 0.9485 | 0.9370 | 0.9240 | 0.9181 |
| 1098 | 0.9263 | 0.9327 | 0.9497 | 0.9383 | 0.9256 | 0.9199 |
| 1104 | 0.9280 | 0.9343 | 0.9508 | 0.9397 | 0.9271 | 0.9216 |
| 1110 | 0.9297 | 0.9358 | 0.9520 | 0.9410 | 0.9287 | 0.9233 |
| 1116 | 0.9314 | 0.9373 | 0.9531 | 0.9424 | 0.9303 | 0.9250 |
| 1122 | 0.9330 | 0.9388 | 0.9542 | 0.9437 | 0.9318 | 0.9267 |
| 1128 | 0.9346 | 0.9403 | 0.9553 | 0.9450 | 0.9334 | 0.9283 |
| 1134 | 0.9362 | 0.9418 | 0.9564 | 0.9463 | 0.9349 | 0.9300 |
| 1140 | 0.9378 | 0.9433 | 0.9575 | 0.9476 | 0.9364 | 0.9316 |
| 1146 | 0.9393 | 0.9447 | 0.9585 | 0.9489 | 0.9379 | 0.9333 |
| 1152 | 0.9408 | 0.9461 | 0.9596 | 0.9501 | 0.9394 | 0.9349 |
| 1158 | 0.9423 | 0.9475 | 0.9606 | 0.9514 | 0.9409 | 0.9365 |
| 1164 | 0.9438 | 0.9489 | 0.9617 | 0.9527 | 0.9424 | 0.9381 |
| 1170 | 0.9452 | 0.9503 | 0.9627 | 0.9539 | 0.9439 | 0.9397 |
| 1176 | 0.9466 | 0.9517 | 0.9638 | 0.9551 | 0.9453 | 0.9413 |
| 1182 | 0.9480 | 0.9530 | 0.9648 | 0.9564 | 0.9468 | 0.9429 |
| 1188 | 0.9494 | 0.9544 | 0.9658 | 0.9576 | 0.9482 | 0.9445 |
| 1194 | 0.9507 | 0.9557 | 0.9668 | 0.9588 | 0.9496 | 0.9460 |
| 1200 | 0.9520 | 0.9570 | 0.9678 | 0.9600 | 0.9511 | 0.9475 |

| Time (min) | 24 Hour Rainfall Distributions | | | | | |
|---|---|---|---|---|---|---|
| | Type II | Type III | NOAA A | NOAA B | NOAA C | NOAA D |
| 1206 | 0.9533 | 0.9583 | 0.9688 | 0.9612 | 0.9525 | 0.9491 |
| 1212 | 0.9546 | 0.9596 | 0.9697 | 0.9623 | 0.9539 | 0.9506 |
| 1218 | 0.9559 | 0.9609 | 0.9707 | 0.9635 | 0.9553 | 0.9521 |
| 1224 | 0.9572 | 0.9621 | 0.9716 | 0.9647 | 0.9566 | 0.9536 |
| 1230 | 0.9584 | 0.9634 | 0.9726 | 0.9658 | 0.9580 | 0.9551 |
| 1236 | 0.9597 | 0.9646 | 0.9735 | 0.9669 | 0.9594 | 0.9566 |
| 1242 | 0.9610 | 0.9658 | 0.9744 | 0.9681 | 0.9607 | 0.9580 |
| 1248 | 0.9622 | 0.9670 | 0.9754 | 0.9692 | 0.9621 | 0.9595 |
| 1254 | 0.9635 | 0.9682 | 0.9763 | 0.9703 | 0.9634 | 0.9609 |
| 1260 | 0.9648 | 0.9694 | 0.9772 | 0.9714 | 0.9647 | 0.9623 |
| 1266 | 0.9660 | 0.9706 | 0.9781 | 0.9725 | 0.9660 | 0.9637 |
| 1272 | 0.9672 | 0.9718 | 0.9789 | 0.9736 | 0.9673 | 0.9652 |
| 1278 | 0.9685 | 0.9730 | 0.9798 | 0.9746 | 0.9686 | 0.9665 |
| 1284 | 0.9697 | 0.9741 | 0.9807 | 0.9757 | 0.9699 | 0.9679 |
| 1290 | 0.9709 | 0.9752 | 0.9815 | 0.9767 | 0.9712 | 0.9693 |
| 1296 | 0.9722 | 0.9764 | 0.9824 | 0.9778 | 0.9724 | 0.9707 |
| 1302 | 0.9734 | 0.9775 | 0.9832 | 0.9788 | 0.9737 | 0.9720 |
| 1308 | 0.9746 | 0.9786 | 0.9840 | 0.9798 | 0.9749 | 0.9734 |
| 1314 | 0.9758 | 0.9797 | 0.9848 | 0.9808 | 0.9762 | 0.9747 |
| 1320 | 0.9770 | 0.9808 | 0.9857 | 0.9818 | 0.9774 | 0.9760 |
| 1326 | 0.9782 | 0.9818 | 0.9864 | 0.9828 | 0.9786 | 0.9773 |
| 1332 | 0.9794 | 0.9829 | 0.9872 | 0.9838 | 0.9798 | 0.9786 |
| 1338 | 0.9806 | 0.9839 | 0.9880 | 0.9848 | 0.9810 | 0.9799 |
| 1344 | 0.9818 | 0.9850 | 0.9888 | 0.9857 | 0.9822 | 0.9812 |
| 1350 | 0.9829 | 0.9860 | 0.9895 | 0.9867 | 0.9834 | 0.9824 |
| 1356 | 0.9841 | 0.9870 | 0.9903 | 0.9876 | 0.9845 | 0.9837 |
| 1362 | 0.9853 | 0.9880 | 0.9910 | 0.9886 | 0.9857 | 0.9849 |
| 1368 | 0.9864 | 0.9890 | 0.9918 | 0.9895 | 0.9868 | 0.9861 |
| 1374 | 0.9876 | 0.9900 | 0.9925 | 0.9904 | 0.9879 | 0.9874 |
| 1380 | 0.9888 | 0.9909 | 0.9932 | 0.9913 | 0.9891 | 0.9886 |
| 1386 | 0.9899 | 0.9919 | 0.9939 | 0.9922 | 0.9902 | 0.9898 |
| 1392 | 0.9910 | 0.9928 | 0.9946 | 0.9931 | 0.9913 | 0.9909 |
| 1398 | 0.9922 | 0.9938 | 0.9953 | 0.9940 | 0.9924 | 0.9921 |
| 1404 | 0.9933 | 0.9947 | 0.9960 | 0.9948 | 0.9935 | 0.9933 |
| 1410 | 0.9944 | 0.9956 | 0.9966 | 0.9957 | 0.9945 | 0.9944 |
| 1416 | 0.9956 | 0.9965 | 0.9973 | 0.9965 | 0.9956 | 0.9956 |
| 1422 | 0.9967 | 0.9974 | 0.9980 | 0.9974 | 0.9967 | 0.9967 |
| 1428 | 0.9978 | 0.9983 | 0.9986 | 0.9982 | 0.9977 | 0.9978 |
| 1434 | 0.9989 | 0.9991 | 0.9992 | 0.9990 | 0.9987 | 0.9989 |
| 1440 | 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 |

## APPENDIX C: MANNING'S N-VALUES (CHOW, 1959)

| Type of Channel and Description | Minimum | Normal | Maximum |
|---|---|---|---|
| Natural streams - minor streams (top width at flood stage < 100 ft) | | | |
| **1. Main Channels** | | | |
| a. clean, straight, full stage, no rifts or deep pools | 0.025 | 0.030 | 0.033 |
| b. same as above, but more stones and weeds | 0.030 | 0.035 | 0.040 |
| c. clean, winding, some pools and shoals | 0.033 | 0.040 | 0.045 |
| d. same as above, but some weeds and stones | 0.035 | 0.045 | 0.050 |
| e. same as above, lower stages, more ineffective | 0.040 | 0.048 | 0.055 |
| f. same as "d" with more stones | 0.045 | 0.050 | 0.060 |
| g. sluggish reaches, weedy, deep pools | 0.050 | 0.070 | 0.080 |
| h. very weedy reaches, deep pools, or floodways | 0.075 | 0.100 | 0.150 |
| **2. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages** | | | |
| a. bottom: gravels, cobbles, and few boulders | 0.030 | 0.040 | 0.050 |
| b. bottom: cobbles with large boulders | 0.040 | 0.050 | 0.070 |
| **3. Floodplains** | | | |
| a. Pasture, no brush | | | |
| 1.short grass | 0.025 | 0.030 | 0.035 |
| 2. high grass | 0.030 | 0.035 | 0.050 |
| b. Cultivated areas | | | |
| 1. no crop | 0.020 | 0.030 | 0.040 |
| 2. mature row crops | 0.025 | 0.035 | 0.045 |
| 3. mature field crops | 0.030 | 0.040 | 0.050 |
| c. Brush | | | |
| 1. scattered brush, heavy weeds | 0.035 | 0.050 | 0.070 |
| 2. light brush and trees, in winter | 0.035 | 0.050 | 0.060 |
| 3. light brush and trees, in summer | 0.040 | 0.060 | 0.080 |
| 4. medium to dense brush, in winter | 0.045 | 0.070 | 0.110 |
| 5. medium to dense brush, in summer | 0.070 | 0.100 | 0.160 |
| d. Trees | | | |
| 1. dense willows, summer, straight | 0.110 | 0.150 | 0.200 |
| 2. cleared land with tree stumps, no sprouts | 0.030 | 0.040 | 0.050 |
| 3. same as above, but with heavy growth of | 0.050 | 0.060 | 0.080 |
| 4. heavy stand of timber, a few down trees, | 0.080 | 0.100 | 0.120 |
| 5. same as 4. with flood stage reaching | 0.100 | 0.120 | 0.160 |
| **4. Excavated or Dredged Channels** | | | |
| a. Earth, straight, and uniform | | | |
| 1. clean, recently completed | 0.016 | 0.018 | 0.020 |
| 2. clean, after weathering | 0.018 | 0.022 | 0.025 |

