Imperial County
Hydrology Manual

OCTOBER 2018

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Imperial County

Hydrology Manual

October 2018

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Abbreviations

A ..... drainage area
ac ..... acre(s)
ARC ..... antecedent runoff condition
C ..... runoff coefficient
CN ..... runoff curve number
CN_C ..... composite runoff curve number
CN_P ..... pervious runoff curve number
C_p ..... pervious runoff coefficient
cfs ..... cubic feet per second
D ..... period (duration) of excess rainfall
DARF ..... depth-area-reduction factor
DEM ..... digital elevation model
DU/A ..... dwelling units per acre
FAA ..... Federal Aviation Administration
FHWA ..... Federal Highway Administration
ft ..... foot, feet
GIS ..... geographic information system
HEC ..... Hydrologic Engineering Center
HMS ..... Hydrologic Modeling System
hr ..... hour(s)
I ..... rainfall intensity
IDF ..... intensity-depth-frequency
IID ..... Imperial Irrigation District
in ..... inch(es)
k ..... intercept coefficient
L ..... flow length
L_C ..... length along watercourse to location nearest to centroid
L_M ..... maximum overland sheet flow length
m ..... empirical coefficient
mi ..... mile(s)
MRM ..... Modified Rational Method
n ..... Manning's roughness coefficient
$\bar{n}$ ..... mean basin Manning's roughness coefficient
NEH ..... National Engineering Handbook
NOAA ..... National Oceanic and Atmospheric Administration
NRCS ..... Natural Resources Conservation Service
P ..... total rainfall
P_{imp} ..... percent imperviousness
PFDS ..... Precipitation Frequency Data Server
Q ..... discharge
Q_p ..... peak discharge
RM ..... Rational Method
R ..... hydraulic radius
R ..... ratio of unconnected impervious area
R_{nD} ..... depth-area adjusted rainfall amount for duration (n x D)
S ..... slope
sec ..... second(s)
T_c ..... time of concentration

T_i.. initial time of concentration
T_l... Corps lag time
T_N ....NRCS lag time
T_p..time to peak
T_t ....travel time
USACE ....United States Army Corps of Engineers
USDA..United States Department of Agriculture
V...velocity

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1 Introduction

1.1 About This Manual

The goal of Imperial County, California, is to provide flood protection for all habitable structures and other non-floodproofed structures, consistent with Imperial County ordinances and design criteria. This manual is intended to provide guidance and recommendations on computational techniques and criteria for the estimation of runoff, discharges, and volumes for use in hydrology study submittals to the County. It is not a substitute for sound engineering judgment. This document is not intended to provide guidelines for the design of drainage structures, but rather the estimated flows to be used in the design of such structures. For guidance with sizing and designing hydraulic structures (e.g., detention basins, storm drains, curb and gutter), consult Imperial County Department of Public Works for the latest design criteria.

The County's Engineering Design Guidelines Manual provides specific recommendations for retention/detention basin sizing including the minimum precipitation depth to consider for the 100-year storm and requirements for drain time. If any proposed development drains to an Imperial Irrigation District (IID) facility, the design will need to meet IID standards and is subject to IID review/approval.

It is not the intent nor purpose of this manual to inhibit sound innovative design or the use of new techniques. Therefore, where special conditions or needs exist, other methods and procedures may be used *with prior approval.*

1.2 Manual Organization

In general, each main section is laid out following a similar format:

- General Description: This segment provides a brief overview of the topics covered in the section.
- Subsection(s): Each section contains sub-sections of main concepts relevant to the larger section. The sub-sections explain the techniques or concepts necessary to perform the desired task or use a certain hydrologic method.
- Instructions: When applicable, procedures to perform detailed calculations are provided.

- Online Resources: Online data resources have website links. In addition, a description including the website owner and data type is provided in case the web page should be moved by the owner.

- Tables and Figures: Related tables and figures are generally located immediately adjacent to the text to which the Table or Figure refers.

- Examples: Example problems demonstrating the use of methods described in a section are located at the end of the section.

- Equations: Equations utilized in a section are numbered according to the section number and order of appearance of the equation in the section.

- Related Equations: Previously defined equations related to a topic of discussion are referenced by the equation number.

1.3 Hydrologic Procedure Guidance

The choice of hydrologic method should be dictated by the intended use of the result. The Rational Method was originally developed to estimate runoff from small areas and assumes a uniform distribution of precipitation over the study area. This is a major reason the Rational Method is applicable only when areas are less than or equal to 640 acres (1.0 square mile). The Rational Method should not be used in circumstances where there is a junction of independent drainage systems. In these instances, the Modified Rational Method should be used to analyze the junction(s) of the independent drainage systems. The Natural Resources Conservation Service (NRCS) Hydrologic Method should be used for watersheds greater than approximately 640 acres (1.0 square mile) in size.

1.4 Acknowledgments

This hydrology manual was prepared by WEST Consultants, Inc. (WEST) on behalf of the Imperial County Department of Public Works. As part of the WEST team, Hromadka and Associates, Inc. provided quality control reviews and content recommendations. Review comments were incorporated based on input from the Imperial County Department of Public Works and the local engineering community.

2 Precipitation Analysis

2.1 General Description

Imperial County is within the Sonoran and Colorado Desert region with high temperatures and an average annual rainfall of 3 inches (U.S. Climate Data, 2016). Storms in Imperial County can be classified as general storms and local storms (Caltrans, 2007).

General storms are usually frontal or convergence storms that typically move in from the Pacific Ocean and produce light rain over relatively large areas. These storms normally occur between October and May, with most occurring in January and February. Although not as common, general storms that occur in the summer are often tropical storms. Typically, the mountain areas receive more precipitation than the lower desert areas.

While general storms bring a large volume of water over time, local storms are small and intense, producing higher peak rainfall amounts. Local thunderstorms can occur in Imperial County at any time of the year but are most common and most intense during the summer months (June to September). They develop as warm, moist tropical air drifts northward and northwestward from Mexico and the Gulf of California, and are sometimes enhanced by moisture and atmospheric circulation drifting northward from tropical storms off the west coast of Baja California. These local thunderstorms can produce very heavy rain for short periods of time over small areas, causing very rapid runoff from small drainages. The result may be flash floods, which can lead to loss of life and substantial property damage. A significant percentage of the largest runoff is likely caused by summer thunderstorms over small basins with drainage areas generally less than 20 square miles.

Because both general storms and local thunderstorms may cause significant runoff in Imperial County, both the 6-hour design storm and the 24-hour design storm should be analyzed when applying the NRCS method (Chapter 4). The design storm that produces the largest peak discharge (or volume, when appropriate) should be selected for use in the runoff calculation.

When applying the Rational Method, the storm duration for the rainfall intensity parameter will be equal to the time of concentration (T_c) (Chapter 3).

This chapter provides guidance for estimating the rainfall intensity for use with the Rational Method when the watershed is less than 640 acres (1.0 square mile) and the NRCS Method when the watershed is larger than 640 acres (1.0 square mile).

2.2 Rainfall Depth and Intensity

Rainfall depth and intensity at a point can be obtained using the National Oceanic and Atmospheric Administration (NOAA) online Precipitation Frequency Data Server (PFDS): http://hdsc.nws.noaa.gov/hdsc/pfds/. NOAA Atlas 14, Volume 6, encompasses Imperial County and was updated in 2011. The NOAA Atlas 14 online tool uses an interactive map or user provided latitude/longitude, once the state has been selected. The required return interval will be dictated by the project.

An assumption of the Rational Method is equal intensity rainfall over the entire drainage basin. For this reason, when using NOAA Atlas 14 for the Rational Method, multiple points within the watershed should be evaluated and the highest value used.

The NRCS Method, for larger areas, requires an average rainfall over the entire watershed. The recommended method to obtain the average precipitation over the watershed is to use GIS software. The PFDS (link above) provides gridded rainfall estimates under the "Supplementary information" tab. Once the recurrence interval and duration are selected, the gridded data can be downloaded. The data will cover the Southwestern United States, which includes Imperial County. Average rainfall can be determined using a georeferenced shapefile of the watershed.

2.3 Depth-Area Reduction Factors

Convective storms are not uniformly distributed in space, typically having a higher rainfall intensity at the storm center and decreasing intensity toward the storm edge. Similarly, general storms tend to have rainfall depths that vary through the spatial extent of the storm.

Rainfall values are selected from point depth duration frequency curves in standard resources like NOAA Atlas 14 as described in Section 2.2. This is the expected rainfall depth at one location in a watershed for the specified duration and frequency. Because storms are not uniformly distributed in space, point rainfall is typically higher than aerially-averaged rainfall depths. Depth Area Reduction Factors (DARFs) are used to convert point precipitation values of a given recurrence interval to an area average precipitation value of the same recurrence.

DARFs are represented by a set of curves relating the DARF to watershed area and return interval. The DARF curves for Imperial County are presented in Figure 2-1 and Figure 2-2 (NOAA, 1973). DARF values range from 0 to 1.0 (shown as 0 to 100 percent on the figures) and reduce the point value to an average areal estimate. After watershed rainfall depth has been determined for the appropriate return interval, the rainfall depth should be reduced using the DARF value from Figure 2-1, Figure 2-2 or Table 2-1 corresponding to the watershed size and rainfall duration. If the watershed size is not represented in Table 2-1, select the next size smaller watershed, interpolate or use Figure 2-1 or Figure 2-2. For watersheds smaller than 5 square miles, use a DARF equal to 1.0. If the watershed is larger than 400 square miles, use the value for 400 square miles. In the case of durations less than 30 minutes, use the 30-minute DARF value. For durations greater than 24 hours, use a DARF equal to 1.0.

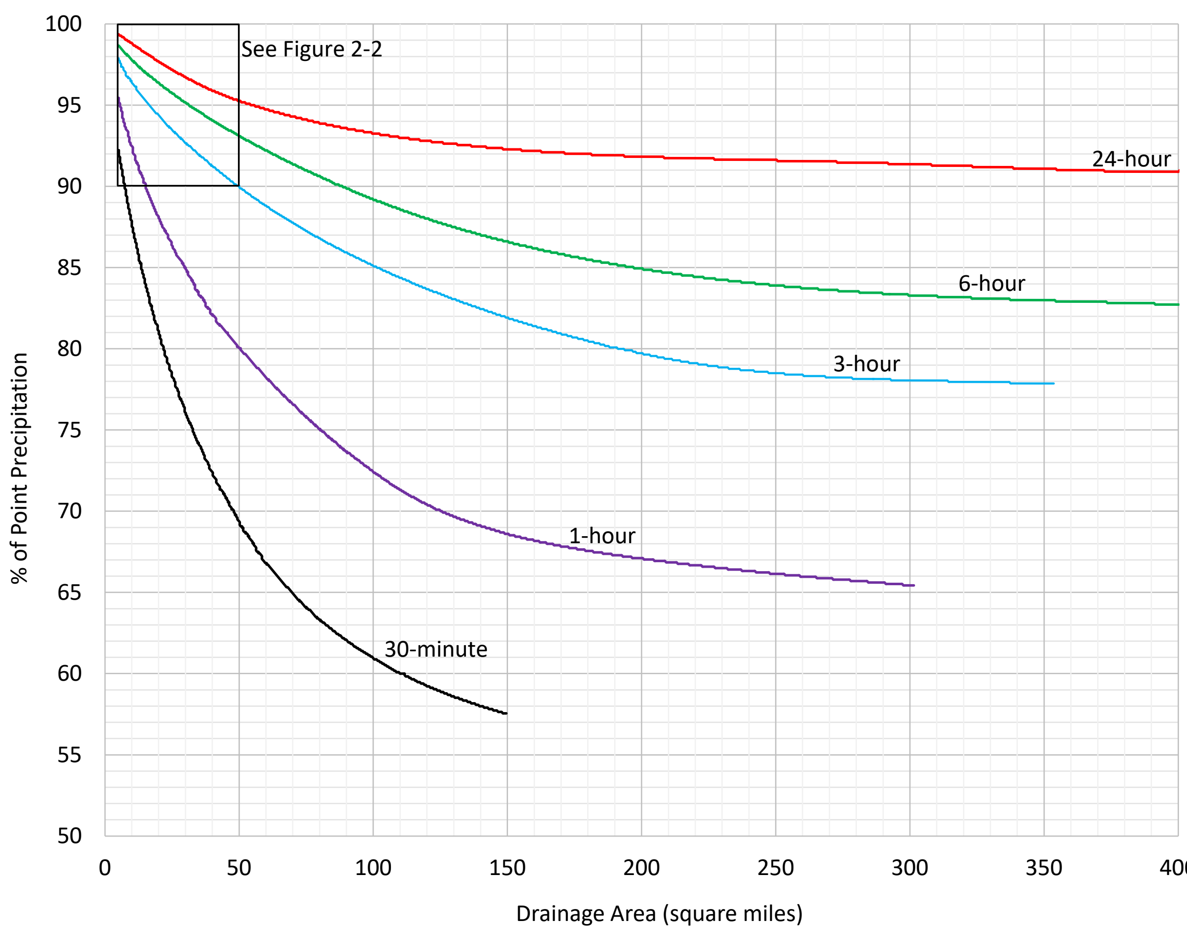


Figure 2-1. Depth-Area Reduction Factor Curves for Imperial County

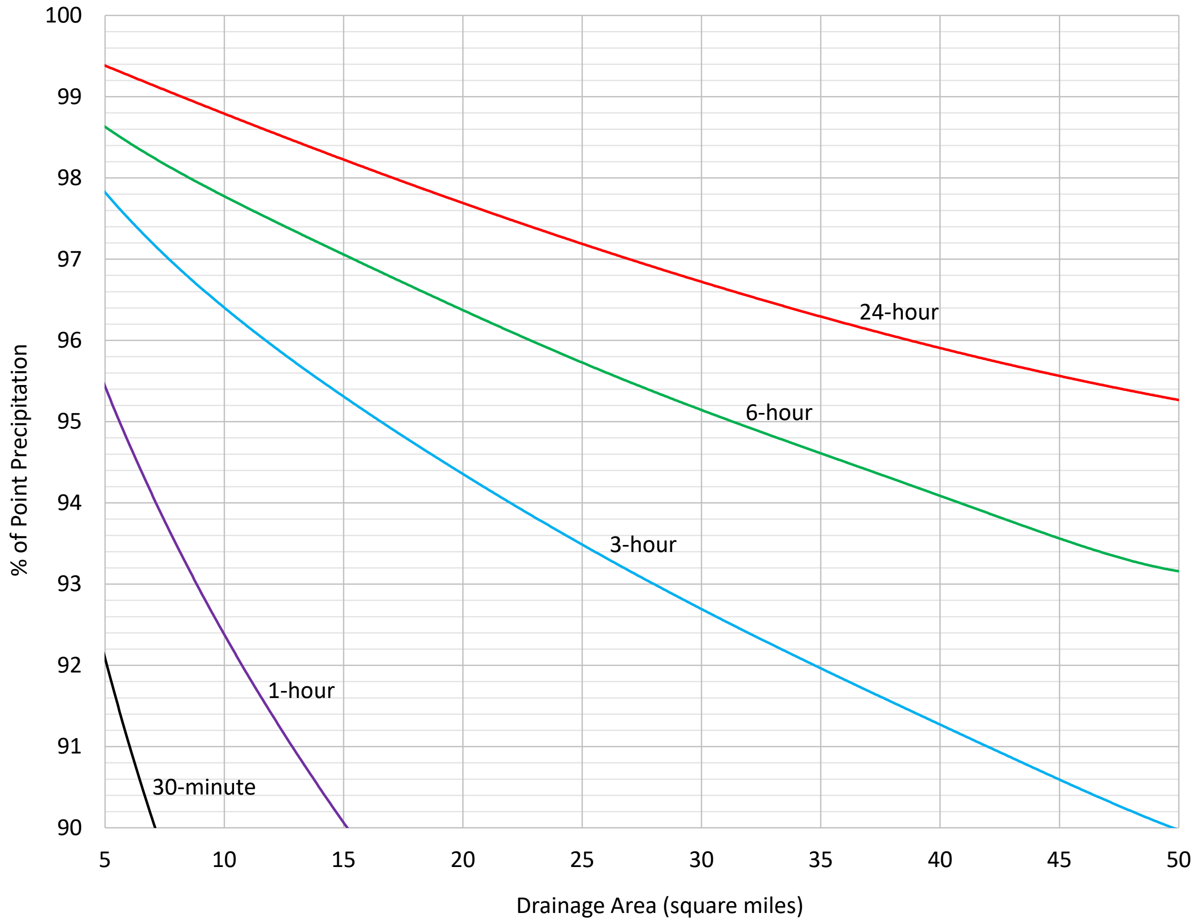


Figure 2-2. Depth-Area Reduction Factor Curves for Imperial County (5 to 50 square miles)