| | | | |
|---|---|---|---|
| 3. gravel, uniform section, clean | 0.022 | 0.025 | 0.030 |
| 4. with short grass, few weeds | 0.022 | 0.027 | 0.033 |
| b. Earth winding and sluggish | | | |
| 1. no vegetation | 0.023 | 0.025 | 0.030 |
| 2. grass, some weeds | 0.025 | 0.030 | 0.033 |
| 3. dense weeds or aquatic plants in deep | 0.030 | 0.035 | 0.040 |
| 4. earth bottom and rubble sides | 0.028 | 0.030 | 0.035 |
| 5. stony bottom and weedy banks | 0.025 | 0.035 | 0.040 |
| 6. cobble bottom and clean sides | 0.030 | 0.040 | 0.050 |
| c. Dragline-excavated or dredged | | | |
| 1. no vegetation | 0.025 | 0.028 | 0.033 |
| 2. light brush on banks | 0.035 | 0.050 | 0.060 |
| d. Rock cuts | | | |
| 1. smooth and uniform | 0.025 | 0.035 | 0.040 |
| 2. jagged and irregular | 0.035 | 0.040 | 0.050 |
| e. Channels not maintained, weeds and brush uncut | | | |
| 1. dense weeds, high as flow depth | 0.050 | 0.080 | 0.120 |
| 2. clean bottom, brush on sides | 0.040 | 0.050 | 0.080 |
| 3. same as above, highest stage of flow | 0.045 | 0.070 | 0.110 |
| 4. dense brush, high stage | 0.080 | 0.100 | 0.140 |
| **5. Lined or Constructed Channels** | | | |
| a. Cement | | | |
| 1. neat surface | 0.010 | 0.011 | 0.013 |
| 2. mortar | 0.011 | 0.013 | 0.015 |
| b. Wood | | | |
| 1. planed, untreated | 0.010 | 0.012 | 0.014 |
| 2. planed, creosoted | 0.011 | 0.012 | 0.015 |
| 3. unplaned | 0.011 | 0.013 | 0.015 |
| 4. plank with battens | 0.012 | 0.015 | 0.018 |
| 5. lined with roofing paper | 0.010 | 0.014 | 0.017 |
| | | | |
| c. Concrete | | | |
| 1. trowel finish | 0.011 | 0.013 | 0.015 |
| 2. float finish | 0.013 | 0.015 | 0.016 |
| 3. finished, with gravel on bottom | 0.015 | 0.017 | 0.020 |
| 4. unfinished | 0.014 | 0.017 | 0.020 |
| 5. gunite, good section | 0.016 | 0.019 | 0.023 |
| 6. gunite, wavy section | 0.018 | 0.022 | 0.025 |
| 7. on good excavated rock | 0.017 | 0.020 | |
| 8. on irregular excavated rock | 0.022 | 0.027 | |
| d. Concrete bottom float finish with sides of: | | | |

| | | | |
|---|---|---|---|
| 1. dressed stone in mortar | 0.015 | 0.017 | 0.020 |
| 2. random stone in mortar | 0.017 | 0.020 | 0.024 |
| 3. cement rubble masonry, plastered | 0.016 | 0.020 | 0.024 |
| 4. cement rubble masonry | 0.020 | 0.025 | 0.030 |
| 5. dry rubble or riprap | 0.020 | 0.030 | 0.035 |
| e. Gravel bottom with sides of: | | | |
| 1. formed concrete | 0.017 | 0.020 | 0.025 |
| 2. random stone mortar | 0.020 | 0.023 | 0.026 |
| 3. dry rubble or riprap | 0.023 | 0.033 | 0.036 |
| f. Brick | | | |
| 1. glazed | 0.011 | 0.013 | 0.015 |
| 2. in cement mortar | 0.012 | 0.015 | 0.018 |
| g. Masonry | | | |
| 1. cemented rubble | 0.017 | 0.025 | 0.030 |
| 2. dry rubble | 0.023 | 0.032 | 0.035 |
| h. Dressed ashlar/stone paving | 0.013 | 0.015 | 0.017 |
| i. Asphalt | | | |
| 1. smooth | 0.013 | 0.013 | |
| 2. rough | 0.016 | 0.016 | |
| j. Vegetal lining | 0.030 | | 0.500 |

| Type of Conduit and Description | Minimum | Normal | Maximum |
|---|---|---|---|
| **1. Brass, smooth:** | 0.009 | 0.010 | 0.013 |
| **2. Steel:** | | | |
| Lockbar and welded | 0.010 | 0.012 | 0.014 |
| Riveted and spiral | 0.013 | 0.016 | 0.017 |
| **3. Cast Iron:** | | | |
| Coated | 0.010 | 0.013 | 0.014 |
| Uncoated | 0.011 | 0.014 | 0.016 |
| **4. Wrought Iron:** | | | |
| Black | 0.012 | 0.014 | 0.015 |
| Galvanized | 0.013 | 0.016 | 0.017 |
| **5. Corrugated Metal:** | | | |
| Subdrain | 0.017 | 0.019 | 0.021 |
| Stormdrain | 0.021 | 0.024 | 0.030 |
| **6. Cement:** | | | |
| Neat Surface | 0.010 | 0.011 | 0.013 |
| Mortar | 0.011 | 0.013 | 0.015 |

| | | | |
|---|---|---|---|
| **7. Concrete:** | | | |
| Culvert, straight and free of debris | 0.010 | 0.011 | 0.013 |
| Culvert with bends, connections, and some debris | 0.011 | 0.013 | 0.014 |
| Finished | 0.011 | 0.012 | 0.014 |
| Sewer with manholes, inlet, etc., straight | 0.013 | 0.015 | 0.017 |
| Unfinished, steel form | 0.012 | 0.013 | 0.014 |
| Unfinished, smooth wood form | 0.012 | 0.014 | 0.016 |
| Unfinished, rough wood form | 0.015 | 0.017 | 0.020 |
| **8. Wood:** | | | |
| Stave | 0.010 | 0.012 | 0.014 |
| Laminated, treated | 0.015 | 0.017 | 0.020 |
| **9. Clay:** | | | |
| Common drainage tile | 0.011 | 0.013 | 0.017 |
| Vitrified sewer | 0.011 | 0.014 | 0.017 |
| Vitrified sewer with manholes, inlet, etc. | 0.013 | 0.015 | 0.017 |
| Vitrified Subdrain with open joint | 0.014 | 0.016 | 0.018 |
| **10. Brickwork:** | | | |
| Glazed | 0.011 | 0.013 | 0.015 |
| Lined with cement mortar | 0.012 | 0.015 | 0.017 |
| Sanitary sewers coated with sewage slime with bends and connections | 0.012 | 0.013 | 0.016 |
| Paved invert, sewer, smooth bottom | 0.016 | 0.019 | 0.020 |
| Rubble masonry, cemented | 0.018 | 0.025 | 0.030 |

# APPENDIX D: TOPOGRAPHIC MAPS

Hydrologic analyses and designs require data for site characterization and input to stormwater models. Map data, defined as watershed physical parameters obtained from published sources such as maps, are used for such purposes. Some of the types of maps and/or map tools one should know where to find and how to use include:

1) StreamStats
2) Aerial photographs
3) Soil maps

## D.1 StreamStats

StreamStats is a Web-based tool that provides streamflow statistics, drainage-basin characteristics, and other information for USGS stream gaging stations and user-selected un-gaged sites on streams. When users select the location of a stream gaging station, StreamStats provides information from a database. When users select a site on an un-gaged stream, StreamStats determines the drainage-basin boundary, computes a variety of drainage-basin characteristics, and solves regression equations to estimate streamflow statistics.

The U.S. Geological Survey (USGS) developed the South Carolina StreamStats application in cooperation with the SCDOT. StreamStats (https://water.usgs.gov/osw/streamstats/) is a map-based web application that provides analytical tools useful for water-resources planning and management, and many engineering analysis and design purposes. The web application delineates drainage areas at user-selected sites on South Carolina streams, generates basin characteristics, and, where appropriate, estimates peak-flow statistics. StreamStats users also can obtain published flow statistics for USGS stream gages, such as peak flow, low flow, and daily mean flow durations.

The functionality of StreamStats includes the ability to:
- Navigate and view base-map features in the user interface, such as roads, streams, political boundaries, and USGS topographic maps.
- Zoom in or out to different map scales based on user input of (1) a drawn rectangle surrounding an area of interest, (2) latitude and longitude coordinates, (3) place name, (4) stream reach code, or (5) a specified scale;
- Access previously published peak-flow frequency and other streamflow statistics, basin characteristics, and descriptive information for USGS stream gages, plus a link to the USGS National Water Information System that provides access to historical and real-time data collected at selected stream gages.
- Delineate and edit a drainage-basin boundary for a user-selected point on a stream.
- Compute basin characteristics for a user-selected point on a stream, such as drainage area, stream slope, mean annual rainfall, and land cover from the National Land Cover Dataset.
- Estimate rural and urban peak-flow frequency statistics and provide indicators of the accuracy of the estimates at a user-selected point on a stream.

- Download shapefiles of the user-selected basin outlet point, the delineated basin, longest flow path, and the points used to calculate the slope of the longest flow path for use in other applications, along with associated basin characteristics and streamflow statistics.
- Trace information upstream or downstream from a user-selected point on a stream to identify the connected stream network and natural or manmade features that may affect the quantity or quality of the streamflow.
- Obtain elevation profiles between selected points on the stream network or the land surface. The profile coordinates (x, y, and z values) can be saved by the user in an Excel format.
- Trace the flow path of water from a selected point on the land surface to the stream network and then downstream; and
- Print the results displayed in the user-interface map frame, the basin characteristics report, or streamflow statistics in the StreamStats un-gaged site report.

### D.2 Topographic Maps

Topographic maps are essential in evaluating geologic, hydrogeologic, hydrologic, and topographic characteristics of a property. Historical topographic maps are useful in determining how topography and property development has changed over time, and they indicate possible risks or concerns. Topographic maps, also known as “Topos,” complement aerial photography to represent the earth’s surface. A topo map is a two-dimensional representation of our three-dimensional world. Both horizontal and vertical data can be recorded. The characteristics of a topographic map can vary but what is distinctive is that the shape of the Earth’s surface is shown by contour lines. Contours are imaginary lines that join points of equal elevation on the surface of the land above or below a reference surface, such as mean sea level. A contour line displays the peaks and the valleys of the land which makes it possible to measure the height of mountains and the steepness of the slope

Topographic maps conventionally show topography, or land contours, by means of contour lines. Contour lines are curves that connect contiguous points of the same altitude. In other words, every point on the marked line of 100 ft elevation is 100 feet above mean sea level.

More than contours are shown on a topographic map. The maps depict the natural features as well as the manmade attributes of the land. The natural features include the mountains and hills, rivers and streams, and type of vegetation. Manmade structures depicted on a topo map may include railway lines, buildings, utilities, houses, cities, schools, roads, and highways.

The various features shown on topo maps are represented by conventional signs or symbols. For example, colors can be used to indicate the classification of roads. These signs are usually explained in the margin of the map, or on a separately published characteristic sheet.

Currently, topographic maps are prepared using photogrammetric interpretation of aerial photography, LiDAR, and other remote sensing techniques. Older topographic maps were prepared using traditional surveying instruments.

### D.3 Soil Maps

Soil mapping is the process of classifying soil types and other soil properties in a given area and geo-encoding such information. It applies the principles of soil science and draws heavily from geomorphology, theories of soil formation, physical geography, and analysis of vegetation and land use patterns. Primary data for the soil survey are acquired by field sampling and by remote sensing. Remote sensing principally uses aerial photography, but LiDAR and other digital techniques are gaining in popularity. In the past, a soil scientist would take hard-copies of aerial photography, topo-sheets, and mapping keys into the field with them. Today, a growing number of soil scientists bring a tablet computer and GPS into the field. The tablet may be loaded with digital aerial photos, LiDAR, topography, soil geodatabases, mapping keys, and more.

The term soil survey may also be used as a noun to describe the published results. These surveys were once published in book form for individual counties by the National Cooperative Soil Survey. Today, soil surveys are no longer published in book form; they are published to the web and accessed on NRCS Web Soil Survey where a person can create a custom soil survey. This allows for rapid flow of the latest soil information. In the past, it could take years to publish a paper soil survey. Today it takes only moments for changes to go live to the public. Also, the most current soil survey data is made available on the Download Soils Data tab at NRCS Web Soil Survey for high end GIS users.

Web Soil Survey (WSS) provides soil data and information produced by the National Cooperative Soil Survey. It is operated by the USDA Natural Resources Conservation Service (NRCS) and provides access to the largest natural resource information system in the world. NRCS has soil maps and data available online for more than 95 percent of the nation’s counties. The site is updated and maintained online as the single authoritative source of soil survey information.

### D.4 Watershed Delineation and Measurements

A watershed is delineated on a topographic map, usually a USGS topographic map, or a topographic map developed from site survey. Topographic maps also can be developed from aerial photographs or with GIS.

The watershed boundary is a ridge. Water falling inside the boundary flows to the watershed outlet. Water falling outside the boundary flows into other drainage basins (another label for watershed) and exits at other outlets.

Watersheds are delineated to isolate the area contributing runoff to a point. Once the watershed is delineated, overlay maps for land use and soil type are prepared and measurements made of all unique intersections of land use and soil type. Such data are necessary to determine NRCS Curve Numbers.

Watershed measurements include:
1. Surface area. Use GIS, computer software or, if doing it manually, with a planimeter.
2. Average watershed slope. Use GIS or grid-overlay method if doing it manually.
3. Hydraulic length (preferably flow path segment lengths)
4. Area of each unique land use and soil type intersection.

### D.5 How to Read Topographic Maps

*The following discussion was prepared by personnel in the New Hampshire office of the USDA Natural Resources Conservation Service and is included in this manual for guidance in using topographic maps.*

To successfully delineate a watershed boundary, the evaluator should visualize the landscape as represented by a topographic map. This is not difficult once the following basic concepts of topographic maps are understood.

Each contour line represents a ground elevation or vertical distance above a reference point (datum), such as mean sea level. A contour line is level with respect to the earth's surface just like the top of a building foundation, i.e., all points along a contour line are at the same elevation.

The difference in elevation between two adjacent contours is called the contour interval. This is given in the map legend. It represents the vertical distance one must climb or descend to go from one contour elevation to the next.

The horizontal distance between contours is determined by the steepness of the landscape. On relatively flat ground, such as found throughout much of the Coastal Plain physiographic province, adjacent 20 and 30-feet contours may be very far apart horizontally. On a steep hillside, such as found in the Valley and Ridge province, adjacent 1020 and 1030-feet contours may be directly above and below each other or, in other words, very close to each other horizontally. In each case, the vertical distance between the contour lines is ten feet. Some maps use a 5-foot or smaller contour interval.

How do contours relate to water flow?  The flow path is always perpendicular to contour lines.[12] In the case of an isolated hill, water flows down on all sides, crossing successively lower elevation contours at 90-degree angles. Water flows from the top of a saddle or ridge, down each side, in the same way water flows down each side of the rooftop on a house.

As the water continues downhill it flows into progressively larger watercourses and ultimately into the ocean. Any point on a watercourse can be used to define a watershed. That is, the entire drainage area of a major river like the Congaree can be considered a watershed, but the drainage areas of each of its tributaries are also watersheds. Each tributary, in turn, has tributaries, and

---

[12] Draw an analogy to groundwater flow nets, contour lines are the same as equipotential lines and water flow pathways are equivalent to streamlines. Streamlines and equipotential lines cross perpendicular.

each one of these tributaries has a watershed. This process of subdivision can continue until very small, local watersheds are defined which drain only a few acres, and may not contain a defined watercourse.

A general rule of thumb is that topographic lines in a valley and adjacent to a stream always point upstream. Why? This is because water flows from regions of higher elevation to regions of lower elevation. With that in mind, it is not difficult to make out drainage patterns and the direction of flow on the landscape even when there is no stream depicted on the map. Ultimately, you must reach the highest point upstream. This is the head of the watershed, beyond which the land slopes downward into another watershed. At every point along the stream, the land slopes up on each side to some high point then down into another watershed. By joining these high points around the stream of concern you define the watershed boundary. High points are generally hilltops, ridges, or saddles.

### D.6 Delineation of a Watershed

The following discussion is designed to help you locate and connect the high points around a watershed manually. Visualizing the landscape represented by the topographic map makes the process much easier than simply trying to follow a method by rote. If visualizing the landscape is difficult, try highlighting the internal drainage system. Start at the outlet and highlight the upstream drainage network by marking the perennial streams (solid blue lines on a topographic map) and the intermittent streams (dashed blue lines). Extend beyond the intermittent streams up each valley to the ridgeline. Remember the contours in a valley point upstream. Also highlight the channels in the headwaters of the adjacent, bordering watersheds. Once you have completed highlighting the internal drainage network in the study watershed and adjacent watersheds, the boundary for the study watershed should be obvious.

Follow the following steps to develop the final delineation of the study watershed.