Table 2-1. Depth-Area Reduction Factors for Imperial County

Watershed Area (mi²)	Duration				
	30-min	1-hr	3-hr	6-hr	24-hr
0	1.000	1.000	1.000	1.000	1.000
5	0.942	0.970	0.980	0.985	0.990
10	0.900	0.947	0.970	0.980	0.985
20	0.834	0.900	0.952	0.963	0.975
30	0.768	0.858	0.932	0.950	0.964
40	0.730	0.830	0.915	0.940	0.958
50	0.692	0.800	0.900	0.928	0.952
60	0.663	0.778	0.883	0.920	0.948
70	0.645	0.760	0.872	0.912	0.945
80	0.630	0.746	0.862	0.904	0.942
90	0.620	0.735	0.853	0.896	0.938
100	0.610	0.722	0.845	0.890	0.935
125	0.588	0.700	0.830	0.878	0.930
150	0.572	0.685	0.818	0.865	0.925

Watershed Area (mi²)	Duration				
	30-min	1-hr	3-hr	6-hr	24-hr
175	0.572	0.672	0.808	0.858	0.922
200	0.572	0.666	0.798	0.851	0.918
225	0.572	0.660	0.790	0.845	0.915
250	0.572	0.655	0.787	0.842	0.914
300	0.572	0.652	0.782	0.838	0.912
350	0.572	0.652	0.780	0.830	0.910
400	0.572	0.652	0.780	0.828	0.908

2.4 Temporal Distribution

When the Rational Method is used, equal distribution of rainfall is assumed and only the peak discharge resulting from the rainfall is estimated. When the NRCS Method is used, there is no assumption of evenly distributed rainfall and the method may be used to estimate a runoff hydrograph (discharge varies with time). Because rainfall may vary over the runoff time period, the temporal distribution of the rainfall event becomes important. The temporal distribution of the rainfall is *when* the rainfall occurs throughout the storm event. The time distribution of rainfall during a storm can be represented graphically as a hyetograph, a chart showing increments of average rainfall during successive units of time during a storm.

The rainfall distribution adopted for this manual is a nested storm pattern, based on the United States Army Corps of Engineers (USACE), Hydrologic Engineering Center (HEC) Training Document Number 15 (HEC TD-15), *Hydrologic Analysis of Ungaged Watersheds Using HEC-1* (USACE, 1982). A 24-hour nested storm shall be used for flood flow computations. The peak of the nested storm will occur at hour 16 of the 24- hour storm. The nested storm will be distributed about hour 16 of the 24-hour storm using a (2/3, 1/3) distribution. The nested storm pattern with 2/3, 1/3 distribution is presented in Figure 2-3.

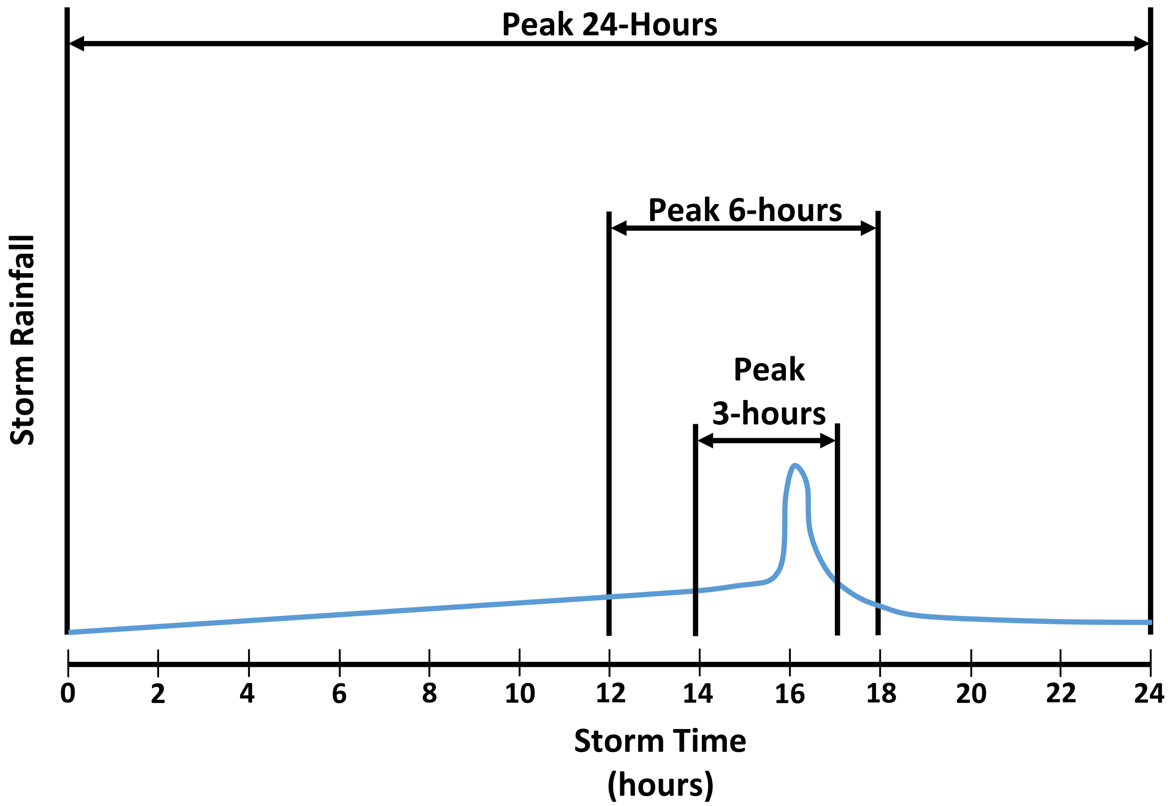


Figure 2-3. Nested Storm Pattern with 2/3, 1/3 Distribution

Creation of the 24-hour nested storm rainfall distribution requires rainfall depths for increments of storm duration from the selected computation interval through 24 hours (e.g., to create the nested storm using a 15-minute computation interval, rainfall depths are required for durations equal to 15 minutes, 30 minutes, 45 minutes, 1 hour, 1.25 hours, and so on through 24 hours). The computation interval is the period of excess rainfall (D) and should be less than or equal to twenty percent of the time to peak ($0.2T_p$). Excess rainfall is the volume of precipitation that falls at any intensity exceeding that which can infiltrate and T_p is the time to peak runoff in the watershed, which is discussed in Section 4.2.5.

Total rainfall amounts for the appropriate 6-hour design duration and/or 24-hour design duration shall be obtained from the NOAA Atlas 14 PFDS as described in Section 2.2. For durations not available from the NOAA Atlas 14 PFDS, log-log interpolation with the nearest duration values may be used to estimate the rainfall for the duration. If the watershed area is greater than 10 square miles, the rainfall depth for each duration must be adjusted using the appropriate depth-area adjustment values based on the watershed area from Table 2-1. For durations less than 30 minutes, the 30-minute depth area adjustment value is used. For durations greater than 30 minutes and not equal to durations with data available in Table 2-1, depth area adjustment is interpolated by linear interpolation between the surrounding data points.

Ordinates of the design storm hyetograph are created using the depth-area adjusted rainfall amounts. The first ordinate R_D is the depth-area adjusted total rainfall amount for the first time increment. The second ordinate R_{2D} - R_D is the depth-area adjusted total rainfall amount for the second time increment minus the depth-area adjusted total rainfall amount for the first time increment. The third ordinate R_{3D} - R_{2D} is the depth-area adjusted total rainfall amount for the third time increment minus depth-area adjusted total rainfall amount for the second time increment, and so on. Note: the sum of the ordinates of the hyetograph should be equal to the depth-area adjusted total rainfall amount for the design duration (6 hours or 24 hours). A worked example of this procedure is presented in the following section of this manual. This procedure is also available within the Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) software (the frequency storm hyetograph option with 67 percent weighting).

To obtain the 2/3, 1/3 temporal distribution, sort the ordinates of the hyetograph into the 2/3, 1/3 order of distribution. The first ordinate is the peak rainfall ordinate. This peak rainfall ordinate occurs at hour 16.0 of the 24-hour storm. The second rainfall ordinate occurs at 16.0 hours - 1D, the third rainfall ordinate occurs at 16.0 hours - 2D, and the fourth rainfall ordinate occurs at 16.0 hours + 1D. The sequence continues alternating two ordinates to the left and one ordinate to the right as presented in Figure 2-4. Creation of such a design storm is required for use of the NRCS Method to determine runoff from watersheds larger than 640 acres (1.0 square mile.) A method using HEC-HMS to perform the calculations is described in Section 4.4.

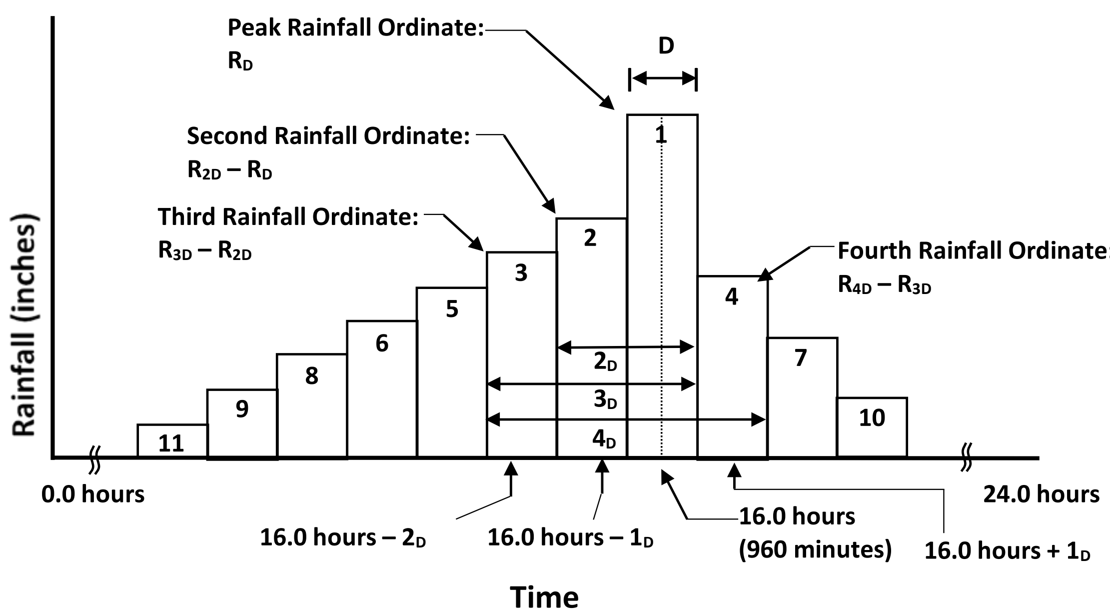


Figure 2-4. Design Storm Hyetograph Construction

2.5 Worked Example

Create a 100-year, 24-hour storm hyetograph. Assume the watershed area is 7,400 acres and the T_p is 5 hours. The center of the watershed is located at approximately 33.1130°N, 115.8755°W.

Because T_p is 5 hours, the duration D is 1 hour ($D = 0.2T_p$). The gridded point precipitation data for the 100-year, 24-hour storm are downloaded from NOAA Atlas 14 as described in Section 2.2. The duration, D, is 1 hour, so required point precipitation frequency estimates are all durations from 1 hour to 24 hours. Available durations are: 60 minute, 2 hour, 3 hour, 6 hour, 12 hour and 24 hour.

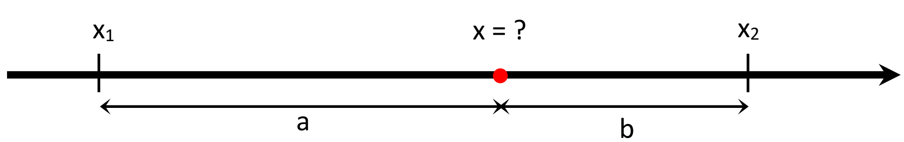
Using GIS software, the watershed boundary is delineated and an average point precipitation in the watershed is estimated for each duration using the gridded point precipitation data. Average point precipitation for this example is presented in Table 2-2.

Table 2-2. Hyetograph Example Average Precipitation

Duration	**60 min**	**2 hr**	**3 hr**	**6 hr**	**12 hr**	**24 hr**
Average Watershed Precipitation (in)	1.58	1.98	2.23	2.67	3.13	4.00

To create the hyetograph, rainfall depths for each multiple of the Duration D not provided by NOAA Atlas 14 are estimated using log interpolation. This is accomplished as follows:

> Precipitation values for hours 1, 2 and 3 were obtained directly from NOAA Atlas 14. The 4th and 5th hour precipitation amounts must be estimated using log interpolation between hour 3 and 6, however. This is accomplished using the formula
>
> $$x = x_2^{\left(\frac{a}{a+b}\right)} x_1^{\left(1 - \frac{a}{a+b}\right)} \tag{2-1}$$
>
> having variables defined as,



> The 4th hour precipitation is then estimated as
>
> $x_1 = 2.23$ $\quad x_2 = 2.67$
>
> $a = (4\text{-}3) = 1$ $\quad b = (6\text{-}4) = 2$

$$x = 2.67^{\left(\frac{1}{1+2}\right)}\ 2.23^{\left(1 - \frac{1}{1+2}\right)}$$

So, 4th hour precipitation, x = 2.37 inches

Similarly, the 5th hour precipitation is then estimated as

$x_1 = 2.23$ $\qquad$ $x_2 = 2.67$

a = (5-3) = 2 $\qquad$ b = (6-5) = 1

$$x = 2.67^{\left(\frac{2}{1+2}\right)}\ 2.23^{\left(1 - \frac{2}{1+2}\right)}$$

So, 5th hour precipitation, x = 2.51 inches

This is repeated until point precipitation values for all hours not available from NOAA Atlas 14 have been estimated. The watershed area is greater than 10 square miles (7,400 acres = 11.6 square miles), so a depth-area reduction will be applied by multiplying the DARF value and the point precipitation for that time period yielding the depth area adjusted precipitation for that time period. The hyetograph ordinate for each time period may then be determined as the difference between the hourly depth-area adjusted precipitation values. Results are summarized in Table 2-3.

Table 2-3. Summarized Values Hyetograph Example

Duration (hr)	Point Precipitation for Duration (in)*	DARF	Depth-Area Adjusted Precipitation (in)	Hyetograph Ordinate (R_{nD}) (in)
1	**1.58**	0.94	1.48	1.48
2	**1.98**	0.95	1.89	0.40
3	**2.23**	0.97	2.16	0.27
4	2.37	0.97	2.30	0.14
5	2.51	0.97	2.45	0.15
6	**2.67**	0.98	2.61	0.16
7	2.74	0.98	2.68	0.07
8	2.82	0.98	2.75	0.07
9	2.89	0.98	2.83	0.07
10	2.97	0.98	2.91	0.08
11	3.05	0.98	2.98	0.08
12	**3.13**	0.98	3.07	0.08
13	3.19	0.98	3.13	0.06
14	3.26	0.98	3.20	0.07
15	3.33	0.98	3.26	0.07

Duration (hr)	Point Precipitation for Duration (in)*	DARF	Depth-Area Adjusted Precipitation (in)	Hyetograph Ordinate (R_{nD}) (in)
16	3.40	0.98	3.33	0.07
17	3.47	0.98	3.40	0.07
18	3.54	0.98	3.47	0.07
19	3.61	0.98	3.55	0.07
20	3.69	0.98	3.62	0.07
21	3.76	0.98	3.70	0.08
22	3.84	0.98	3.77	0.08
23	3.92	0.98	3.85	0.08
24	**4.00**	0.98	3.93	0.08
				Σ = 3.93

*Bold values are directly from data, others are interpolated

Duration rainfall amounts are the hyetograph ordinates in Table 2-3 arranged in descending order in a 2/3, 1/3 fashion centered on hour 16, i.e., hour 16 = 1.48 inches, hour 15 = 0.41 inches, hour 14 = 0.27 inches, hour 17 = 0.14 inches, hour 13 = 0.15 inches, etc. The resulting, completed hyetograph is presented in Figure 2-5.

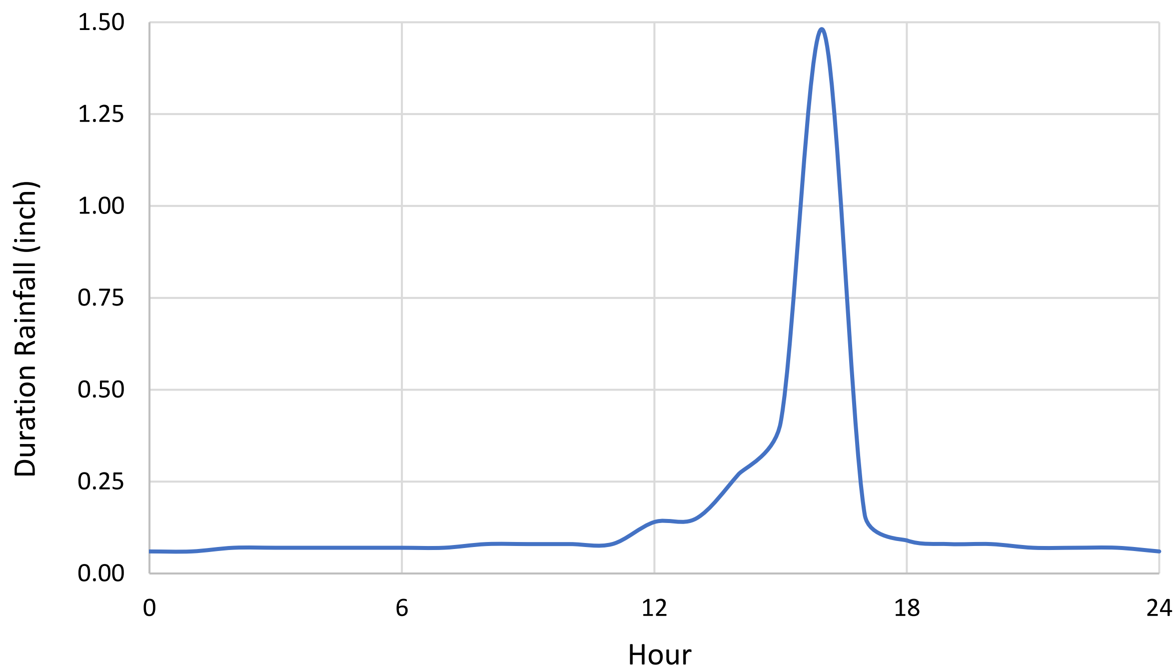


Figure 2-5. Completed Design Storm Hyetograph Example

3 Small Area Hydrologic Procedure – Rational Method

3.1 General Description

The Rational Method (RM) is a mathematical formula used to determine the maximum runoff rate from a given rainfall. It has particular application in urban storm drainage, where it is used to estimate peak runoff rates from small urban and rural watersheds for the design of storm drains and small drainage structures. The RM is recommended for analyzing the runoff response from drainage areas up to approximately 640 acres (1.0 square mile) in size. When independent drainage systems are present within the watershed being analyzed using the RM, the Modified Rational Method (MRM) should be used in order to combine the flows of the independent systems at junctions (see Section 3.4). When the watershed size exceeds 640 acres the Natural Resources Conservation Service (NRCS) Hydrologic Method should be used (see Section 4).

The RM can be applied using any design storm return interval (e.g., 100-year, 50-year, 10-year, etc.). Precipitation estimates are based on National Oceanic and Atmospheric Administration (NOAA) Atlas 14. Precipitation frequency estimates for the required storm frequency and duration can be attained via the NOAA Atlas 14 online Precipitation Frequency Data Server (PFDS) as described in Section 2.2.

3.1.1 Rational Method Formula

The RM formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area (A), runoff coefficient (C), and rainfall intensity (I). The intensity is a function of the rainfall duration and is determined for a duration set equal to the time of concentration (T_c), which is the time required for water to flow from the most hydraulically remote point of the basin to the location being analyzed. The RM formula is expressed as follows:

$$Q_p = C \cdot I \cdot A \tag{3-1}$$

Where: Q_p = peak discharge, in cubic feet per second (cfs)

C = runoff coefficient, proportion of the rainfall that runs off the surface (no units)

I = average rainfall intensity for a duration equal to the T_c for the area, in inches per hour (Note: If the computed T_c is less than 5 minutes, use 5 minutes for computing the peak discharge, Q_p)

A = drainage area contributing to the design location, in acres

Combining the units for the expression CIA yields:

$$\left(\frac{\text{acres}\cdot\text{inches}}{\text{hour}}\right)\left(\frac{43{,}560\text{ square feet}}{\text{acres}}\right)\left(\frac{1\text{ foot}}{12\text{ inches}}\right)\left(\frac{1\text{ hour}}{3{,}600\text{ seconds}}\right)=1.008\text{ cfs} \qquad (3\text{-}2)$$

For practical purposes the unit conversion coefficient difference of 0.8% can be ignored.

The RM formula is based on the assumption that for constant rainfall intensity, the peak discharge rate at a point will occur when the raindrop that falls at the most hydraulically remote point in the tributary drainage basin arrives at the point of interest. The most hydraulically remote point is the location from which drainage will take the longest to arrive at the point of interest. Figure 3-1 demonstrates this concept.

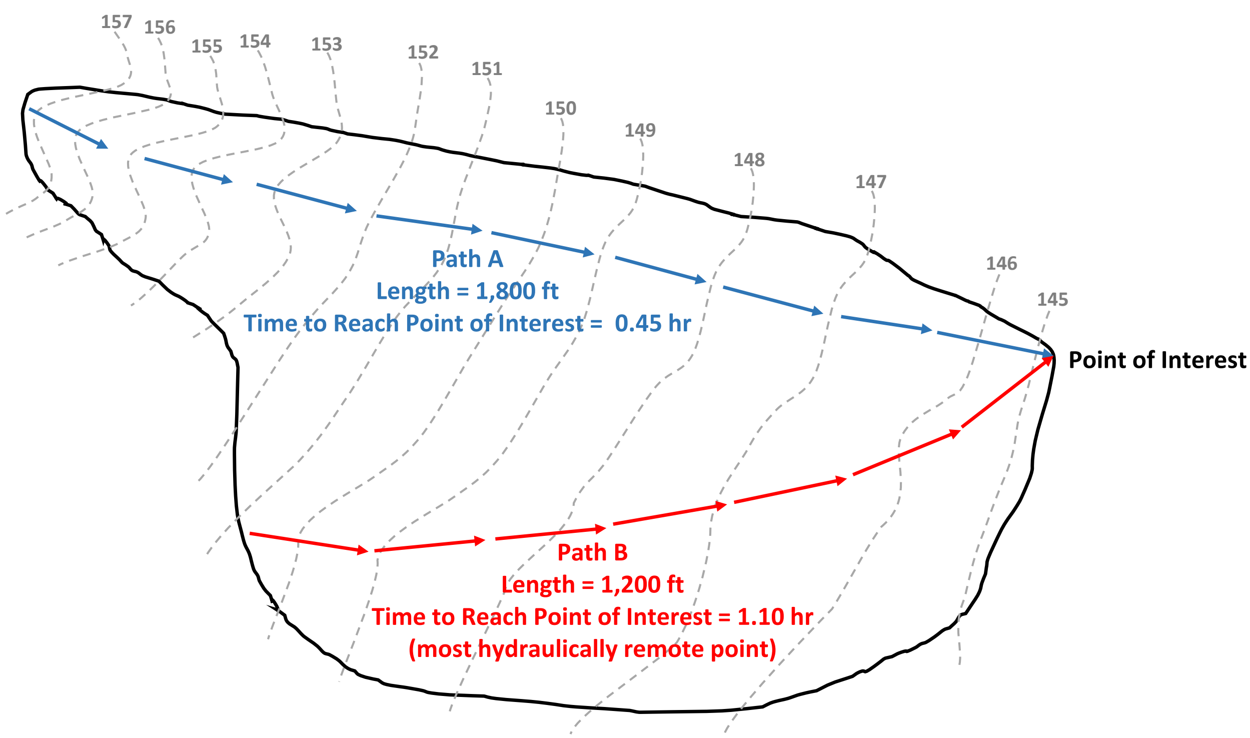


Figure 3-1. Most Hydraulically Remote Point

Unlike the Modified Rational Method (MRM) (discussed in Section 3.4) or the NRCS hydrologic method (discussed in Section 4), the RM does not create hydrographs and therefore does not add separate subarea hydrographs at collection points.

As discussed above, the characteristics of the RM are summarized as follows:

1) Peak flow occurs when the entire watershed is contributing to the flow.

2) Rainfall intensity is the same over the entire drainage area.

3) Rainfall intensity is uniform over a time duration equal to T_c.

4) The storm frequency of peak discharges is the same as that of I for the given T_c.

5) The fraction of rainfall that becomes runoff (or the runoff coefficient, C) is dependent on the return period.

6) The peak rate of runoff is the only information produced by using the RM.

3.1.2 Runoff Coefficient

The runoff coefficient (C) corresponds to the percentage of rainfall that becomes runoff. An estimated value for C may be determined from Table 3-2 or Table 3-3. Table 3-2 provides ranges of runoff coefficient values based on land use. Table 3-3 provides urban runoff coefficients based on land use and soil type.

Soil type determination should be done using a method approved by the County prior to work being done. If the County has no preferred method at the site, two possible methods are soil testing at the site or using the USDA NRCS Web Soil Survey online tool available here: http://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx. An appropriate runoff coefficient (C) for each type of land use in the subarea should be selected from Table 3-3 and multiplied by the percentage of the total area (A) included in that class. The sum of the products for all land uses is the weighted runoff coefficient $\sum(C \cdot A)$. Good engineering judgment should be used when applying the values presented in Table 3-3, as adjustments to these values may be appropriate based on site-specific characteristics.

Table 3-2 and Table 3-3 provide approximate runoff coefficient values for various development types. In urban areas the runoff coefficient can also be estimated based on the percent of impervious area and the percent of open space based on the following formula:

$$C = 0.90 \times (\%\ \text{Impervious}) + C_p \times (1 - \%\ \text{Impervious}) \qquad (3\text{-}3)$$

Where: C_p = Pervious Coefficient Runoff Value for the soil type (shown in Table 3-3 as Undisturbed Natural Terrain/Permanent Open Space, 0% Impervious). Soil type can be determined as previously described.

The values in Table 3-3 are typical for most urban areas. However, if the basin contains rural or agricultural land use, parks, golf courses, or other types of nonurban land use that are expected to be permanent, the appropriate value should be selected based upon the soil and cover and approved by the County.

The determined runoff coefficient (C) is for storm return periods up to 10 years. Less frequent, higher intensity storms tend to generate more runoff requiring a modification to the runoff coefficient. For these storms, the adjusted C value is obtained by multiplying C by the appropriate value in Table 3-1. *The final runoff coefficient may never exceed 1.0.* (If the modified runoff coefficient exceeds 1.0, use the value 1.0.)

Table 3-1. 'C' Modification Value Based on Return Period

Return Period (years)	'C' Modification Value
25	1.1
50	1.2
100	1.25

after Caltrans Highway Design Manual, July 1, 2015. pp. 810-18

Table 3-2. Runoff Coefficient Values

Land Use	'C' Coefficient Range		Soil Type	'C' Coefficient Range
Business			**Lawns, slope**	
downtown areas	0.70 – 0.95		sandy soil, flat, 2%	0.05 – 0.10
neighborhood areas	0.50 – 0.70		sandy soil, avg., 2 – 7%	0.10 – 0.15
Residential			sandy soil, steep, 7%	0.15 – 0.20
single family areas	0.30 – 0.50		heavy soil, flat, 2%	0.13 – 0.17
multi units, detached	0.40 – 0.60		heavy soil, avg., 2 – 7%	0.18 – 0.22
multi units, attached	0.60 – 0.75		heavy soil, steep, 7%	0.25 – 0.35
suburban	0.25 – 0.40		**Agricultural land**	
Industrial			*bare packed soil*	
light areas	0.50 – 0.80		smooth	0.30 – 0.60
heavy areas	0.60 – 0.90		rough	0.20 – 0.50
Parks and Cemeteries	0.60 – 0.90		*cultivated rows*	
Playgrounds	0.60 – 0.90		heavy soil, no crop	0.30 – 0.60
Railroad yard areas	0.60 – 0.90		heavy soil, with crop	0.20 – 0.50
			sandy soil, no crop	0.20 – 0.40
			sandy soil, with crop	0.10 – 0.25
			pasture	
			heavy soil	0.15 – 0.45
			sandy soil	0.05 – 0.25
			woodlands	0.05 – 0.25

Table 3-3. Runoff Coefficients for Urban Areas

Land Use		Runoff Coefficient "C"				
			Soil Type			
NRCS Elements	Structure(s) Utilization	% IMPER	A	B	C	D
Undisturbed Natural Terrain (Natural)	Permanent Open Space	0*	0.20	0.25	0.30	0.35
Low Density Residential (LDR)	Residential, 1.0 DU/A or less	10	0.27	0.32	0.36	0.41
Low Density Residential (LDR)	Residential, 2.0 DU/A or less	20	0.34	0.38	0.42	0.46
Low Density Residential (LDR)	Residential, 2.9 DU/A or less	25	0.38	0.41	0.45	0.49
Medium Density Residential (MDR)	Residential, 4.3 DU/A or less	30	0.41	0.45	0.48	0.52
Medium Density Residential (MDR)	Residential, 7.3 DU/A or less	40	0.48	0.51	0.54	0.57
Medium Density Residential (MDR)	Residential, 10.9 DU/A or less	45	0.52	0.54	0.57	0.60
Medium Density Residential (MDR)	Residential, 14.5 DU/A or less	50	0.55	0.58	0.60	0.63
High Density Residential (HDR)	Residential, 24.0 DU/A or less	65	0.66	0.67	0.69	0.71
High Density Residential (HDR)	Residential, 43.0 DU/A or less	80	0.76	0.77	0.78	0.79
Commercial/Industrial (N. Com)	Neighborhood Commercial	80	0.76	0.77	0.78	0.79
Commercial/Industrial (G. Com)	General Commercial	85	0.80	0.8	0.81	0.82
Commercial/Industrial (O.P. Com)	Office Professional/Commercial	90	0.83	0.84	0.84	0.85
Commercial/Industrial (Limited I.)	Limited Industrial	90	0.83	0.84	0.84	0.85
Commercial/Industrial (General I.)	General Industrial	95	0.87	0.87	0.87	0.87

*The values associated with 0% impervious may be used for direct calculation of the runoff coefficient as described in Section 3.12 (representing the pervious runoff coefficient, C_p, for the soil type), or for areas that will remain undisturbed in perpetuity. Justification must be given that the area will remain natural forever (e.g., the area is located in Cleveland National Forest).