1. Draw a circle, an X, or a prominent dot at the outlet or downstream point of the watershed in question.
2. Put small "X's" at the high points along both sides of the watercourse, working your way upstream towards the headwaters of the watershed.
3. Starting at the mark (circle, X or dot) that was made in step one, draw a line connecting the "X's" along one side of the watercourse. This line should always cross the contours at right angles.
4. Continue the line until it passes around the head of the watershed and down the opposite side of the watercourse. Eventually, it will connect with the mark from which you started. At this point, you have delineated the watershed contributing surface runoff to the indicated outlet.

When delineating a watershed, do not forget a most fundamental rule. Get out of your office, go to the watershed, and ground-truth what you suggest is the boundary. To ground truth means you visually inspect the entire watershed boundary. There may be drainage features present that are not indicated or properly interpretable on a topographic map. For example, a lot of roads follow what originally were the trails and pathways of the original inhabitants and early settlers. As such, many of these roads closely follow ridges. During construction, i.e., grading and paving, the roadside ditches and culvert cross-drains altered the original watershed by diverting runoff from additional areas into the watershed in question or by diverting runoff from the watershed in question into other watersheds. There have been too many legal cases because of problems created by improper watershed delineation.

### D.7 Determining Average Watershed Slope with the Grid-Overlay Method

The grid-overlay is a preferred method to determine the average watershed slope. This is a variation to a method originally developed by Robert Horton (Horton, R.E. 1932. "Drainage Basin Characteristics." Transactions, American Geophysical Union, 13:350-361), one of the fathers of modern-day hydrology.

This method involves overlaying the watershed (drainage area) with a rectangular grid, as shown in Figure C-1. Use a grid interval of 0.25-0.50 inches, preferably 0.25 inches. The elevation contours should be spaced at the same interval, $\Delta h$. Measure each horizontal grid line between its intersections with the watershed boundary, and then sum these lengths to obtain the total length ($L_H$) of horizontal grid lines contained within the watershed boundary. Do the same for the vertical grid lines to obtain $L_V$, the sum of all vertical grid lines. Next, count the number of intersections of each horizontal grid line with contour lines and sum these to obtain $N_H$. Do the same for the vertical grid lines to obtain $N_V$. Evaluate the watershed slope in the horizontal, SH, as

$$S_H = \frac{N_H \cdot \Delta h}{L_H} \tag{C.1}$$

and the average watershed slope in the vertical as

$$S_V = \frac{N_V \cdot \Delta h}{L_V} \tag{C.2}$$

The average watershed slope is evaluated as the average of the horizontal and vertical slopes.

$$S = \frac{S_H + S_V}{2} \tag{C.3}$$

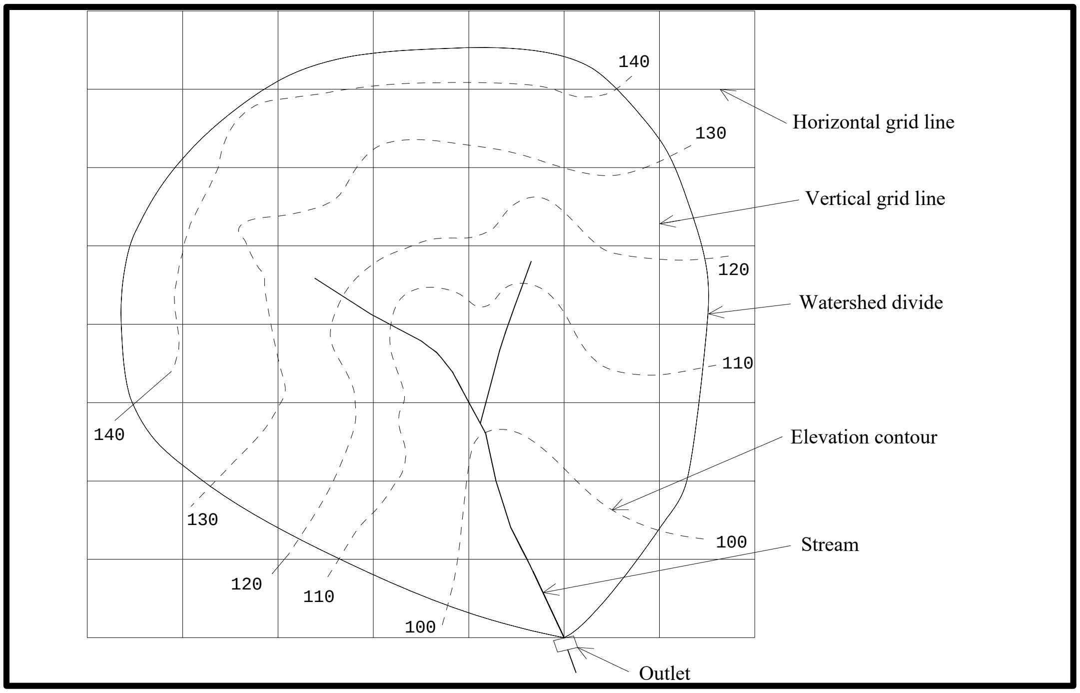



**Figure D-1. Grid Overlay Method to Determine Average Watershed Slope**

# APPENDIX E: EROSION AND SEDIMENTATION PROCESSES

Sedimentation in surface water is the end-result of several processes, including erosion, sediment production, transport and deposition, and in-stream morphological processes.

## E.1 Description of Processes

### E.1.1 Hillslope Erosion

Erosion is the wearing away of the Earth's surface by wind, water, ice, or gravity. Sediment is the mineral or organic material that is displaced by these forces and delivered to water bodies. Sedimentation is the settlement or deposition of sediment out of the water column. Erosion and sedimentation are natural processes critical to developing and maintaining stream channel form and function.

Rain splash, sheet wash, rills, and gullies associated with overland runoff account for most hillslope erosion. Other sources include mass wasting and soil creep. Mass wasting usually occurs on steep slopes that slide, or slump, when saturated soils weaken to the point of failing to hold in place against gravity. Soil creep occurs on more gentle slopes where soil particles move downslope very slowly. While these are naturally occurring processes, human activities can cause or accelerate them.

Rain splash erosion occurs when raindrops impact and displace exposed soil. Vegetation and litter cover on the ground absorb virtually all the kinetic energy of rainfall and prevent most rain splash erosion. Thus, protection of soil cover is an important strategy for minimizing this type of erosion.

Sheet erosion occurs when overland flow travels downslope in an irregular, sheet like fashion. This type of erosion occurs as tiny streams of water moving back and forth across the slope. It can transport already detached sediment as well as dislodge soil particles. Several site characteristics including soil particle size and pore space, bulk density, and organic matter content affect sheet erosion processes by influencing soil infiltration capacity. The latter three can be directly affected by management activities.

Rill erosion occurs when sheet flow cuts small, separate channels as it moves downslope. Gullies are rills greater than 1 foot wide and 1 foot deep. Exposed soil in rills and gullies is especially vulnerable to rain splash erosion, so rills and gullies can grow rapidly. Gully erosion can be dramatic, contributing large sediment loads to streams. Nevertheless, rain splash and sheet erosion generally account for over 70 percent of total hillslope erosion.

### E.1.2 Stream Channel Erosion Processes

Channel erosion can be caused by a variety of factors. Most stream channel erosion is caused by the action of instream water. Water in motion exerts fluid stress, or applied stress, on the streambed and varies with velocity. When applied stress reaches the point that bed particles begin to move, channel erosion results.

The capacity of a stream to carry sediment also increases with stream velocity. At a given flow, velocity varies within channels longitudinally and in cross section. Thus, channel erosion and sedimentation occur simultaneously. The magnitude of these processes is affected by flow rate; high flows increase channel erosion, and low flows increase sedimentation, or deposition.

Sediment size and load vary with stream discharge and stream channel slope, and all exist in a state of dynamic equilibrium. Changes in one variable lead to adjustments by one or more of the others. For example, when sediment delivery to a channel exceeds its transport capacity, sedimentation results. Conversely, reductions in sediment supply below a minimum limit deprive streamflow of sediment, and channels can erode.

### E.1.3 Hydrologic Responses

Stream equilibrium is also sensitive to hydrologic response of watersheds, especially peak flow. The most important peak flows for channel formation are associated with bank-full events. Bank-full recurs about every 1.5 years, on average. During bank-full floods, streambed material is mobile and channels experience change.

Factors affecting peak flow include the area of impervious material, soil infiltration capacity, time of concentration, drainage density, and antecedent soil moisture. Changes in any of these factors can alter peak flows.

### E.1.4 Channel Alteration

Channel straightening effectively reduces total channel length over a given elevation change, resulting in increased stream channel slope. Increases in slope frequently increase stream velocity and can cause upstream channel erosion. The effect proceeds upstream until stream slope equilibrium is re-attained.

Constrictions at stream crossings (culverts, bridges) can increase downstream velocity (result in downstream channel scour) and decrease upstream velocity (increase upstream sedimentation).