DU/A = dwelling units per acre

3.1.3 Rainfall Intensity

The rainfall intensity (I, inches/hour) is the rainfall rate for a duration equal to the time of concentration (T_c) for a selected storm frequency. Once a particular storm frequency has been selected for design and a T_c calculated for the drainage area, the rainfall intensity can be determined from the NOAA Atlas 14 Point Precipitation Frequency Estimates as described in Section 2.2. Interpolation will likely be necessary to obtain the rainfall intensity corresponding to T_c.

3.1.4 Time of Concentration

The time of concentration (T_c) is the time required for runoff to flow from the most hydraulically remote part of the drainage area to the point of interest. The T_c is composed of two components: initial time of concentration (T_i) and travel time (T_t). Methods of computation for T_i and T_t are discussed below. The T_i is the time required for runoff to travel as sheet flow across the surface of the most remote subarea in the study, or "initial subarea." Guidelines for designating the initial subarea are provided within the discussion of computation of T_i in the following section. The T_t is the time required for the runoff to flow in a watercourse (e.g., swale, channel, gutter, and pipe) or series of watercourses from the initial subarea to the point of interest. For the RM, the T_c at any point within the drainage area is given by:

$$T_c = T_i + T_t \tag{3-4}$$

Methods of calculation differ for natural watersheds (non-urbanized) and for urban drainage systems, however, if T_C is estimated to be less than 5 minutes, use 5 minutes in natural or urban watersheds. When analyzing storm drain systems, the designer must consider the possibility that an existing natural watershed may become urbanized during the useful life of the storm drain system. Future land uses must be used for T_c and runoff calculations, and can be determined by consulting with the County.

3.1.4.1 Initial Time of Concentration

The initial time of concentration (T_i) is typically based on sheet flow at the upstream end of a drainage basin. Sheet flow is the shallow mass of runoff on a planar surface with a uniform depth across the sloping surface. This usually occurs at the headwater of streams over relatively short distances, rarely more than about 400 feet, and possibly less than 80 feet. Maximum overland sheet flow lengths based on land use and slope are provided in Table 3-4. Suggested initial T_i values based on average C values are also provided in the table. Alternatively, the initial time of concentration (T_i) may be estimated using Equation (3-5) developed by the Federal Aviation Administration (FAA) (still observing maximum overland sheet flow length).

$$T_i = \frac{1.8(1.1\text{-}C)\sqrt{L}}{S^{1/3}} \qquad (3\text{-}5)$$

Where: T_i = sheet flow travel time, minutes

C = runoff coefficient (use Table 3-3 or Equation (3-3) and modify using Table 3-1 according to the return period)

L = flow length, feet (subject to Table 3-4)

S = surface slope, %

The sheet flow that is predicted by the FAA equation is limited to conditions that are similar to runway topography. Some considerations that limit the extent to which the FAA equation applies are identified below:

Urban Areas - This “runway type” runoff includes:

1) Flat roofs, sloping at 1%.

2) Parking lots at the extreme upstream drainage basin boundary (at the “ridge” of a catchment area). Even a parking lot is limited in the amounts of sheet flow it can produce. Parked or moving vehicles “break-up” the sheet flow, concentrating runoff into streams that are not characteristic of sheet flow.

3) Driveways are constructed at the upstream end of catchment areas in some developments. However, if flow from a roof is directed to a driveway through a downspout or other conveyance mechanism, flow is concentrated.

4) Flat slopes are prone to meandering flow that tends to be disrupted by minor irregularities and obstructions. Maximum Overland Flow lengths are shorter for flatter slopes (see Table 3-4).

Rural or Natural Areas - The FAA equation is applicable to these conditions since (0.5% to 10%) slopes that are uniform in width of flow (e.g. flow depth and velocity are not being greatly affected by widely varying lateral boundaries) have slow velocities consistent with the equation. Irregularities in terrain limit the length of application.

1) Most hills and ridge lines have a relatively flat area near the drainage divide. However, with flat slopes of 0.5%, minor irregularities cause flow to concentrate into streams.

2) Parks, lawns and other vegetated areas have slow velocities that are consistent with the FAA Equation.

Table 3-4. Maximum Overland Sheet Flow Length (L_M) in feet and Corresponding T_i Estimate in minutes

Land Use*	DU/ acre	.5%		1%		2%		3%		5%		10%	
		L_M	T_i	L_M	T_i	L_M	T_i	L_M	T_i	L_M	T_i	L_M	T_i
Natural		50	13.2	70	12.5	85	10.9	100	10.3	100	8.7	100	6.9
LDR	1	50	12.2	70	11.5	85	10.0	100	9.5	100	8.0	100	6.4
LDR	2	50	11.3	70	10.5	85	9.2	100	8.8	100	7.4	100	5.8
LDR	2.9	50	10.7	70	10.0	85	8.8	95	8.1	100	7.0	100	5.6
MDR	4.3	50	10.2	70	9.6	80	8.1	95	7.8	100	6.7	100	5.3
MDR	7.3	50	9.2	65	8.4	80	7.4	95	7.0	100	6.0	100	4.8
MDR	10.9	50	8.7	65	7.9	80	6.9	90	6.4	100	5.7	100	4.5
MDR	14.5	50	8.2	65	7.4	80	6.5	90	6.0	100	5.4	100	4.3
HDR	24	50	6.7	65	6.1	75	5.1	90	4.9	95	4.3	100	3.5
HDR	43	50	5.3	65	4.7	75	4.0	85	3.8	95	3.4	100	2.7
N. Com		50	5.3	60	4.5	75	4.0	85	3.8	95	3.4	100	2.7
G. Com		50	4.7	60	4.1	75	3.6	85	3.4	90	2.9	100	2.4
O.P. Com		50	4.2	60	3.7	70	3.1	80	2.9	90	2.6	100	2.2
Limited I.		50	4.2	60	3.7	70	3.1	80	2.9	90	2.6	100	2.2
General I.		50	3.7	60	3.2	70	2.7	80	2.6	90	2.3	100	1.9

*Source: Hill, 2002. See Table 3-3 for land use abbreviations.

Because the rainfall intensity, (I), depends on T_C and T_C is not initially known, the computation of T_C is an iterative process. An initial estimate of T_C is assumed to be T_i, computed from Equation (3-5). The *initial estimate* of T_C is then used to obtain I from the Intensity-Depth-Frequency (IDF) curve for the locality. A more complete T_C is then computed from Equation (3-5) by incorporating travel time (Section 3.1.4.2). The T_C which incorporates T_i and T_t is then used to select a new rainfall intensity and T_C is calculated again. If the first and second calculated T_C are not the same, a new rainfall intensity is determined and Equation (3-5) is used to calculate T_C again. The process is repeated until two successive T_C estimates are the same.

3.1.4.2 Travel Time

Sheet flow is the first type of flow to occur when a rain drop falls on the most hydraulically remote point of the basin. This is typically followed by shallow concentrated flow and eventually open channel or pipe flow. The shallow concentrated flow time and open channel or pipe flow travel time together comprise the total travel time (T_t). Both of these are determined by calculating the velocity of flow and dividing by the travel length. Per Equation (3-4) when added to the initial sheet flow time, one obtains the time of concentration T_c.

Because the velocity normally changes with change in flow rate or slope, such as at an inlet or grade break, the total T_t must be computed as the sum of the T_t's for each section of the flow path. Figure 3-2 is a typical street gutter cross section and shows two possible flow depths: (1) all flow is contained in the concrete section adjacent to the curb and (2) flow fills the concrete portion of the gutter and extends out onto the asphalt. For street gutter geometries sufficiently similar to Figure 3-2, use Figure 3-3 to estimate shallow concentrated flow velocity. To estimate shallow concentrated flow velocity for other land covers, use Equation (3-6). To estimate average velocities in channels or pipes (or street gutter geometries not sufficiently similar to Figure 3-2), use Equation (3-7) (Manning's equation).

When flow is through a closed conduit where no additional flow can enter the system during travel, length, velocity and T_t are determined using the peak flow in the conduit. In cases where the conduit is not closed and additional flow from a contributing subarea is added to the total flow during travel (e.g., street flow in a gutter), calculation of velocity and T_t is performed using an assumed average flow based on the total area (including upstream subareas) contributing to the point of interest. The Manning equation is typically used to determine velocity. A reasonable initial estimate of average discharge for small watersheds is 2 to 3 cfs per acre, dependent on land use, drainage area, slope, and rainfall intensity.

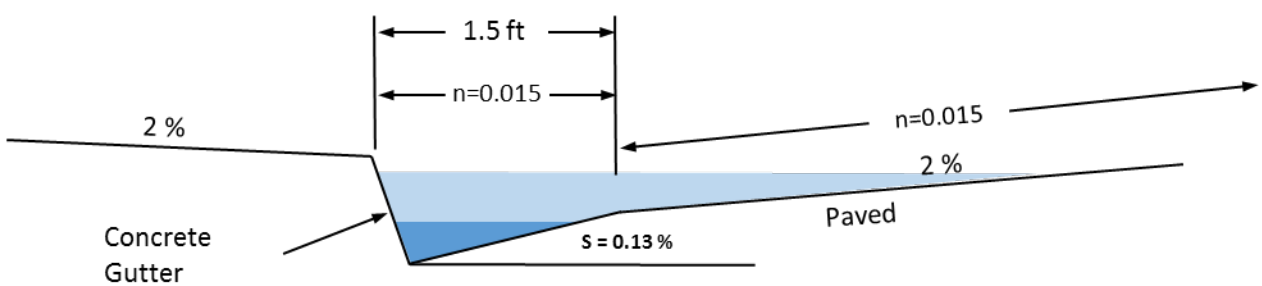
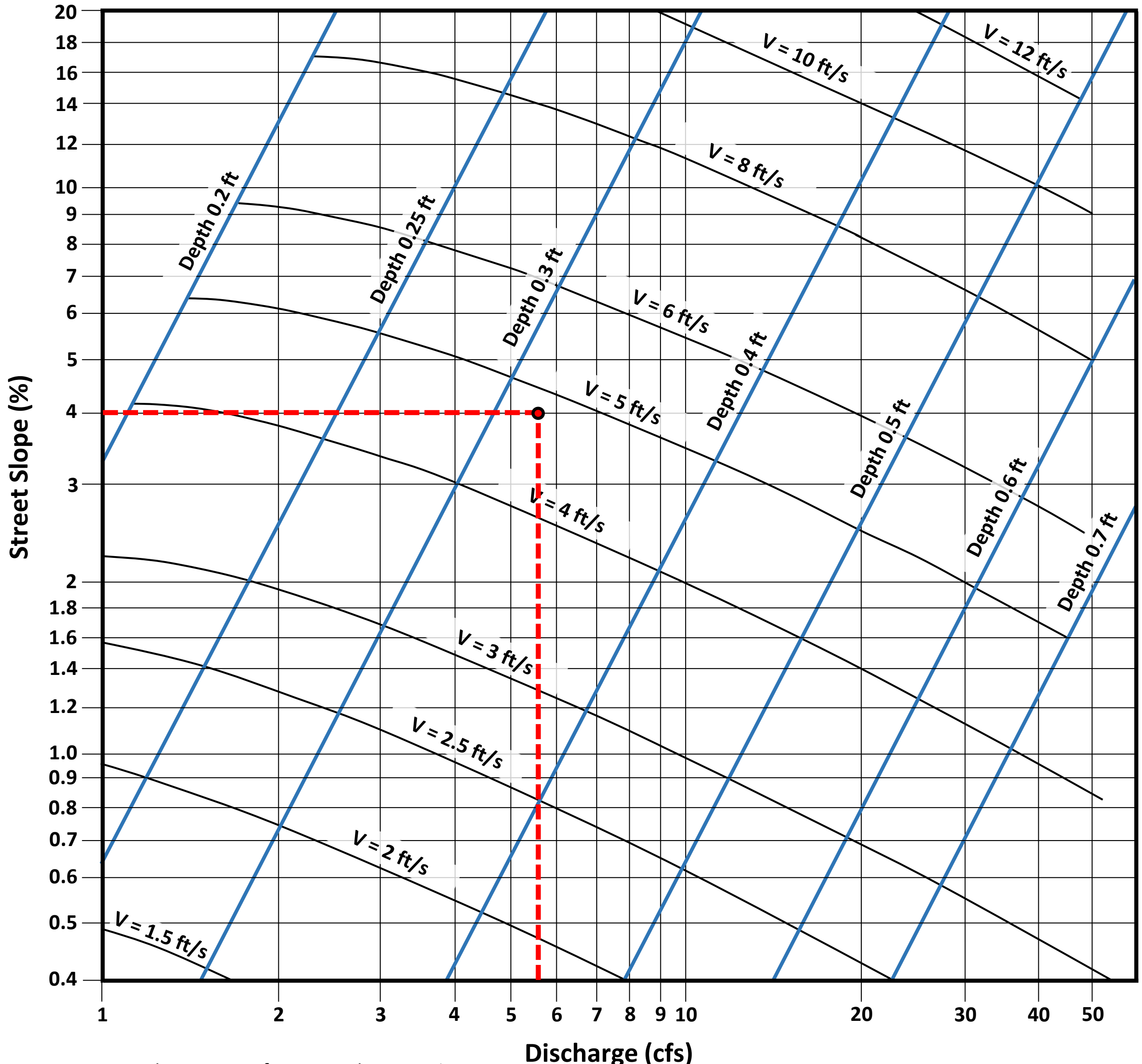


Figure 3-2. Street Gutter Geometry



Example: Q = 5.5 cfs, Street Slope = 4%
Then, Flow Depth = 0.32 ft and V = 4.8 ft/s.

Figure 3-3. Street Gutter Flow Velocity (San Diego County, 2003)

Shallow concentrated flow begins when sheet flow ends, without a well-defined channel, and with flow depths of 0.1 to 0.5 feet. Shallow concentrated flow continues until justification can be made for defining it as an open channel or pipe flow. Engineering judgment may be called for in deciding where shallow concentrated flow ends and open channel flow begins. Equation (3-6) can be used to estimate shallow concentrated flow velocity (FHWA, 2013):

$$V = 3.28 \cdot k \cdot \sqrt{S} \qquad (3\text{-}6)$$

Where: V = velocity, feet/second

k = intercept coefficient (see Table 3-5)

S = slope, %

Table 3-5. Shallow Concentrated Flow Intercept Coefficients (k) (FHWA, 2013)

Land Cover	k
Forest with heavy ground litter; hay meadow	0.076
Trash fallow or minimum tillage cultivation; contour or strip cropped; woodland	0.152
Short grass pasture	0.213
Cultivated straight row	0.274
Nearly bare and untilled; alluvial fans in western mountain regions	0.305
Grassed waterway	0.457
Unpaved	0.491
Paved area; small upland gullies	0.619

$$V = \frac{1.49}{n} \cdot R^{2/3} \cdot S^{1/2} \qquad (3\text{-}7)$$

Where: V = velocity, feet/second

n = roughness coefficient (see Table 3-6)

R = hydraulic radius (cross sectional flow area divided by wetted perimeter), feet

S = slope, foot/foot

Table 3-6. Typical Manning's Coefficient (*n*) Ranges for Channels and Pipes (FHWA, 2013)

Material	Manning's *n**
Closed Conduits	
Concrete pipe	0.010 - 0.015
Corrugated Metal Pipe (CMP)	0.011 - 0.037
Plastic pipe (smooth)	0.009 - 0.015
Plastic pipe (corrugated)	0.018 - 0.025
Pavement/gutter sections	0.012 - 0.016
Small Open Channels	
Concrete	0.011 - 0.015
Rubble or riprap	0.020 - 0.035

Material	Manning's *n**
Vegetation	0.020 - 0.150
Bare Soil	0.016 - 0.025
Rock Cut	0.025 - 0.045
Natural channels (minor streams, top width at flood stage < 30 m (100 ft))	
Fairly regular section	0.025 - 0.050
Irregular section with pools	0.040 - 0.150

*Lower values are usually for well-constructed and maintained (smoother) pipes and channels

A common mistake in urbanized areas is to assume travel velocities that are too slow. Another common error is to not check the runoff peak resulting from only part of the catchment. Sometimes a lower portion of the catchment or a highly impervious area produces a larger peak than that computed for the whole catchment. This error is most often encountered when the catchment is long or the upper portion contains grassy open land and the lower portion is more developed.

3.2 Input Data Development for the Rational Method

This section describes the development of the necessary data to perform Rational Method (RM) calculations. Section 3.3 describes the RM calculation process. Input data for calculating peak flows and T_c's with the RM should be developed as follows:

1) On a digital elevation map (DEM) or topographic base map create a drainage map of existing conditions:

   a) Delineate the drainage area boundary, and

   b) Mark drains, including drains adjacent to the delineated drainage area and overland flow paths. (Mark existing and proposed drains if evaluating existing and proposed conditions, otherwise mark existing drains for an existing conditions study and proposed drains for a proposed conditions study.)

2) Visit the site to verify the accuracy of the drainage map.

3) Divide the drainage area into subareas by locating significant points of interest. These divisions should be based on topography, soil type, and land use. Ensure that an appropriate first subarea is delineated. The first subarea is the area that is most hydraulically distant and whose runoff will

take the longest to reach the outlet. For natural areas, the first subarea flow path length should be less than or equal to 4,000 feet plus the overland flow length (see Table 3-4 for maximum allowable overland sheet flow lengths). For developed areas, the initial subarea flow path length should be consistent with Table 3-4. The topography and slope within the initial subarea should be generally uniform.

4) Working from upstream to downstream, label subareas and subarea drainage outlet locations.

5) Determine the areal coverage in acres (A) of each subarea in the drainage area.

6) Determine the length and effective slope(s) of the flow path in each subarea.

7) Identify the soil type for each subarea.

8) Determine the runoff coefficient (C) for each subarea based on Table 3-3 or Equation (3-3). If the subarea contains more than one type of development classification, determine a weighted average for C in the subarea. In determining C, use future land use taken from the applicable community plan, Multiple Species Conservation Plan, National Forest land use plan, etc.

9) Calculate the $(C \cdot A)$ value for the subarea.

10) Calculate the $(C \cdot A)$ value(s) for the subareas upstream of the point(s) of interest. Determine C for each subarea based on guidance in Section 3.1.2

3.3 Performing Rational Method Calculations

Using the developed input data, calculation of peak flows and T_c's should be performed as follows:

1) Determine T_i for the first subarea. An example is presented as Subarea A_1 in Figure 3-4. Use Table 3-4 or Equation (3-5) as discussed in Section 3.1.4.1. Additional travel time (T_t) to the downstream end of the first subarea should be added to T_i to obtain the T_c if the flow path in the first subarea is longer than the maximum length for sheet flow. Refer to Section 3.1.4.2.

2) Determine I for the subarea using NOAA Atlas 14. If T_i is less than 5 minutes, use the 5 minute time to determine intensity for calculating the flow.

3) Calculate the peak discharge flow rate for the subarea, where

$$Q_p=(C_1 \cdot A_1)I \qquad (3\text{-}8)$$

4) In case the downstream flow rate is less than the upstream flow rate, due to lower I resulting from the long travel time that is not offset by the additional subarea runoff, use the upstream peak flow for design purposes until downstream flows increase again.

5) Estimate the T_t to the next point of interest.

6) Add the T_t to the previous T_c to obtain a new T_c.

7) Continue with step 2, above, summing subareas and corresponding C values, until the final point of interest is reached.

$$Q_p= \sum_{n=1}^{\#\text{ of subareas}} (C_n \cdot A_n)I \qquad (3\text{-}9)$$

Note: The MRM should be used to calculate the peak discharge when there is a junction incorporating flows from independent subareas into the drainage system.

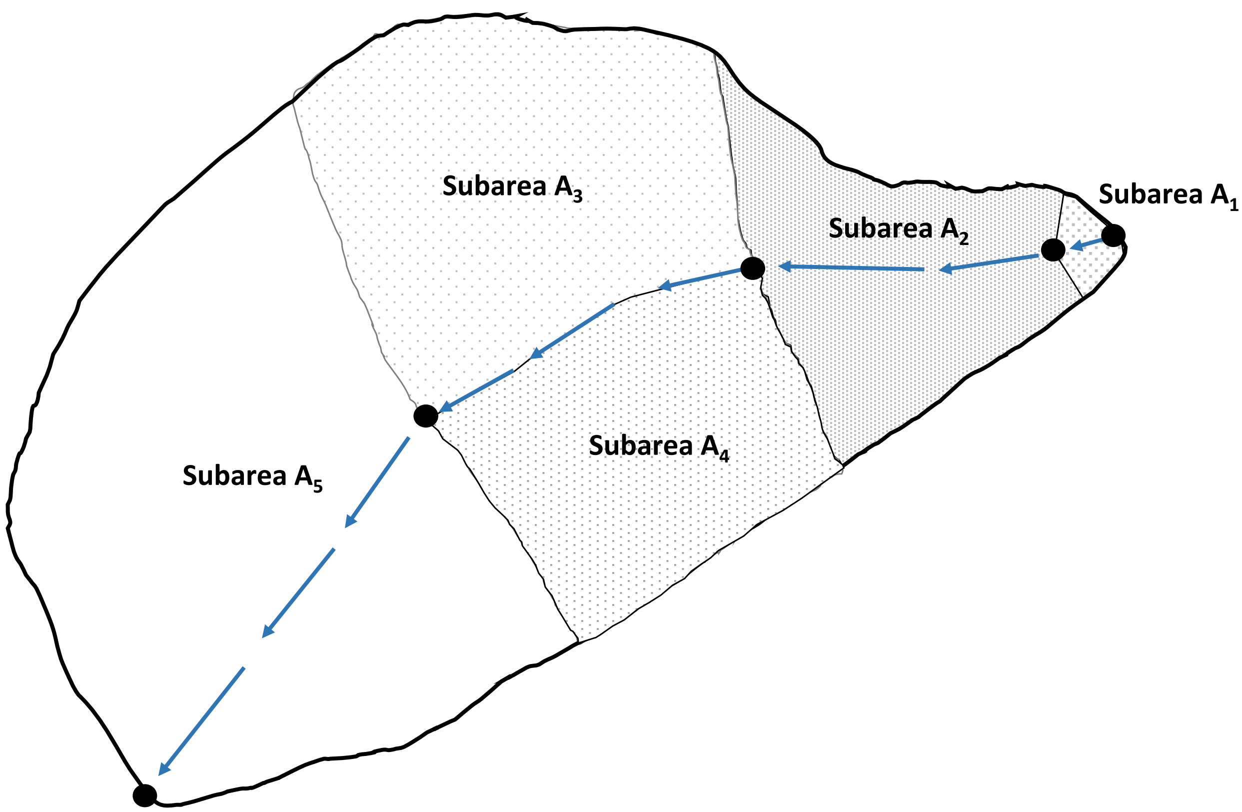


Figure 3-4. Rational Method Calculation Subareas

The flow path having the longest time of concentration to the point of interest in the storm drainage system will usually define the duration used in selecting the intensity value in the Rational Method. Exceptions to the general application of the Rational Equation exist. For example, a small relatively impervious area within a larger drainage area may have an independent discharge higher than that of the total area. This anomaly may occur because of the high runoff coefficient (C value) and high intensity resulting from a short time of concentration. If an exception does exist, it can generally be classified as one of two exception scenarios.

The first scenario occurs when a highly impervious section exists at the most downstream area of a watershed and the total upstream area flows through the lower impervious area. When this situation occurs, two separate calculations should be made.

1) Calculate the runoff from the total drainage area with its weighted C value and the intensity associated with the longest time of concentration.

2) Calculate the runoff using only the smaller less pervious area. The typical procedure would be followed using the C value for the small less pervious area and the intensity associated with the shorter time of concentration.

The results of these two calculations should be compared and the largest value of discharge should be used for design.

The second scenario exists when a smaller less pervious area is tributary to the larger primary watershed. When this scenario occurs, two sets of calculations should also be made.

1) Calculate the runoff from the total drainage area with its weighted C value and the intensity associated with the longest time of concentration.

2) Calculate the runoff to consider how much discharge from the larger primary area is contributing at the same time the peak from the smaller less pervious tributary area is occurring. When the small area is discharging, some discharge from the larger primary area is also contributing to the total discharge. In this calculation, the intensity associated with the time of concentration from the small less pervious area is used. The C coefficients for the larger and smaller areas should be determined independently of each other; the larger primary area C coefficient should *not include* the smaller, less pervious tributary area. The portion of the larger primary area to be considered is determined by the following equation:

$$A_C = A \cdot \left(\frac{T_{C1}}{T_{C2}}\right) \qquad (3\text{-}10)$$

Where: A_C = most downstream part of the larger primary area that will contribute to the discharge during the time of concentration associated with the smaller, less pervious area,

A = area of the larger primary area,

T_{C1} = time of concentration of the smaller, less pervious, tributary area,

T_{C2} = Time of concentration associated with the larger primary area as is used in the first calculation

3.4 Modified Rational Method (for Junction Analysis)

The purpose of this section is to describe the steps necessary to develop an analysis for a small watershed using the Modified Rational Method (MRM). It is necessary to use the MRM if the watershed contains junctions of independent drainage systems. The general process description for using this method, including an example of the application of this method, is described below. (Another option is to use available software acceptable to the County that performs these calculations.)

The engineer should only use the MRM for total drainage areas up to approximately 640 acres (1.0 mi^2) in size. If the overall watershed will significantly exceed 640 acres, then the NRCS method described in Section 4 should be used. The engineer may choose to use either the RM or the MRM for calculations for up to an approximately 640 acres area and then transition the study to the NRCS method for additional downstream areas that exceed approximately 640 acres. The transition process is described in Section 4.

The general process for the MRM differs from the RM only when a junction of independent drainage systems is reached. The peak Q, T_c, and I for each of the independent drainage systems at the point of the junction are calculated using the RM. The independent drainage systems are then combined using the MRM procedure described below. The peak Q, T_c, and I for each of the independent drainage systems at the point of the junction must be calculated using the RM prior to using the MRM procedure to combine the independent drainage systems. After the independent drainage systems have been combined, RM calculations are continued to the next point of interest.

3.4.1 Procedure for Combining Independent Drainage Systems at a Junction

Calculate the peak Q, T_c, and I for each of the independent drainage systems using the RM at the point of the junction. These values will be used for the MRM calculations.

At the junction of two or more independent drainage systems, the respective peak flows are combined to obtain the maximum flow out of the junction at T_c. Based on the approximation that total runoff increases directly in proportion to time, a general equation may be written to determine the maximum Q and its corresponding T_c using the peak Q, T_c, and I for each of the independent drainage systems at the junction. The general equation requires that contributing Q's be numbered in order of increasing T_c.

Let Q_1, T_1, and I_1 correspond to the tributary area with the shortest T_c. Likewise, let Q_2, T_2, and I_2 correspond to the tributary area with the next longer T_c. Continuing ranking Q's, T_c's, and I's according to increasing T_c, until all contributing drainage areas to the junction are ranked. If only two independent drainage systems are combined, only Q_1, T_1, I_1, Q_2, T_2, and I_2 will be in the equation. Combine the independent drainage systems using the Junction Equations (3-11):

$$
\begin{aligned}
Q_{T1} &= Q_1 + \frac{T_1}{T_2}Q_2 + \frac{T_1}{T_3}Q_3 + \cdots + \frac{T_1}{T_n}Q_n \\
Q_{T2} &= Q_2 + \frac{I_2}{I_1}Q_1 + \frac{T_2}{T_3}Q_3 + \cdots + \frac{T_2}{T_n}Q_n \\
Q_{T3} &= Q_3 + \frac{I_3}{I_1}Q_1 + \frac{I_3}{I_2}Q_2 + \cdots + \frac{T_3}{T_n}Q_n \\
&\vdots \\
Q_{Tn} &= Q_n + \frac{I_n}{I_1}Q_1 + \frac{I_n}{I_2}Q_2 + \cdots + \frac{I_n}{I_{n-1}}Q_{n-1}
\end{aligned}
\tag{3-11}
$$

Calculate Q_{T1}, Q_{T2}, Q_{T3}, up to Q_{Tn}. Select the largest Q and use the T_c associated with that Q for further calculations (see Note #1 and Note #2 below for options). If the largest calculated Q's are equal (e.g., $Q_{T1} = Q_{T2} > Q_{Tn}$), use the shorter of the T_c's associated with that Q.

This equation may be expanded for a junction of more independent drainage systems using the same procedure. In general, when the Q from a selected subarea (e.g., Q_2) is combined with Q from another subarea with a shorter T_c (e.g., Q_1), the Q from the subarea with the shorter T_c is reduced by the ratio of the rainfall intensities (I_2/I_1); and when the Q from a selected subarea (e.g., Q_2) is combined with the Q from another subarea with a longer T_c (e.g., Q_3), the Q from the subarea with the longer T_c is reduced by the ratio of the T_c's (T_2/T_3).

Note #1: At a junction of two independent drainage systems that have the same T_c, the tributary flows may be added to obtain the Q_p: $Q_p = Q_1 + Q_2$; when $T_1 = T_2$; and $T_c = T_1 = T_2$. This can be verified by using the junction equation above. Let Q_3, T_3, and $I_3 = 0$. When T_1 and T_2 are the same, I_1 and I_2 are also the same, and T_1/T_2 and $I_2/I_1 = 1$. T_1/T_2 and I_2/I_1 are cancelled from the equations. At this point, $Q_{T1} = Q_{T2} = Q_1 + Q_2$.

Note #2: In the upstream part of a watershed, a conservative computation is acceptable. When the times of concentration (T_c's) are relatively close in magnitude (within 10%), use the shorter T_c for the intensity and the equation $Q_p = \sum_{n=1}^{\#\text{ of subareas}} (C_n \cdot A_n) I$.

3.5 Example of Rational Method

A developer is sizing a storm inlet for a site that is to be developed. Plans are to develop the site with single family residential homes on ½ acre lots. For this example, a 50-year return period will be used.

From topographic data and a field survey, the area of the drainage basin upstream of the culvert is found to be 41.9 acres. In addition the following data were measured or determined from proposed plans:

Length of overland flow = 570 feet

Slope of overland flow = 3.5%

Length of gutter flow = 1,500 feet

Slope of gutter = 2.2%

Figure 3-5 is a sketch of the site with key Rational Method calculation points defined in Table 3-7.