## E.2 Universal Soil Loss Equation (USLE)

USLE is the acronym for Universal Soil Loss Equation developed by USDA Agricultural Research Service (ARS) scientists W. Wischmeier and D. Smith. USLE has been the most widely accepted and utilized soil loss equation. Designed as a method to predict average annual soil loss caused by sheet and rill erosion, the USLE is often criticized for its lack of applications. While it can estimate long - term annual soil loss and guide conservationists on proper cropping, management, and conservation practices, it cannot be applied to a specific year or a specific storm. The USLE is mature technology and enhancements to it are limited by the simple equation structure.

The USLE for estimating average annual soil erosion is

$$A = RKLSCP \tag{E.1}$$

where

A = average annual soil loss in t/a (tons per acre)
R = rainfall erosivity index
K = soil erodibility factor
LS = topographic factor - L is for slope length and S is for slope
C = cropping or soil disturbance factor
P = conservation practice factor

These terms are explained in the following section.

### E.2.1 USLE Factors

R - the rainfall factor or rainfall erosivity index

Most appropriately called the erosivity index, it is a statistic calculated from the annual summation of rainfall energy in every storm (correlates with raindrop size) times its maximum 30 - minute intensity. As expected, it varies geographically.

K - the soil erodibility factor

This factor quantifies the cohesive or bonding character of a soil type and its resistance to dislodging and transport due to raindrop impact and overland flow.

LS - the topographic factor

Steeper slopes produce higher overland flow velocities. Longer slopes accumulate runoff from larger areas and result in higher flow velocities. Thus, both result in increased erosion potential, but in a non - linear manner. For convenience L and S are frequently lumped into a single term.

C - the crop management factor

This factor is the ratio of soil loss from land cropped under specified conditions to corresponding loss under tilled, continuous fallow conditions. The most computationally complicated of USLE factors, it incorporates effects of tillage management (dates and types), crops, seasonal erosivity index distribution, cropping history (rotation), and crop yield level (organic matter production potential).

P - the conservation practice factor

Practices included in this term are contouring, strip cropping (alternate crops on a given slope established on the contour), and terracing.

### E.3 Modified Universal Soil Loss Equation (MUSLE)

A shortcoming of the USLE is that it was developed to predict annual erosion and not the erosion for individual storms. To overcome this limitation, Williams (1975) modified the USLE to estimate sediment yield for a single runoff event. On the basis that runoff is a superior indicator of sediment yield than rainfall, i.e., no runoff yields no sediment and there can be rainfall with little or no runoff, Williams replaced the R (rainfall erosivity) factor with a runoff factor. His analyses revealed the product of runoff volume and peak discharge for an event yielded more accurate sediment yield predictions, especially for large events, than the USLE with the R factor.

The Modified USLE, or MUSLE, is given by the following equation:

$$S = 95\left(QQ_p\right)^{0.56} KLSCP \tag{E.2}$$

where S is the single storm event soil erosion in tons, Q is the event runoff volume in cubic feet, $Q_p$ is the peak runoff in cfs, and K, LS, C, P are USLE parameters identified and explained in Section E.2.1.

The MUSLE has been tested and found to perform satisfactorily on grassland and mixed land use watersheds. As expected, the performance of MUSLE greatly depends on the accuracy of the hydrologic inputs.

**Table E-1 Example USLE and MUSLE Erosion Estimates**

**USLE and MUSLE RESULTS**

| USLE: A = R *K LS C P* | | |
|---|---|---|
| Predicted Soil Loss | | |
| A = | 6.5 | tons/acre/year |
| Gross Erosion | | |
| A*Area = | 645.2 | tons/year |

| Factors: | |
|---|---|
| R = | 300 |
| K = | 0.24 |
| LS = | 0.25 |
| C = | 0.40 |
| P = | 0.90 |

| MUSLE: S = 95(Q*Qp )^0.56 K LS C P | | | |
|---|---|---|---|
| Erosion for Single Event D-Hour Storms | | | |
| AEP = | | 4 | % |
| Storm Duration (hours) | Runoff Volume (ac-ft) | Peak Runoff (cfs) | Gross Erosion (tons) |
| 1 | 17.21 | 107.17 | 137.8 |
| 2 | 22.39 | 127.85 | 176.2 |
| 3 | 24.37 | 128.33 | 185.2 |
| 6 | 28.62 | 133.41 | 207.1 |
| ==> 12 | 31.45 | 130.45 | 215.6 <== |
| 24 | 28.07 | 100.93 | 175.2 |

| | |
|---|---|
| Soil Type: | NEESES-TOP |
| Texture: | LS |
| Slope (%): | 2 |
| Length (feet): | 210 |
| Cover: | Jute mesh |
| Practice: | Track-walked up and down slope |

## E.4 Reference


Williams, J.R. (1 975). "Sediment-Yield Prediction with Universal Equation Using Runoff Energy Factor," in Present and Prospective Technology for Predicting Sediment Yields and Sources, ARS-S-40, USDA-ARS.

## APPENDIX F: UofSC SEDIMENT POND TRAPPING EFFICIENCY EQUATION

The UofSC TE equation is the result of a study to develop trapping efficiency curves for the analysis of sediment ponds in South Carolina. An extensive pond performance database was simulated using a modified version of the SEDIMOT II program. Over 40 different soils were selected to characterize eight textural groups: clay loam, silty clay loam, loam, sandy clay loam, sand, loamy sand, sandy loam, and silty loam. Eroded grain size distributions were generated for all soils using equations developed for the CREAMS program. These data were input to the program and simulations were performed for a range of watershed, storm, and pond characteristics. The simulated pond trapping efficiencies were correlated with various sediment, hydrograph and pond parameters, and regression equations developed that predict trapping efficiency in terms of dimensionless parameters for particle size gradation, pond overflow rate, peak flow reduction (pond detention storage) and pond retention storage.

### F.1 Introduction

Traditional approaches to stormwater management have considered only the rates and volumes of runoff, and not water quality parameters such as sediment load. In cases of land disturbing activities, modern regulations require comprehensive plans for erosion and sediment control, in addition to stormwater control. Because of the complexity of the calculations, most design engineers use computer simulation programs such as SEDIMOT II (Wilson, et al., 1982) and SEDCAD (Warner and Schwab, 1992). It generally is acknowledged this approach is appropriate and required for large sites and sensitive areas. At small sites where channel erosion does not contribute significantly to the total sediment load and where a pond is to be used to trap sediment and regulate the outflow hydrograph, a simple desktop method for determining pond performance can be a cost-effective alternative. The results of a study (Meadows and Kollitz, 1994) to develop trapping efficiency curves (equations) for the analysis of sediment ponds in South Carolina are presented. That study was conducted in parallel with, and as a beta test of the SCDHEC Design Aids developed by Hayes and Barfield (1993) and to provide an alternate design aid.

### F.2 UofSC Approach

An extensive pond performance database was simulated using a modified version of the SEDIMOT II program. Over 40 different soils were selected to characterize eight textural groups: clay loam, silty clay loam, loam, sandy clay loam, sand, loamy sand, sandy loam, and silty loam. Eroded grain size distribution curves were generated for each soil using equations developed for the CREAMS program (Foster, et al., 1985). These data were input to the program and simulations were performed for a range of watershed, storm, and pond characteristics. The simulated pond trapping efficiencies were correlated with various sediment, hydrograph and pond parameters, and regression equations developed that predict trapping efficiency in terms of dimensionless parameters for particle size gradation, pond overflow rate, peak flow reduction (pond detention storage) and pond retention storage.

### F.3 Soil Classification and Selection

Primary particle compositions for 386 soils found in South Carolina were obtained from the Soil Conservation Service. Each soil was classified according to the U.S. Department of Agriculture triangular soil classification chart (textural triangle). Four textural groups, clay, silty clay loam, sandy clay, and silt, were represented by 5 soils or less and were omitted from the analysis.

The position of each soil was plotted on the textural triangle and representative soils were selected for each group from those that fell around the perimeter of the textural region. Those soils selected for study and their position on the textural triangle are shown in Figure F-1.

### F.4 Simulation Parameters

To obtain a large and representative data set, simulations were performed for each of the 43 soils on multiple watershed and sediment pond configurations. Each contributing watershed was modeled as a rectangular plane discharging to a lateral drainage ditch. Values for watershed parameters were varied as follows: area (3, 10, and 20 acres); length to width ratio (1, 2, 3, 4, 5, and 10); overland slope (0.5, 2.5 and 6%); channel slope (0.5, 1, and 3%); curve number (75 and 85), and unit hydrograph peak rate factor (325, 484 and 550). Watershed time of concentration was estimated with the NRCS travel time method.

Three basic pond and riser configurations were used. The ponds were modeled with a rectangular base and 3:1 side-slopes. Surface areas at the base were chosen as 0.04, 0.12 and 0.30 acres. A unique riser was associated with each pond; diameters were fixed at 36, 48 and 54 inches. Pond length to width ratio was varied as 1, 2, 3, and 5.