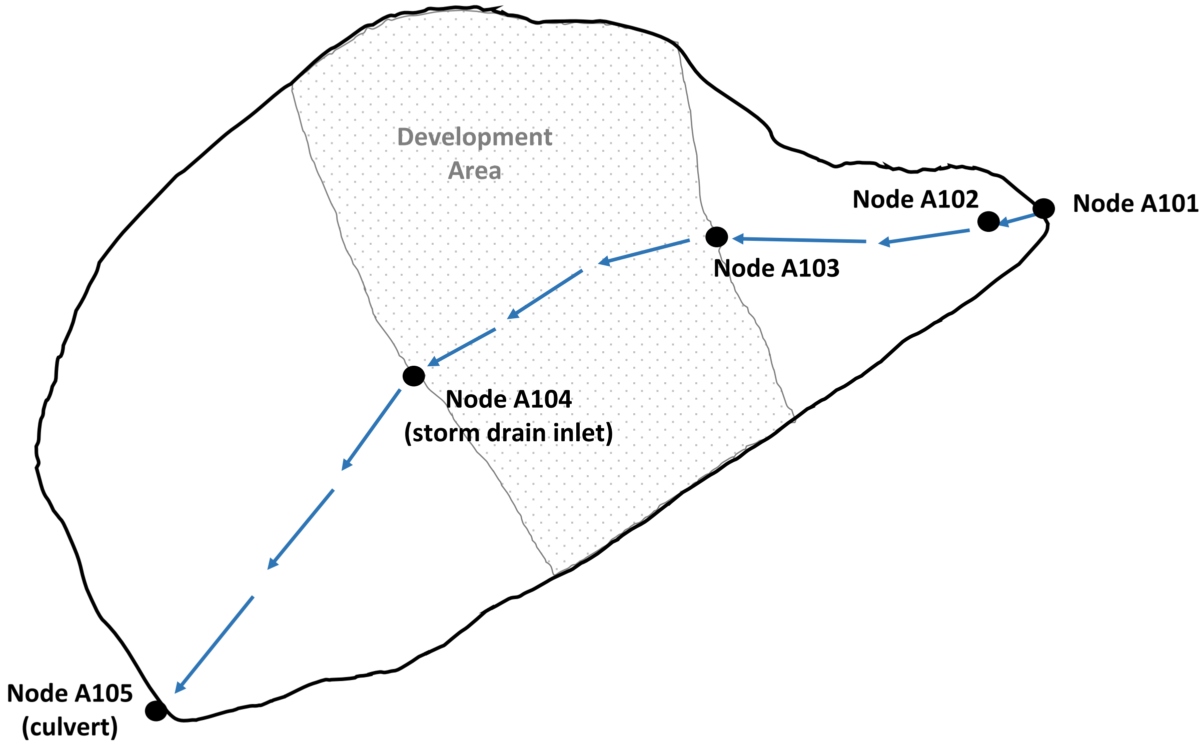


Figure 3-5. Rational Method Example Site

Table 3-7. Rational Method Example Node Descriptions

Location	Description
Node A101	most remote hydraulic point location
Node A102	beginning of shallow overland flow
Node A103	beginning of gutter flow
Node A104	storm drain inlet

After a review of topography and site development plans, key data is summarized in Table 3-8.

Table 3-8. Rational Method Example – Key Data

Watercourse	Description	Length (ft)	Slope (%)	Contributing Drainage Area (ac)	Land Use	Hydrologic Soil Group
Node A101 to A102	sheet flow	100	3.5	0.4	natural	B
Node A102 to A103	shallow overland flow	470	3.5	11.5	natural	B
Node A103 to A104	gutter flow	1,500	2.2	30.0	Residential (low density, 2 DU/A)	B

To use the Rational Method, the initial time of concentration must first be determined. From development plans, the most hydraulically remote point is "Natural" land use and the slope is 3.5%. From Table 3-4 and a slope of 3.5%, it is determined the maximum length of sheet flow is 100 feet. The drainage area for the initial sheet flow runoff is determined from the plans to be 0.4 acre. From Table 3-3 the runoff coefficient is determined to be C = 0.25. Because the return period is 50 years, the runoff coefficient is modified using Table 3-1 The sheet flow runoff coefficient is C = 0.25 x 1.2 = 0.30. To estimate T_i, Equation (3-5) is used. Evaluating Equation (3-5):

$$T_i = \frac{1.8(1.1\text{-}0.30)\sqrt{100}}{3.5^{1/3}} = 9.5 \text{ minutes}$$

The length of overland flow was determined to be 570 feet. The first 100 feet is sheet flow and the remaining 470 feet is shallow overland flow. The travel time (T_t) for this portion is determined using Equation (3-6). The natural area is nearly bare so an intercept coefficient (k) of 0.31 is assigned. The slope is 3.5%.

$$V = (3.28)\cdot(0.31)\cdot\sqrt{3.5} = 1.9 \text{ feet/second}$$

The shallow overland flow travel time is,

$$T_t = {}^{470 \text{ feet}}\Big/_{1.9\ \frac{\text{feet}}{\text{second}}} = 247 \text{ sec} = 4.1 \text{ minutes.}$$

Rainfall intensity determination is an iterative process based on the total T_c. The sheet flow and shallow overland flow travel time is 13.6 minutes (9.5 minutes + 4.1 minutes). Rainfall intensity is determined using NOAA Atlas 14. Using the latitude and longitude of the site, NOAA Atlas 14, the 50-year rainfall value for 10 minutes is 0.573 inches and 15 minutes is 0.693 inches. After interpolating to obtain an intensity value for 13.6 minutes, I = 2.96 inches/hour.

Travel time in the gutter is a function of discharge and slope and can be determined using Figure 3-3. Discharge in the gutter is from the area along the length of gutter flow in addition to the sheet flow and shallow overland flow contributing areas. The area contributing to sheet flow was determined to be 0.4 acre. The area contributing to shallow overland flow is determined to be 11.5 acres. Since soil type and land use are the same, the runoff coefficient for the shallow concentrated flow is determined to be the same as for sheet flow. Use Equation (3-9) to estimate discharge at the upstream end of the gutter:

$$Q_p = \Sigma(C \cdot A)I = [(CA_{A101\text{-}A102}) + (CA_{A102\text{-}A103})]\ I$$

$$Q_p = \Sigma(C \cdot A)I = [(0.3 \times 0.4) + (0.3 \times 11.5)]\ (2.96) = 10.6 \text{ cfs}$$

The area contributing flow directly to the 1,500 feet of gutter is determined to be 30 acres (denoted as $A_{A103\text{-}A104}$). The gutter is not a closed conduit and velocity in the gutter depends on discharge. For this reason, travel time in the gutter must be determined in an iterative fashion. To find velocity, assume an average Q over the gutter length (discharges for small watersheds typically range from 2 to 3 cfs per acre, depending on land use, drainage area, slope, and rainfall intensity), and proceed as follows:

1) Assume the average discharge in the gutter is the upstream discharge plus the average inflow into the gutter along the watercourse

$$Q_{AVG} = Q_{A103} + (\text{average Q per ac})\ \frac{(A_{A103\text{-}A104})}{2}$$

$$Q_{AVG} = 10.57 \text{ cfs} + \left(2.5\ \frac{\text{cfs}}{\text{acre}}\right) \frac{(30 \text{ acre})}{2} = 48.2 \text{ cfs}$$

2) Using the gutter discharge, slope (2.2%) and Figure 3-3)

$$V = 5.6\ \frac{\text{feet}}{\text{second}}$$

3) Calculate travel time in the gutter, $T_{t\text{-gutter}}$

$$T_{t\text{-gutter}} = \frac{1{,}500 \text{ feet}}{5.6\ \frac{\text{feet}}{\text{second}}} = 267.9 \text{ seconds} = 4.5 \text{ minutes}$$

4) Calculate time of concentration, T_c from sheet flow, shallow concentrated flow and gutter flow times

$$T_C = 9.5 \text{ minutes} + 4.1 \text{ minutes} + 4.5 \text{ minutes} = 18.1 \text{ minutes}$$

5) Re-determine rainfall intensity using NOAA Atlas 14 and a time of 18.1 minutes. After interpolation, I = 2.59 inches/hour.

6) Check the Q_{AVG} assumption of 48.2 cfs,

$$Q_p = \Sigma(C \cdot A)I \rightarrow Q_{A104} = (CA_{A101} + CA_{A102} + CA_{A103})\ I$$

$$Q_{A104} = [(0.3\times0.4)+(0.3\times11.5)+(0.3\times30)]\ (2.59) = 32.6\ \text{cfs}$$

$$32.6 \neq 48.2\ \text{cfs}$$

7) Since the assumption of average runoff of 2.5 cfs was incorrect, make a different assumption and re-calculate.

8) Re-calculate Q_p at the upstream end of the gutter,

$$Q_p = \Sigma(C\cdot A)I = [(0.3\times0.4)+(0.3\times11.5)]\ (2.59) = 9.3\ \text{cfs}$$

9) Assume a different average discharge per acre (1.55 cfs/acre, this time)

$$Q_{AVG} = 9.3\ \text{cfs} + \left(1.55\ \frac{\text{cfs}}{\text{acre}}\right)\frac{(30\ \text{acre})}{2} = 32.3\ \text{cfs}$$

10) Using the new gutter discharge, slope and Figure 3-3

$$V = 5.1\ \frac{\text{feet}}{\text{second}}$$

11) Re-calculate travel time in the gutter, $T_{t\text{-gutter}}$

$$T_{t\text{-gutter}} = \frac{1{,}500\ \text{feet}}{5.1\ \frac{\text{feet}}{\text{second}}} = 294.1\ \text{second} = 4.9\ \text{minutes}$$

12) Re-calculate time of concentration, T_c from sheet flow, shallow concentrated flow and gutter flow times

$$T_C = 9.5\ \text{minutes} + 4.1\ \text{minutes} + 4.9\ \text{minutes} = 18.5\ \text{minutes}$$

13) Re-determine rainfall intensity using NOAA Atlas 14 and a time of 18.5 minutes. After interpolation, I = 2.57 inches/hour.

14) Check the Q_{AVG} assumption of 32.3 cfs,

$$Q_{A104} = [(0.3\times0.4)+(0.3\times11.5)+(0.3\times30)]\ (2.57) = 32.3\ \text{cfs}$$

$$32.3 = 32.3\ \text{cfs}$$

15) Check that conditions relating to exceptions to applying the Rational Method do not exist:

a) There is not a highly impervious section at the most downstream area of the watershed with the total upstream area flowing through a lower impervious area.

b) There is not a smaller, less pervious area tributary to the larger primary watershed.

Therefore, the estimated 50-year return period peak discharge at the inlet is 32.3 cfs.

3.6 Example - Modified Rational Method

A developer is sizing a storm inlet at the junction between a new site under development and two existing, independent drainage systems. The site under development is the small urban watershed of the previous example where the RM was applied. The small urban watershed is to be connected to an existing drainage system comprised by two additional independent watersheds. The total peak flow at the junction resulting from the contributions of the small urban watershed under development and the two independent drainage watersheds will be computed using the MRM.

Figure 3-6 is a sketch of the watershed considered for the Modified Rational Method. The watershed is composed of three independent drainage systems labelled A, B and C. System A is the small watershed under development considered in the previous example. System B and C are the two additional independent drainage systems. The three drainage systems have storm runoff that drains to the junction node labelled D101. The description of the nodes is reported in Table 3-9 and the key data for each system are defined in Table 3-10. Subareas have been defined based on land use, topography, and drainage structures, and node numbers have been placed at points of interest. The procedure for calculating flow at the junction using the MRM is described in the text below.

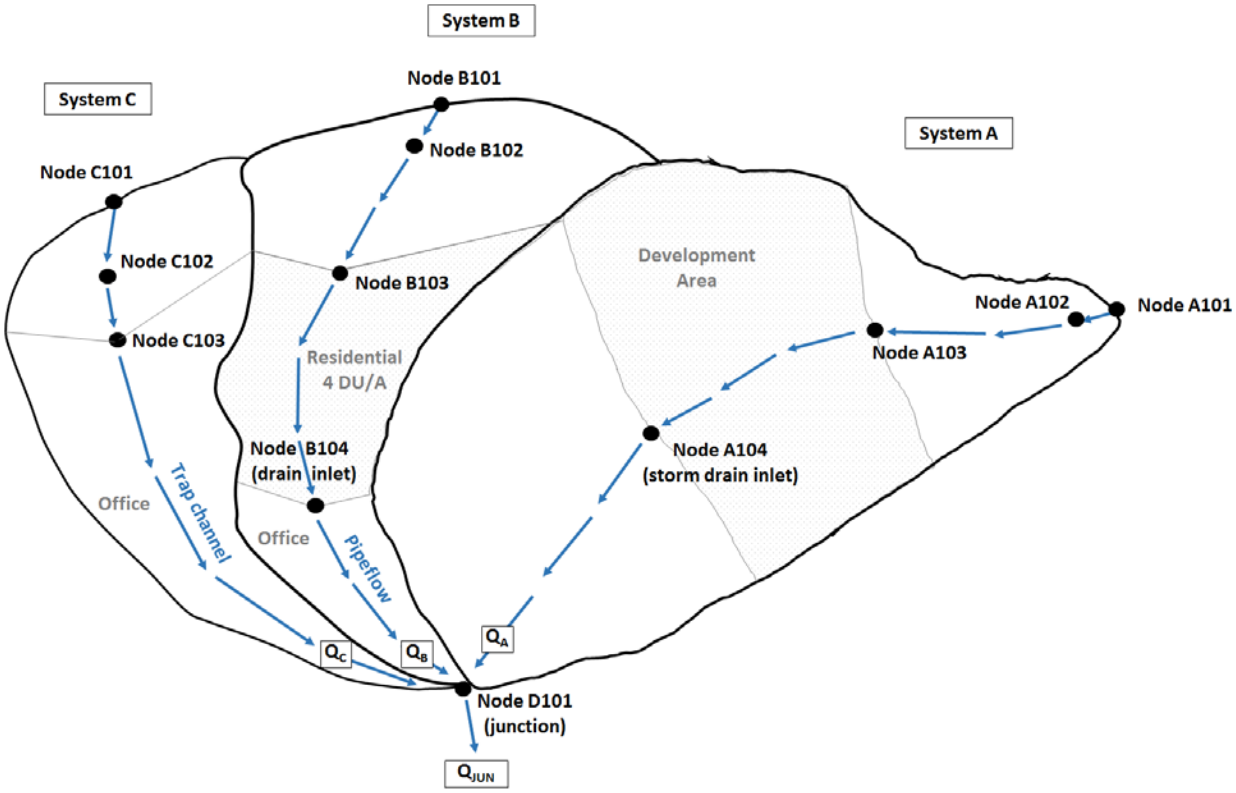


Figure 3-6. Modified Rational Method Example Site

Table 3-9. Modified Rational Method Example – Node Descriptions

Independent drainage system	Location	Description
A	Node A101	most remote hydraulic point location
	Node A102	beginning of shallow overland flow
	Node A103	beginning of gutter flow
B	Node B101	most remote hydraulic point location
	Node B102	beginning of shallow overland flow
	Node B103	beginning of gutter flow
	Node B104	storm drain inlet, beginning of pipe flow
C	Node C101	most remote hydraulic point location
	Node C102	beginning of shallow overland flow
	Node C103	beginning of trap channel

Table 3-10. Modified Rational Method Example – key data

System	Watercourse	Description	Length (ft)	Slope (%)	Drainage Area (ac)	Land Use	Hydrologic Soil Group
A	**Node A101 to A102**	sheet flow	100	3.5	0.4	natural	B
	Node A102 to A103	shallow overland flow	470	3.5	11.5	natural	B
	Node A103 to A104	gutter flow	1,500	2.2	30.0	Residential (low density, 2 DU/A)	B
B	**Node B101 to B102**	sheet flow	85	2.0	0.6	natural	B
	Node B102 to B103	shallow overland flow	515	2.0	7.8	natural	B
	Node B103 to B104	gutter flow	1,000	1.8	25	Residential (medium density, 4 DU/A)	B
	Node B104 to D101	pipe flow	850	1.4	15	office commercial	B
C	**Node C101 to C102**	sheet flow	70	2.5	1	natural	D
	Node C102 to C103	shallow overland flow	130	2.5	9	natural	D
	Node C103 to D101	trapezoidal channel	2,500	1.2	35	office commercial	D

The flow from System A was computed in the previous example and is equal to 32.3 cfs with a time of concentration of 18.5 minutes, a rainfall intensity of 2.57 inches/hour and a drainage area of 41.9 acres. The flow from System B was computed to be 41.7 cfs with a time of concentration of 22.0 minutes, a rainfall intensity of 2.39 inches/hour and a drainage area of 48.4 acres. The flow from System C was computed to be 89.9 cfs with a time of concentration of 18.0 minutes, a rainfall intensity of 2.60 inches/hour and a drainage area of 45.0 acres. The computation for each independent system can be performed with the RM as shown in the previous example. Table 3-11 presents a summary of the results.