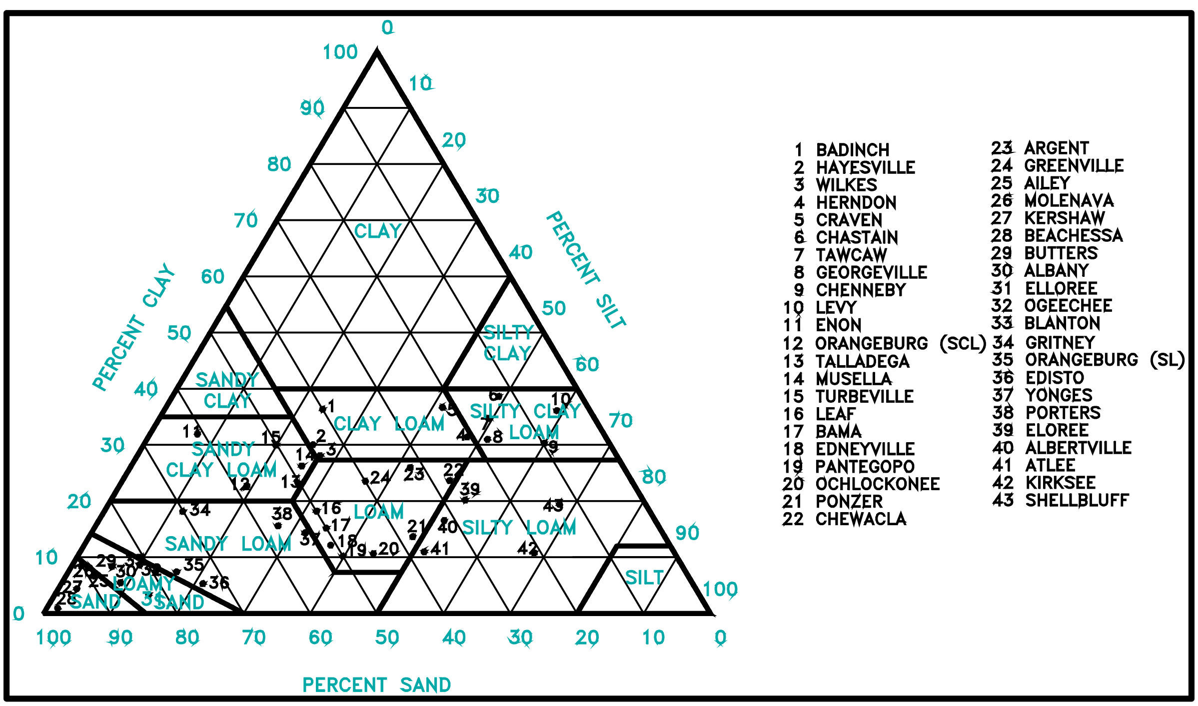



**Figure F-1. Position of Selected Soils on Textural Triangle**

### F.5 Dimensionless Parameters

A set of dimensionless parameters was sought to explain the variation in the simulated pond trapping efficiencies in terms of sediment, hydrograph, and pond characteristics. The following parameters were found to correlate strongly with the trapping efficiency data.

D* ($D_{85}/D_{15}$) - A dimensionless parameter which characterizes the eroded particle size distribution. $D_{85}$ and $D_{15}$ are the particle diameters for which 85% and 15%, respectively, of the sediment by weight are finer.

F* ($Q_{po}/A_oV_{15}$) - A dimensionless parameter that is the reciprocal of the pond overflow rate equation (Haan, et al., 1994). $Q_{po}$ is the peak outflow rate from the pond ($m^3/s$), determined by routing the inflow hydrograph through the pond; $A_o$ is the surface area of the pond ($m^2$) at the crest of the outflow riser, $V_{15}$ the settling velocity (m/s) of the $D_{15}$ particle.

q* ($Q_{po}/i_p$) - A dimensionless pond performance parameter. This parameter characterizes the pond detention routing effects to reduce the outflow peak rate. $Q_{po}$ is the peak outflow from the pond and $i_p$ is the peak inflow.

S* ($V_{ret}/V_{runoff}$) - A dimensionless storage parameter which divides the volume of water retained below the crest of the riser ($V_{ret}$) by the watershed runoff volume ($V_{runoff}$). When evaluating this parameter, express volume in $m^3$.

Statistical analysis revealed these four parameters are strongly correlated to trapping efficiency, but that q* and S* are collinear. This presented a problem because estimates determined using collinear parameters might have high standard errors. To remove the collinearity, yet maintain the information contained in q* and F*, a new parameter Q* was formed as the product (Q* = q*F*). Subsequent statistical analyses to determine prediction equations for trapping efficiency used the reduced parameter set D*, S* and Q*.

### F.6 Trapping Efficiency Curves

Various regression models were tested to determine which provided the best fit of the trapping efficiency data to the dimensionless parameter data. It was found a linear model gave the most favorable statistical fit. The general form of the equation is:

$$TE = a + bS^* + cD^* + dQ^* \qquad \text{(F.1)}$$

in which TE is pond trapping efficiency in percent, S*, D* and Q* are defined previously, and a, b, c, and d are soil texture specific coefficients determined by regression.

The coefficients for (1) were determined for each of the eight soil textural groups using multiple linear regression. The results are summarized in Table F-1. Also shown in this table are the standard deviation, $r^2$ statistic, and the number of data points (NOBS). As indicated by the $r^2$ statistic, the regression equations fit the data for all textural groups very well, with the possible exception being sand. Note, however, the intercept for sand is very large, indicating sand has a high trapping efficiency.

The results in Table F-1 are consistent among the different textural groups except Silty Clay Loam (SiCL), which reversed the sign of the c-value and had the lowest a-value. No explanation was found.

**Table F-1**
**Regression Equation Coefficients and Statistics**

| Soil Type[1] | a | b | c | d | Std. Dev. | $r^2$ | NOBS |
|---|---|---|---|---|---|---|---|
| CL | 88.60 | 23.78 | -0.12 | -0.40 | 1.23 | 0.98 | 17275 |
| SiCL | 54.23 | 23.64 | 0.14 | -0.28 | 1.24 | 0.99 | 17275 |
| SCL | 89.56 | 13.93 | -0.08 | -0.70 | 0.75 | 0.98 | 17275 |
| L | 88.53 | 19.99 | -0.11 | -0.74 | 1.77 | 0.95 | 22164 |
| S | 98.75 | 2.56 | -0.16 | -0.70 | 0.53 | 0.87 | 13822 |
| LS | 94.63 | 6.22 | -0.04 | -2.15 | 0.46 | 0.96 | 17275 |
| SL | 91.84 | 11.52 | -0.08 | -1.29 | 0.69 | 0.98 | 17277 |
| SiL | 79.34 | 23.54 | -0.01 | -0.77 | 1.46 | 0.98 | 17279 |

[1] CL=Clay Loam, SiCL=Silty Clay Loam, SCL=Sandy Clay Loam, L=Loam, S=Sand, LS=Loamy Sand, SL=Sandy Loam, SiL=Silty Loam

## F.7 References

Foster, G.R., Young, R.A., and Neibling, W.H. (1985). "Sediment composition for non-point source pollution analyses." Trans ASAE 28(1), 133-146.

Haan, C.T., Barfield, B.J., and Hayes, J.C. (1994). Design hydrology and sedimentology for small catchments. Academic Press, San Diego, CA.

Hayes, J.C. and Barfield. B.J. (1993). "Engineering Aids and Design Guidelines for Control of Sediment in South Carolina," SCDHEC, Environmental Quality Control.

Meadows, M.E. and Kollitz, M.M. (1995) "Sediment Pond Trapping Efficiency Curves," Proc., ASCE First International Conference on Water Resources Engineering, San Antonio, TX, pp. 1258-1262.

Warner, R.C. and Schwab, P.J. (1992). "SEDCAD+ Version 3 training manual," Civil Software Design, Ames, IA.

Wilson, B.N, Barfield, B.J., and Moore, I.D. (1982). "A hydrology and sedimentology watershed model. I. Modeling technique," Department of Agricultural Engineering, University of Kentucky, Lexington, KY.

# APPENDIX G: STORMWATER PONDS

## G.1 Introduction

Land use changes to a plot of land adds to its history of uses such as natural woods or open space developed into a residential neighborhood, a business, or an industrial center. Such changes involve roads and streets that alter the direction and capacity of flow pathways which impacts runoff peak flow rates, volumes, and timing. Changes in land use affect how the land responds to rainfall. Increased impervious area, compaction of the natural soils, filling of existing depressions, and introduction of pollutants result in less infiltration, more runoff, higher rates of runoff, reduction in storage capacity, and degradation of stormwater quality. Stormwater ponds are implemented to mitigate these changes by providing storage, attenuating peak discharge rates, and improving water quality.

Some stormwater ponds hold water while others are dry. This difference is not simply based on the length of time from the last storm event. Some ponds are designed to drain completely soon after a storm; these are detention ponds. Some ponds are designed to hold a portion of the stormwater for an extended time, while discharging the extra runoff to downstream channels; these are retention-detention ponds. Other stormwater ponds are designed to hold all the runoff from a rainfall event. These are retention ponds and recover storage volume through infiltration and evaporation. Purposes for stormwater ponds are to provide storage of stormwater runoff, either temporary or long-term, attenuate peak runoff rates, and effectively treat the stormwater to improve water quality.

## G.2 General Design Procedures

This section discusses the general design procedures for designing stormwater detention ponds. The design procedures for all storage facilities are the same even if they include a permanent pool of water. In the latter case, the permanent pool elevation is taken as the “bottom” of storage and is treated as if it were a solid basin bottom for routing purposes.

### G.2.1 Data Needs

The following data are needed for stormwater pond design and routing calculations:

- inflow hydrographs for all design storms,
- stage-storage curve for the proposed storage facility, and
- stage-discharge curve(s) for all outlet control structures.

### G.2.2 Computational Steps

1. Compute inflow hydrograph(s) for the design storm(s) for specified land use conditions, which should include pre-land use change conditions, construction phase, and post-development land uses.

2. Perform preliminary calculations to evaluate detention storage requirements for the hydrographs from Step 1.

3. Determine the physical basin dimensions necessary to hold the volumes determined in Step 2, including freeboard. The maximum storage requirement calculated from Step 2 should be used. For the selected basin shape, determine the maximum depth in the pond.

4. Select the type of outlet(s) and size each outlet structure. The outlet type and size will depend on the type of basin (detention, extended detention or retention) as well as the allowable discharge. The estimated peak stage will occur for the estimated volume from Step 2. The outlet structure(s) should be sized to convey the allowable discharge at this stage.

5. Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using a storage routing computer model

6. If the routed construction phase and post-development peak discharges exceed the existing conditions peak discharges, then revise the available storage volume, outlet device(s), etc., and return to Step 3 until the basin size, basin depth, outlet type, and outlet size meet the allowable discharge requirements.

7. Evaluate the control structure outlet velocity and provide channel and bank stabilization if the velocity will cause erosion problems downstream.

The location of a storage facility can have a sizeable impact on the effectiveness of such facilities to control downstream impacts. In addition, multiple storage facilities located in the same drainage basin will affect the timing of the runoff through the downstream conveyance system, which could decrease or increase flood peaks in different downstream locations. A downstream peak flow analysis should be performed as part of the storage facility design process

### G.3 Detention Pond Stage-Storage Formulas

One geometry used to represent a trial stormwater detention pond is the inverted quadrilateral frustum, i.e., one with rectangular base ($L_o$ x $W_o$) and trapezoidal side slope (z) in both directions. As such, the length and width dimensions at any elevation, h, are given as

$$\begin{aligned} L(h) &= L_o + 2zh \\ W(h) &= W_o + 2zh \end{aligned} \tag{G.1}$$

The horizontal area at any elevation, h, is given as the product

$$A(h) = L(h)\cdot W(h) = L_o W_o + 2zh(L_o + W_o) + 4z^2h^2 \tag{G.2}$$

The storage at any elevation, h, can be found as the integral

$$\begin{aligned} S(h) &= \int_0^h \left(L_o W_o + 2zh(L_o + W_o) + 4z^2h^2\right)\cdot dh \\ &= L_o W_o h + zh^2(L_o + W_o) + \frac{4}{3}z^2h^3 \end{aligned} \tag{G.3}$$

Another analytical solution can be developed from the formula for the volume of a quadrilateral frustum as found in the CRC Math Tables. Let $A_1$ = area of lower base, $A_2$ = area of upper base, h = vertical height, and V = volume.

$$V = \frac{h}{3}\left(A_1 + A_2 + \sqrt{A_1 A_2}\right) \tag{G.4}$$

This relationship is used to develop a general formula for storage at any elevation h, where $S(h)=V(h)$, $A_2=A_h=A(h>0.0)$ and $A_1=A(h=0.0)$.

$$S(h) = \frac{h}{3}\left(A_1 + A_h + \sqrt{A_1 A_h}\right) \tag{G.5}$$

A third and approximate solution is obtained by computing the incremental storage between any two elevations, h and h+Δh, and then accumulating these over the entire depth of ponding.

$$A(h) = L(h)\cdot W(h) = L_o W_o + 2zh(L_o + W_o) + 4z^2h^2 \tag{G.6}$$

$$A(h+\Delta h) = L(h+\Delta h)\cdot W(h+\Delta h) = L_o W_o + 2z(h+\Delta h)(L_o + W_o) + 4z^2(h+\Delta h)^2 \tag{G.7}$$

$$\Delta S = \frac{A(h) + A(h+\Delta h)}{2}\Delta h \tag{G.8}$$

$$S(h) = \sum_{h=0}^{h} \Delta S_i \tag{G.9}$$

The following example compares results obtained using the three formulas G.3, G.5, and G.9.

Example: Evaluate Trial Detention Pond Storage with Different Formulas

Given: A trial detention pond has base length = 100-ft, base width=80-ft and side slope=3.

Find: Evaluate the pond storage to a depth of 6 feet.

Sol'n: The following worktable summarizes the calculations and pond storage determined with Eqns. G.3, G.5, and G.9. Note the differences are minor.

Detention Pond Storage Equations

| | | | | | Storage, cu ft | | |
|---|---|---|---|---|---|---|---|
| h, ft | L, ft | W, ft | A, sq ft | ΔS, cu ft | Eq. G.3 | Eq. G.5 | Eq. G.9 |
| 0 | 100 | 80 | 8,000 | – | 0 | 0 | 0 |
| 1 | 106 | 86 | 9,116 | 8,558 | 8,552 | 8,552 | 8,558 |
| 2 | 112 | 92 | 10,304 | 9,710 | 18,256 | 18,255 | 18,268 |
| 3 | 118 | 98 | 11,564 | 10,934 | 29,184 | 29,182 | 29,202 |
| 4 | 124 | 104 | 12,896 | 12,230 | 41,408 | 41,404 | 41,432 |
| 5 | 130 | 110 | 14,300 | 13,598 | 55,000 | 54,993 | 55,030 |
| 6 | 136 | 116 | 15,776 | 15,038 | 70,032 | 70,020 | 70,068 |

### G.4 Stage-Discharge Relationship

A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility. A typical storage facility has two outlets or spillways: a principal outlet and a secondary (or emergency) outlet. The principal outlet is usually designed with a capacity to convey the design flows without allowing flow to enter the emergency spillway. A pipe culvert, weir, or other appropriate outlet can be used for the principal spillway or outlet.

The emergency spillway is sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal outlet. This spillway should be designed to account for the potential threat to downstream areas if the storage facility were to fail. The stage-discharge curve should account for the discharge characteristics of both the principal spillway and the emergency spillway. Development of a stage-discharge curve (table) is included in Example F.2.

### G.5. Pond Routing with the Modified Puls Routing Model

The purpose of this example is to illustrate the steps involved in routing a hydrograph through a detention pond. Functionally, these are steps involved in the design of a stormwater detention pond. For this example, the computational time interval is 10 minutes. Consistent with current practice, the rainfall burst duration and the computational time interval are 6 minutes. For this example, 10 minutes are used to give readers experience with different time intervals, which happens when using different methods and models.

Given: Inflow hydrograph and pond stage-storage-discharge data as summarized in the following tables.

Inflow Hydrograph

| Time, min | 0 | 10 | 20 | 30 | 40 | 50 | 60 | 70 | 80 | 90 | 100 | 110 | 120 | 130 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Q, cfs | 0.0 | 1.0 | 3.5 | 6.5 | 8.9 | 10.9 | 13.6 | 14.7 | 13.6 | 8.7 | 5.7 | 3.0 | 0.9 | 0.0 |

Pond Stage-Storage-Discharge Data

| Stage | Q-orifice | Q-weir | Q-total | Length, ft | Width, ft | Area | Storage, cu ft |
|---|---|---|---|---|---|---|---|
| 0 | 0.00 | 0.00 | 0.00 | 30 | 20 | 600 | 0 |
| 1 | 3.78 | 0.00 | 3.78 | 36 | 26 | 936 | 768 |
| 2 | 5.35 | 0.00 | 5.35 | 42 | 32 | 1,344 | 1,908 |
| 3 | 6.55 | 0.00 | 6.55 | 48 | 38 | 1,824 | 3,492 |
| 4 | 7.56 | 0.00 | 7.56 | 54 | 44 | 2,376 | 5,592 |
| 5 | 8.46 | 0.00 | 8.46 | 60 | 50 | 3,000 | 8,280 |
| 6 | 9.26 | 0.00 | 9.26 | 66 | 56 | 3,696 | 11,628 |
| 7 | 10.01 | 0.00 | 10.01 | 72 | 62 | 4,464 | 15,708 |
| 8 | 10.70 | 1.32 | 13.02 | 78 | 68 | 5,304 | 20,592 |

Find: Pond outflow hydrograph

Sol'n: Apply the modified Puls routing method.

$$\frac{2S_2}{\Delta t} + Q_2 = \frac{2S_1}{\Delta t} - Q_1 + I_1 + I_2$$

First, develop the $\frac{2S}{\Delta t} + Q$ rating curve. Use $\Delta t = 10 \text{ min}$ consistent with the interval between inflow hydrograph ordinates.

| Stage | Q-total | Storage, cu ft | $\frac{2S}{\Delta t} + Q$ |
|---|---|---|---|
| 0.0 | 0.00 | 0 | 0.00 |
| 1.0 | 3.78 | 768 | 6.34 |
| 3.0 | 5.35 | 1,908 | 11.71 |
| 3.0 | 6.55 | 3,492 | 18.19 |
| 4.0 | 7.56 | 5,592 | 26.20 |
| 5.0 | 8.46 | 8,280 | 36.06 |
| 6.0 | 9.26 | 11,628 | 48.02 |
| 7.0 | 10.01 | 15,708 | 63.37 |
| 8.0 | 13.02 | 20,592 | 80.76 |

Following are copies of the EXCEL spreadsheets used to solve this problem including convolution of the unit hydrograph with the rainfall excess, development of the stage-storage and stage-outflow curves, and pond routing with the Modified Puls Method.