Table 3-11. Modified Rational Method Example – Summary of discharges

System	Time of concentration (min)	Rainfall intensity (in/hr)	Drainage area (ac)	Peak discharge (cfs)	Symbols
A	18.5	2.57	41.9	32.3	T_A, I_A, Q_A
B	22.0	2.39	48.4	41.7	T_B, I_B, Q_B
C	18.0	2.60	45.0	88.3	T_C, I_C, Q_C

Once the T_c, I and peak Q are known for each independent drainage system, they need to be sorted based on increasing time of concentration. This step is required in order to establish the time at which the flows from each independent drainage system reach the junction point. Once the time of concentrations, intensities and discharges are sorted, Equations (3-11) are applied to combine them and compute the junction peak flow. The MRM procedure is as follows:

1) Sort the peak Q based on T_c

$$T_1 < T_2 < T_3$$

$$T_C < T_A < T_B$$

$$\begin{cases} T_1 = T_C = 18.0 \\ T_2 = T_A = 18.5 \\ T_3 = T_B = 22.0 \end{cases} \quad \begin{cases} I_1 = I_C = 2.60 \\ I_2 = I_A = 2.57 \\ I_3 = I_B = 2.39 \end{cases} \quad \begin{cases} Q_1 = Q_C = 88.3 \\ Q_2 = Q_A = 32.3 \\ Q_3 = Q_B = 41.7 \end{cases}$$

2) Apply equations (3-11) for each time of concentrations

$$Q_{T1} = Q_1 + \frac{T_1}{T_2} Q_2 + \frac{T_1}{T_3} Q_3$$

$$Q_{T1} = Q_C + \frac{T_C}{T_A} Q_A + \frac{T_C}{T_B} Q_B$$

$$Q_{T1} = 88.3 + \frac{18.0}{18.5} 32.3 + \frac{18.0}{22.0} 41.7 = 153.8 \text{ cfs}$$

$$Q_{T2} = Q_2 + \frac{I_2}{I_1} Q_1 + \frac{T_2}{T_3} Q_3$$

$$Q_{T2} = Q_A + \frac{I_A}{I_C} Q_C + \frac{T_A}{T_B} Q_B$$

$$Q_{T2} = 32.3 + \frac{2.57}{2.60} 88.3 + \frac{18.5}{22.0} 41.7 = \ 154.6 \text{ cfs}$$

$$Q_{T3} = Q_3 + \frac{I_3}{I_1} Q_1 + \frac{I_3}{I_2} Q_2$$

$$Q_{T3} = Q_B + \frac{I_B}{I_C} Q_C + \frac{I_B}{I_A} Q_A$$

$$Q_{T2} = 41.7 + \frac{2.39}{2.60} 88.3 + \frac{2.39}{2.57} 32.3 = \ 152.9 \text{ cfs}$$

3) Identify the largest Q and use the T_c associated with that Q and select it for the junction peak flow. Note that if the largest calculated Q's are equal (e.g., $Q_{T1} = Q_{T2} > Q_{T3}$), use the shorter of the T_c's associated with that Q.

$$Q_{JUN} = \max(Q_{T1}, Q_{T2}, Q_{T3}) = Q_{T2} = 154.6$$

$$T_{JUN} = T_2 = 18.5$$

Therefore, the estimated peak discharge and time of concentration at the junction are 154.6 cfs and 18.5 min, respectively. These estimates could be used to route the peak downstream to a new point of interest using the RM.

4 Large Area Hydrologic Procedure – NRCS Hydrologic Method

The Natural Resources Conservation Service (NRCS) hydrologic method requires basic data similar to the RM: drainage area, a "runoff curve number" (CN) describing the proportion of rainfall that becomes runoff, time to peak (T_p, the elapsed time from the beginning of unit effective rainfall to the peak flow at the point of concentration), and total rainfall (P). The NRCS approach is more sophisticated than the RM in that it considers the time distribution of rainfall, initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. Rainfall losses and resulting runoff should be estimated using the NRCS hydrologic method for study areas approximately 1 square mile and greater in size. The NRCS hydrograph is calculated using the synthetic unit hydrograph S-graph technique. Details of the methodology can be found in the NRCS National Engineering Handbook (NEH), Part 630 Hydrology (NEH-630) (USDA, 2010).

The NRCS hydrologic method may be used for the entire study area, or the RM or MRM may be used for approximately 1 square mile of the study area and then transitioned to the NRCS hydrologic method using the procedure described in Section 4.5. The recommended approach for applying the NRCS hydrologic method is to develop required input parameters for the method and use HEC-HMS software to perform the calculations.

4.1 General Description

The NRCS hydrologic method differs from the Rational Method in two fundamental ways: (1) the NRCS hydrologic method provides a method to estimate the amount of rainfall that is initially intercepted and *does not* contribute to runoff (precipitation losses) and an infiltration rate that decreases during a storm event while the Rational Method C factor determines what proportion of rainfall becomes runoff, and (2) the NRCS hydrologic method considers the time distribution of rainfall thus enabling the creation of a runoff hydrograph which estimates runoff discharge over a period of time whereas the Rational Method estimates only the peak discharge.

The recommended approach to precipitation losses is the NRCS Curve Number approach. Because there is little observed data for the rainfall-runoff hydrograph relationship in Imperial County, the recommended hydrograph approach is the synthetic unit hydrograph S-graph technique using calibrated s-graphs available from nearby, similar regions. A necessary component to utilizing the S-graph is watershed lag which should be calculated using the U.S. Army Corps of Engineers (Corps) method (1976). The large area hydrologic method includes the following steps:

1) Determination of rainfall losses and runoff,

2) S-graph selection, and

3) Hydrograph calculation using HEC-HMS.

4.2 NRCS Precipitation Losses and Runoff

The storm runoff hydrograph from a drainage area is based in part on the physical characteristics of the watershed. The principal physical watershed characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types, and land slope. The NRCS method uses a combination of soil conditions, land uses (ground cover) and land treatment (generally agricultural practices) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CNs), indicate the runoff potential of an area. The higher the CN, the higher the runoff potential. The CN does not account for land slope. However, in the NRCS hydrologic method, land slope is accounted for in the determination of watershed lag time (see Section 4.2.5). The steps for estimating rainfall runoff are:

1) Delineate the watershed on a map and determine watershed physical characteristics including location of centroid, total length of longest watercourse, length along the watercourse to location nearest the centroid, soil type, and land use/land treatment,

2) Determine a composite curve number (CN) for the watershed, which will represent the combination of land use and soil type within the drainage area and describe the proportion of rainfall that runs off,

3) Determine frequency of the design storm, total rainfall amount for the design storm and *Antecedent Runoff Condition* (ARC) for the watershed location,

4) Adjust CN based on the Antecedent Runoff Condition (ARC),

5) Prepare the incremental rainfall distribution,

6) Determine the excess rainfall amounts using the composite CN for the watershed and the depth-area adjusted incremental rainfall distribution.

7) Select an appropriate S-graph,

8) Use the HEC-HMS software to compute a runoff hydrograph.

The CN values in Table 4-1, Table 4-2 and Table 4-3 are suitable for preparing hydrographs in accordance with the methods shown in Chapters 10 and 16 of NEH-630 and described in Section 4.2 of this manual. The CN values are based on hydrologic soil group and land use/land treatment. When a drainage area has more than one land use, hydrologic soil group or hydrologic condition, a composite CN should be calculated and used in the analysis. It should be noted that when composite CNs are used, the analysis does not take into account the location of the specific land uses but treats the drainage area as a uniform land use represented by the composite CN.

4.2.1 Watershed Delineation

Once the accumulation point has been determined, watershed delineation may be accomplished by hand or using GIS methods. Depending on the size and distribution of soil types, vegetative cover, land uses and other factors affecting rainfall runoff, it may be necessary to subdivide the watershed into smaller sub-basins. Ideally, sub-basins would have similar hydrologic characteristics. Each sub-basin will be analyzed separately, creating runoff hydrographs for each which are subsequently combined creating the runoff hydrograph for the entire watershed.

Required watershed (or sub-basin) attributes for the NRCS method are: basin area, basin centroid, length (miles) of the longest watercourse from the accumulation point to the basin boundary, length (miles) along the longest watercourse from the accumulation point to a point opposite the basin centroid, average slope (feet per mile) of the longest watercourse, soil hydrologic classification (NEH-630, Chapter 7) and vegetative cover and condition.

4.2.2 Curve Number Determination

Once the watershed and sub-basins have been delineated, hydrologic soil types determined, and vegetative cover and condition estimated, the Curve Number (CN) can be estimated. The combination of soil type and vegetative cover and condition is the hydrologic soil-cover complex. If a sub-basin contains more than one complex, a composite CN for the sub-basin must be determined using a weighted area approach. A more detailed description of hydrologic soil-cover complexes and Curve Number is available in NEH-630, Chapter 9 and Chapter 10 (USDA, 2004).

Table 4-1 through Table 4-3 are from NEH-630 (USDA, 2004) and provide guidance in selecting CN based on hydrologic complex. The CNs in the table assume the initial abstraction (I_a) is equal to 20% of the total runoff retention capacity of the watershed ($I_a = 0.2S$), which is the standard assumption put forth in NEH-630 (USDA, 2004). Any assumption other than $I_a = 0.2S$ would require determination of different CNs for the hydrologic soil complexes. When impervious areas are part of the basin, it must be determined if they are connected or unconnected to the drainage system and treated accordingly. Treatment of connected and unconnected impervious areas is discussed following Table 4-1. Also note that the CN for some urban cover types assumes a certain percent imperviousness and these areas should not be double-counted.

Table 4-1. Runoff Curve Numbers for Urban Areas[1]

Cover Description			Curve Number by Hydrologic Soil Group			
Cover Type	**Hydrologic Condition**	**Average % Impervious Area[2]**	**A**	**B**	**C**	**D**
Fully developed urban areas (vegetation established):						
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]	Poor (grass cover < 50%)		68	79	86	89
	Fair (grass cover 50 to 75%)		49	69	79	84
	Good (grass cover > 75%)		39	61	74	80
Impervious areas:						
Paved parking lots, roofs, driveways, etc. (excluding un-improved right-of-way)			98	98	98	98
Streets and roads:						
Paved; curbs and storm sewers (excluding un-improved right-of-way)			98	98	98	98
Paved; open ditches (including right-of-way)			83	89	92	93
Gravel (including right-of-way)			76	85	89	91
Dirt (including right-of-way)			72	82	87	89
Western desert urban areas:						
Natural desert landscaping (pervious areas only)[4]			63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-			96	96	96	96

Cover Description			Curve Number by Hydrologic Soil Group			
Cover Type	Hydrologic Condition	Average % Impervious Area[2]	A	B	C	D
inch sand or gravel mulch and basin borders)						
Urban districts:						
Commercial and business		85	89	92	94	95
Industrial		72	81	88	91	93
Residential districts by average lot size:						
1/8 acre or less (town houses)		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 areas (pervious areas only, no vegetation)			77	86	91	94

[1] Average runoff condition and $I_a = 0.2S$.

[2] The average percent impervious area shown was used to develop composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition.

[3] CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space type.

[4] Composite CNs for natural desert landscaping should be computed using Figure 4-1 or Figure 4-2 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CNs are assumed equivalent to desert shrub in poor hydrologic condition.

Impervious areas can be connected or unconnected to the drainage system and the distinction can affect the composite CN. From USDA (2010), an impervious area is considered connected if runoff from it flows directly into the drainage system. It is also considered connected if runoff from it occurs as shallow concentrated flow running over a pervious area and into a drainage system. If all impervious area is directly connected to the drainage system, but the impervious area percentages in Table 4-1 or the pervious land use assumptions are not applicable, use Equation (4-1) or Figure 4-1 to compute a composite CN.

$$CN_C = CN_P + \left(\frac{P_{imp}}{100}\right)(98 - CN_P) \qquad (4\text{-}1)$$

Where: CN_C = composite runoff curve number,

CN_P = pervious runoff curve number,

P_{imp} = percent imperviousness.

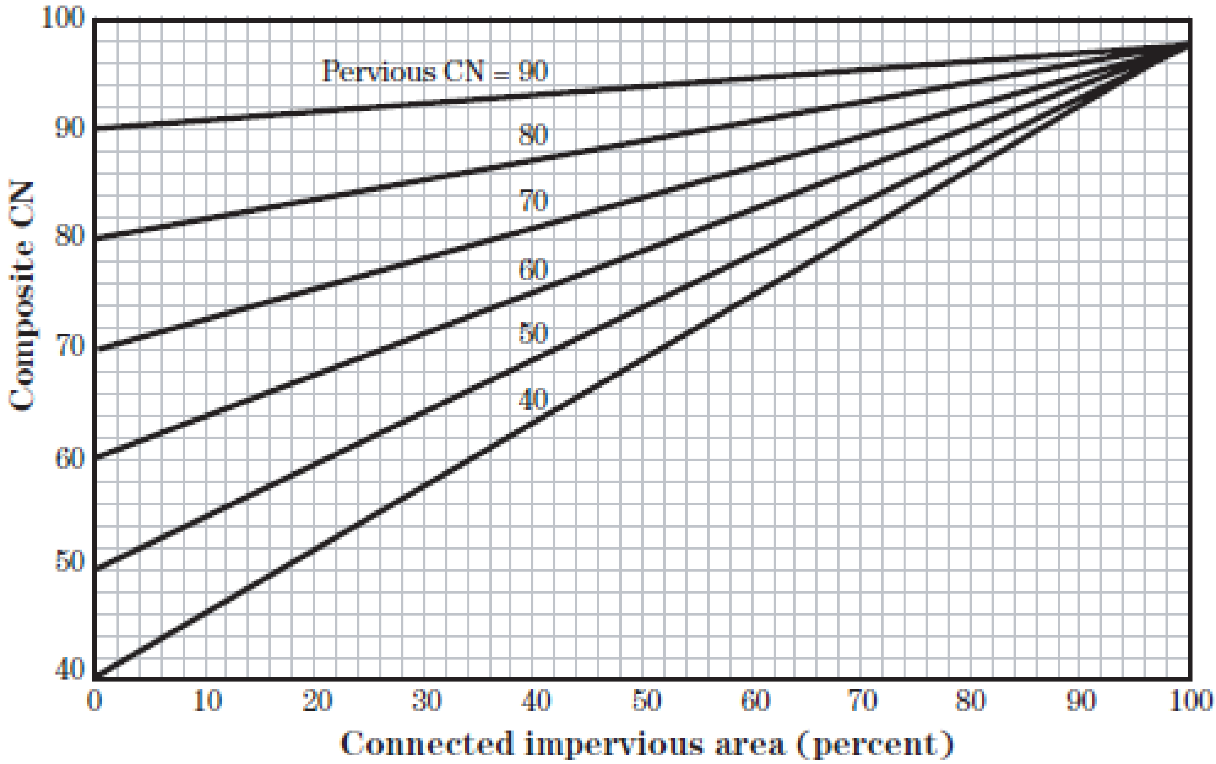


Figure 4-1. Composite CN with Connected Impervious Area (USDA, 2010)

If runoff from impervious areas flows over a pervious area as sheet flow prior to entering the drainage system, the impervious area is unconnected. To determine CN when all or part of the impervious area is not directly connected to the drainage system, use Equation (4-2) or Figure 4-2 (USDA, 2010) if the total impervious area is less than 30 percent of the total area or use Equation (4-1) or Figure 4-1 if the total impervious area is equal to or greater than 30 percent of the total area (as the absorptive capacity of the remaining pervious areas will not significantly affect runoff).

$$CN_C = CN_P + \left(\frac{P_{imp}}{100}\right)(98 - CN_P)(1 - 0.05R) \qquad (4\text{-}2)$$

Where: R = ratio of unconnected impervious area,

and other variables are as defined in Equation (4-1).