First, determine the runoff hydrograph by convoluting the rainfall excess distribution with the unit hydrograph. There are six bursts, which will result in six burst hydrographs. The first burst hydrograph begins at the start of the storm, i.e. with the beginning of burst 1 excess rainfall. The second burst hydrograph starts with the beginning of burst 2, i.e. it lags the start of burst 1 by DT minutes. Successive burst hydrographs lag the preceding burst hydrograph by DT minutes. As such, the hydrograph for burst n lags the start of the first burst by (n-1)*DT. Note the Lag 1 through Lag 5 notations in the computational table below.

The runoff hydrograph duration will equal unit hydrograph duration plus (n-1)*DT where n is the number of rainfall bursts and DT is the computational time interval, equal to 10 minutes for this example. The UH duration is 80 minutes. The runoff hydrograph duration is 80 + (5-1)*10 = 130 minutes.

| **Burst Excess Rain =** | | **0.10** | **0.15** | **0.05** | **0.10** | **0.20** | **0.15** | **Summation = Runoff Hydrograph** |
|---|---|---|---|---|---|---|---|---|
| **Time, min** | **UH, cfs** | **Burst 1** | **Burst 2** | **Burst 3** | **Burst 4** | **Burst 5** | **Burst 6** | |
| 0 | 0 | 0.0 | **Lag 1** | | | | | 0.0 |
| 10 | 10 | 1.0 | 0.0 | **Lag 2** | | | | |
| 20 | 20 | 2.0 | 1.5 | 0.0 | **Lag 3** | | | 3.5 |
| 30 | 30 | 3.0 | 3.0 | 0.5 | 0.0 | **Lag 4** | | 6.5 |
| 40 | 24 | 3.4 | 4.5 | 1.0 | 1.0 | 0.0 | **Lag 5** | 8.9 |
| 50 | 18 | 1.8 | 3.6 | 1.5 | 2.0 | 2.0 | 0.0 | 10.9 |
| 60 | 12 | 1.2 | 3.7 | 1.2 | 3.0 | 4.0 | 1.5 | 13.6 |
| 70 | 6 | 0.6 | 1.8 | 0.9 | 3.4 | 6.0 | 3.0 | 14.7 |
| 80 | 0 | 0.0 | 0.9 | 0.6 | 1.8 | 4.8 | 4.5 | 13.6 |
| 90 | | | 0.0 | 0.3 | 1.2 | 3.6 | 3.6 | 8.7 |
| 100 | | | | 0.0 | 0.6 | 3.4 | 3.7 | 5.7 |
| 110 | | | | | 0.0 | 1.2 | 1.8 | 3.0 |
| 120 | | | | | | 0.0 | 0.9 | 0.9 |
| 130 | | | | | | | 0.0 | 0.0 |

Next, determine the pond stage-storage table. Pond geometry is an inverted quadrilateral frustum with base length and slope of 30 and 20 feet and side slope 3 horizontal to 1 vertical.

| **h, ft** | **L, ft** | **W, ft** | **A, sq ft** | **ΔS, cu ft** | **S, cu ft** |
|---|---|---|---|---|---|
| 0 | 30 | 20 | 600 | 0 | 0 |
| 1 | 36 | 26 | 936 | 768 | 768 |
| 2 | 42 | 32 | 1,344 | 1,140 | 1,908 |
| 3 | 48 | 38 | 1,824 | 1,584 | 3,492 |
| 4 | 54 | 44 | 2,376 | 2,100 | 5,592 |
| 5 | 60 | 50 | 3,000 | 2,688 | 8,280 |
| 6 | 66 | 56 | 3,696 | 3,348 | 11,628 |
| 7 | 72 | 62 | 4,464 | 4,080 | 15,708 |
| 8 | 78 | 68 | 5,304 | 4,884 | 20,592 |

Develop the pond stage-outflow table. The pond has a first stage 12-inch orifice located at the bottom of the riser and a second stage rectangular weir with opening width or 0.4 feet and weir crest elevation at 7 feet.

| h, ft | h-orifice, ft | Orifice | h-weir, ft | Weir | Outflow |
|---|---|---|---|---|---|
| 0 | 0 | 0.00 | 0.00 | 0.00 | 0.00 |
| 1 | 1 | 3.78 | 0.00 | 0.00 | 3.78 |
| 2 | 2 | 5.35 | 0.00 | 0.00 | 5.35 |
| 3 | 3 | 6.55 | 0.00 | 0.00 | 6.55 |
| 4 | 4 | 7.56 | 0.00 | 0.00 | 7.56 |
| 5 | 5 | 8.46 | 0.00 | 0.00 | 8.46 |
| 6 | 6 | 9.26 | 0.00 | 0.00 | 9.26 |
| 7 | 7 | 10.01 | 0.00 | 0.00 | 10.01 |
| 8 | 8 | 10.70 | 1.00 | 1.32 | 12.02 |

Use the pond stage-storage and stage-outflow data to develop a storage indication curve, i.e., a relationship between outflow and $2S/\Delta t + Q$. Use this as an interpolation table during pond routing to solve for the outflow at the end of each $\Delta t$.

| Stage | Q-total | Storage cu ft | $\frac{2S}{\Delta t}+Q$ |
|---|---|---|---|
| 0.0 | 0.00 | 0 | 0.00 |
| 1.0 | 3.78 | 768 | 6.34 |
| 2.0 | 5.35 | 1,908 | 11.71 |
| 3.0 | 6.55 | 3,492 | 18.19 |
| 4.0 | 7.56 | 5,592 | 26.20 |
| 5.0 | 8.46 | 8,280 | 36.06 |
| 6.0 | 9.26 | 11,628 | 48.02 |
| 7.0 | 10.01 | 15,708 | 62.37 |
| 8.0 | 12.02 | 20,592 | 80.76 |

Last step: perform pond routing. Develop and use a computational table like the following.

| Time min | Inflow cfs | $I_1+I_2$ | $2S_1/\Delta t\text{-}Q_1$ | $2S_2/\Delta t+Q_2$ | $Y_2$ | $Q_2$ |
|---|---|---|---|---|---|---|
| 0 | 0.0 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 10 | 1.0 | 1.00 | 0.00 | 1.00 | 0.16 | 0.60 |
| 20 | 3.5 | 4.50 | -0.19 | 4.31 | 0.68 | 2.57 |
| 30 | 6.5 | 10.00 | -0.83 | 9.17 | 1.53 | 4.61 |
| 40 | 8.9 | 15.40 | -0.04 | 15.36 | 2.56 | 6.03 |
| 50 | 10.9 | 19.80 | 3.31 | 23.11 | 3.61 | 7.17 |
| 60 | 13.6 | 24.50 | 8.77 | 33.27 | 4.72 | 8.20 |
| 70 | 14.7 | 28.30 | 16.86 | 45.16 | 5.76 | 9.07 |
| 80 | 13.6 | 27.30 | 27.02 | 55.32 | 6.51 | 9.64 |
| 90 | 8.7 | 21.30 | 36.04 | 58.34 | 6.72 | 9.80 |
| 100 | 5.7 | 14.40 | 38.74 | 53.14 | 6.36 | 9.53 |
| 110 | 3.0 | 8.70 | 34.08 | 42.78 | 5.56 | 8.91 |
| 120 | 0.9 | 3.90 | 24.96 | 28.86 | 4.27 | 7.80 |
| 130 | 0.0 | 0.90 | 13.26 | 14.16 | 2.38 | 5.80 |
| 140 | 0.0 | 0.00 | 2.55 | 2.55 | 0.40 | 1.52 |

## G.6 References

Cunge, J.A. (1969). "On the Subject of a Flood Propagation Computation Method (Muskingum Method)," J. Hydraulic Research, Vol. 7, No. 2, pp. 95-101.

McCarthy, G.T. (1938). The Unit Hydrograph and Flood Routing, Presented at the Conference of the North Atlantic Division, U.S. Army Corps of Engineers.

McCuen, R.H. (1998). Hydrologic Analysis and Design, 2nd Ed., Prentice Hall, Englewood Cliffs, NJ.

Meadows, M.E. (1981). "Modeling the Impact of Stormwater Runoff," Proceedings, International Symposium on Urban Hydrology, Hydraulics, and Sediment Control, University of Kentucky, Lexington, KY, pp. 316-319.

Stevenson, D. and Meadows, M.E. (1986). Kinematic Hydrology and Modelling, Elsevier Science Publishers, Amsterdam, The Netherlands.