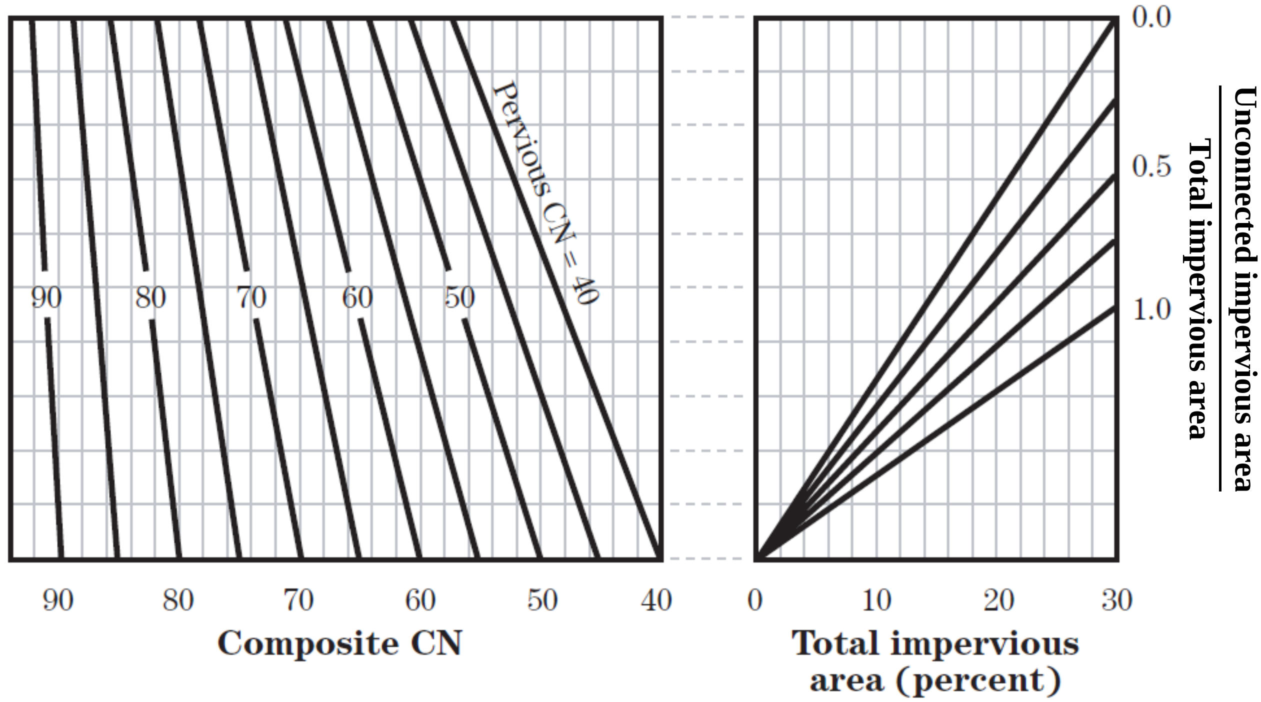


Figure 4-2. Composite CN: Unconnected Impervious Areas, Total Impervious Area < 30%

When impervious area is less than 30%, obtain the composite CN by entering the right half of Figure 4-2 with the percentage of total impervious area and the ratio of total unconnected impervious area to total impervious area. Then move horizontally to the left to the appropriate pervious CN and read down to find the composite CN.

Table 4-2. Runoff Curve Numbers for Arid and Semiarid Rangelands[1]

Cover Description		Curve Number by Hydrologic Soil Group			
Cover Type	**Hydrologic Condition[2]**	**A[3]**	**B**	**C**	**D**
Herbaceous – mixture of grass, weeds and low-growing brush, with brush the minor element	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Oak-aspen – mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple, and other brush	Poor		66	74	79
	Fair		48	57	63
	Good		30	41	48
Pinyon-juniper – pinyon, juniper, or both: grass understory	Poor		75	85	89
	Fair		58	73	80
	Good		41	61	71
Sage-grass – sage with an understory of grass	Poor		67	80	85
	Fair		51	63	70

Cover Description		Curve Number by Hydrologic Soil Group			
Cover Type	**Hydrologic Condition[2]**	**A[3]**	**B**	**C**	**D**
	Good		35	47	55
Desert shrub – major plans include saltbush, greasewood, creosotebush, blackbrush, bursage, paloverde, mesquite, and cactus	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84

[1] Average runoff condition and $I_a = 0.2S$. For range in humid regions, use Table 4-3.

[2] Poor: < 30% ground cover (litter, grass, and brush overstory).
Fair: 30 to 70% ground cover.
Good: > 70% ground cover.

[3] CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space type.

Table 4-3. Runoff Curve Numbers for Agricultural Lands[1]

Cover Description			Curve Number by Hydrologic Soil Group			
Cover Type	**Treatment[2]**	**Hydrologic Condition[3]**	**A**	**B**	**C**	**D**
Fallow	Bare soil	---	77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured and terraced (C & T)	Poor	66	74	80	82
		Good	62	71	78	81
	C & T + CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85

Cover Description			Curve Number by Hydrologic Soil Group			
Cover Type	Treatment[2]	Hydrologic Condition[3]	A	B	C	D
		Good	61	73	81	84
Small grain	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	C & T	Poor	61	72	79	82
		Good	59	70	78	81
	C & T + CR	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadow	SR	Poor	66	77	85	89
		Good	58	72	81	85
	C	Poor	64	75	83	85
		Good	55	69	78	83
	C & T	Poor	63	73	80	83
		Good	51	67	76	80
Pasture, grassland, or range-continuous forage for grazing[4]		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
Meadow-continuous grass, protected from grazing and generally mowed for hay		Good	30	58	71	78
Brush-brush-forbs-grass mixture with brush the major element[5]		Poor	48	67	77	83
		Fair	35	56	70	77
		Good	30[6]	48	65	73
Woods-grass combination (orchard or tree farm)[7]		Poor	57	73	82	86
		Fair	43	65	76	82
		Good	32	58	72	79
Woods[8]		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	30	55	70	77
Farmstead – buildings, lanes, driveways, and surrounding lots		---	59	74	82	86

Cover Description			Curve Number by Hydrologic Soil Group			
Cover Type	Treatment[2]	Hydrologic Condition[3]	A	B	C	D
Roads (including right-of-way):						
Dirt		---	72	82	87	89
Gravel		---	76	85	89	91

[1] Average runoff condition and $I_a = 0.2S$.

[2] Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.

[3] Hydrologic condition is based on combinations of factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes, (d) percent of residue cover on the land surface (good $\geq$ 20%), and (e) degree of surface toughness.
Poor: Factors impair infiltration and tend to increase runoff.
Good: Factors average and better than average infiltration and tend to decrease runoff.30 to 70% ground cover.
For conservation tillage poor hydrologic condition, 5 to 20% of the surface is covered with residue (< 750 pounds per acre for row crops or 300 pounds per acre for small grain.)
For conservation tillage good hydrologic condition, more than 20% of the surface is covered with residue (> 750 pounds per acre for row crops or 300 pounds per acre for small grain.)

[4] Poor: < 50% ground cover or heavily grazed with no mulch.
Fair: 50 to 75% ground cover and not heavily grazed.
Good: > 75% ground cover and lightly or only occasionally grazed.

[5] Poor: < 50% ground cover.
Fair: 50 to 75% ground cover.
Good: > 75% ground cover.

[6] If actual CN is less than 30, use CN = 30 for runoff computation.

[7] CNs shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CNs for woods and pastures.

[8] Poor: Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair: Woods are grazed, but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

4.2.3 Rainfall and the Antecedent Runoff Condition (ARC)

Determination of design storm frequency is based on County and project requirements. Once the design storm frequency has been determined, rainfall amounts can be obtained by following the procedure in Section 2.2.

Basin conditions at the onset and during a storm can affect the quantity of runoff. Factors including rainfall intensity and duration, total rainfall, soil moisture conditions, cover density, stage of growth and temperature can all contribute to variability in the amount of rainfall that becomes runoff. Collectively these factors are called the Antecedent Runoff Condition (ARC). ARC is divided into three classes: II for average conditions, I for dryer than normal conditions, and III for wetter than normal conditions. Provided adequate justification can be made and acceptable conservatism demonstrated, an ARC adjustment to CNs may be valid. In general a design ARC Class II should be used.

4.2.4 Antecedent Runoff Condition adjustment values

CN values presented in Table 4-1 through Table 4-3 assume an ARC II condition. ARC II CN values and the corresponding ARC I and ARC III values are presented in Table 4-4.

Table 4-4. Antecedent Runoff Condition (ARC) CN Values

Curve Number by ARC										
ARC II	ARC I	ARC III		ARC II	ARC I	ARC III		ARC II	ARC I	ARC III
100	100	100		**74**	55	88		**48**	29	68
99	97	100		**73**	54	87		**47**	28	67
98	94	99		**72**	53	86		**46**	27	66
97	91	99		**71**	52	86		**45**	26	65
96	89	99		**70**	51	85		**44**	25	64
95	87	98		**69**	50	84		**43**	25	63
94	85	98		**68**	48	84		**42**	24	62
93	83	98		**67**	47	83		**41**	23	61
92	81	97		**66**	46	82		**40**	22	60
91	80	97		**65**	45	82		**39**	21	59
90	78	96		**64**	44	81		**38**	21	58
89	76	96		**63**	43	80		**37**	20	57
88	75	95		**62**	42	79		**36**	19	56
87	73	95		**61**	41	78		**35**	18	55
86	72	94		**60**	40	78		**34**	18	54
85	70	94		**59**	39	77		**33**	17	53
84	68	93		**58**	38	76		**32**	16	52
83	67	93		**57**	37	75		**31**	16	51
82	66	92		**56**	36	75		**30**	15	50
81	64	92		**55**	35	74		**25**	12	43
80	63	91		**54**	34	73		**20**	9	37
79	62	91		**53**	33	72		**15**	6	30
78	60	90		**52**	32	71		**10**	4	22
77	59	89		**51**	31	70		**5**	2	13
76	58	89		**50**	31	70		**0**	0	0
75	57	88		**49**	30	69				

Once basin CN estimates have been finalized, a storm hyetograph is prepared.

4.2.5 Preparation of incremental rainfall distribution

The variation in rainfall intensity that occurs from the beginning of the storm through the storm peak and to the end of the storm is represented in the time distribution of rainfall. The time distribution of rainfall during a storm should be tabulated and can be represented graphically as a hyetograph, a chart showing increments of average rainfall during successive units of time during a storm. As discussed in Section 2.4, the rainfall distribution pattern adopted by Imperial County is a nested storm pattern with 2/3, 1/3 distribution. The time to peak (T_p) necessary for determining duration D of the hyetograph should be determined using the Corps lag method (USACE, 1976). Corps lag (T_l) in hours is expressed by the empirical formula,

$$T_l = 24\,\bar{n}\left(\frac{LL_C}{\sqrt{S}}\right)^m \qquad (4\text{-}3)$$

and time to peak, T_p, is

$$T_p = 0.862\,T_l \qquad (4\text{-}4)$$

Where: $\bar{n}$ = the visually estimated mean of all Manning's *n* values for watercourses in the basin,

L = length of the longest watercourse in miles,

L_C = length along the longest watercourse measured from the outlet to a point opposite the basin centroid, in miles,

m = 0.38 (empirically determined coefficient estimated for Southern California),

S = slope of the longest watercourse between the outlet and the headwaters in feet per mile,

Descriptive aids for estimating the basin $\bar{n}$ factor, based on Plate 21 from USACE (1976) are:

$\bar{n}$ = 0.015, drainage area has fairly uniform, gentle slopes with most watercourses either improved or along paved streets. Ground cover consists of some grasses with appreciable areas developed to the extent that a large percentage of the area is impervious. Main watercourse is improved channel or conduit.

$\bar{n}$ = 0.020, drainage area has some graded and non-uniform, gentle slopes with over half of area watercourses either improved or along paved streets. Ground cover consists of equal amount grasses and impervious area. Main watercourse is partly improved channel or conduit and partly greenbelt (unimproved).

n̄ = 0.025, drainage area is generally rolling with gentle slopes and some drainage improvements in the area such as streets and canals. Ground cover consists mostly of scattered brush and grass with a low % impervious area. Main watercourse is straight channel with turf or stony bed and weeds on earthen bank.

n̄ = 0.030, drainage area is generally rolling, with rounded ridges and moderate side slopes and no drainage improvements in the area. Ground cover includes scattered brush and grasses. Watercourses meander in fairly straight, unimproved channels with some boulders and lodged debris.

n̄ = 0.040, drainage area is steep upper canyons with moderate slopes in lower canyons and no drainage improvements in the area. Ground cover is mixed brush and trees with grasses in lower canyons. Watercourses have moderate bends and are moderately impeded by boulders and debris with meandering courses.

n̄ = 0.050, drainage area is quite rugged, with sharp ridges and narrow, steep canyons and no drainage improvements in the area. The ground cover, excluding small areas of rock outcrops, includes many trees and considerable underbrush. Watercourses meander around sharp bends, over large boulders and considerable debris obstruction.

n̄ = 0.100, the drainage area has extensive vegetation, including grass, or is farmed with contoured plowing, and streams that contain a large amount of brush, grass or other vegetation that slows water velocity.

n̄ = 0.200, the drainage area has comparatively uniform slopes with no drainage improvements. Groundcover consists of cultivated crops or substantial growths of grass and fairly dense small shrubs, cacti or similar vegetation. Surface characteristics are such that channelization dies not occur

In addition, the *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Floodplains* (USGS Water Supply paper 2339) and *Open Channel Hydraulics* by Ven Te Chow may provide supplementary guidance.

Once Corps basin lag time is determined, NRCS lag time (T_N) may be determined using (San Diego County, 2003):

$$T_N = 0.862\ T_l - \frac{D}{2} \qquad (4\text{-}5)$$

A hyetograph creation example is provided in Section 2.5. As discussed in Section 2.4, if warranted, the depth-area rainfall reduction should be applied prior to arranging the incremental rainfall amounts in the 2/3, 1/3 distribution. Tabulated and/or graphical hyetograph representations should be converted to units of inches per hour if not already determined as such.

4.2.6 Determination of excess rainfall amounts

Excess rainfall is the precipitation that becomes runoff. To estimate excess rainfall, obtain the partial duration rainfall values as described in Section 2.2, apply a depth-area reduction factor as described in Section 2.4 (if appropriate) and use HEC-HMS software, along with CN, percent impervious, NRCS lag (T_N) and the appropriate S-graph to determine the excess rainfall runoff hydrograph. The process is described in detail in Section 4.4.

4.3 S-graph selection

As previously discussed, long term rainfall and streamgage data is sparse in the County. For this reason, the S-graph method has been chosen as the preferred hydrograph calculation approach. From Caltrans (2007), because no two drainage areas have identical hydrologic characteristics, the runoff patterns from these areas are generally dissimilar and the time distribution of runoff may differ considerably. Therefore, direct transposition of the characteristic time distribution of runoff from drainage areas for which rainfall-runoff data are available to nearby areas for which data are not available is usually not advisable. The S-graph method uses a basic time-runoff relationship for a watershed type in a form suitable for application to ungaged basins.

The Desert and Foothill S-graphs of other, local Southern California regions best approximate the watershed response most likely to be present in Imperial County. The Desert and Foothill S-graphs are presented in **Error! Reference source not found.** and tabulated in Appendix A. The Foothill S-graph is for watersheds characterized by natural channels incised in canyon bottoms with overbank flows confined near the main channel. The Desert S-graph is for use in undeveloped desert areas. The recommended approach for hydrograph calculation with the S-graphs is using HEC-HMS (HMS) (USACE, 2016) software. The process is described in Section 4.4.

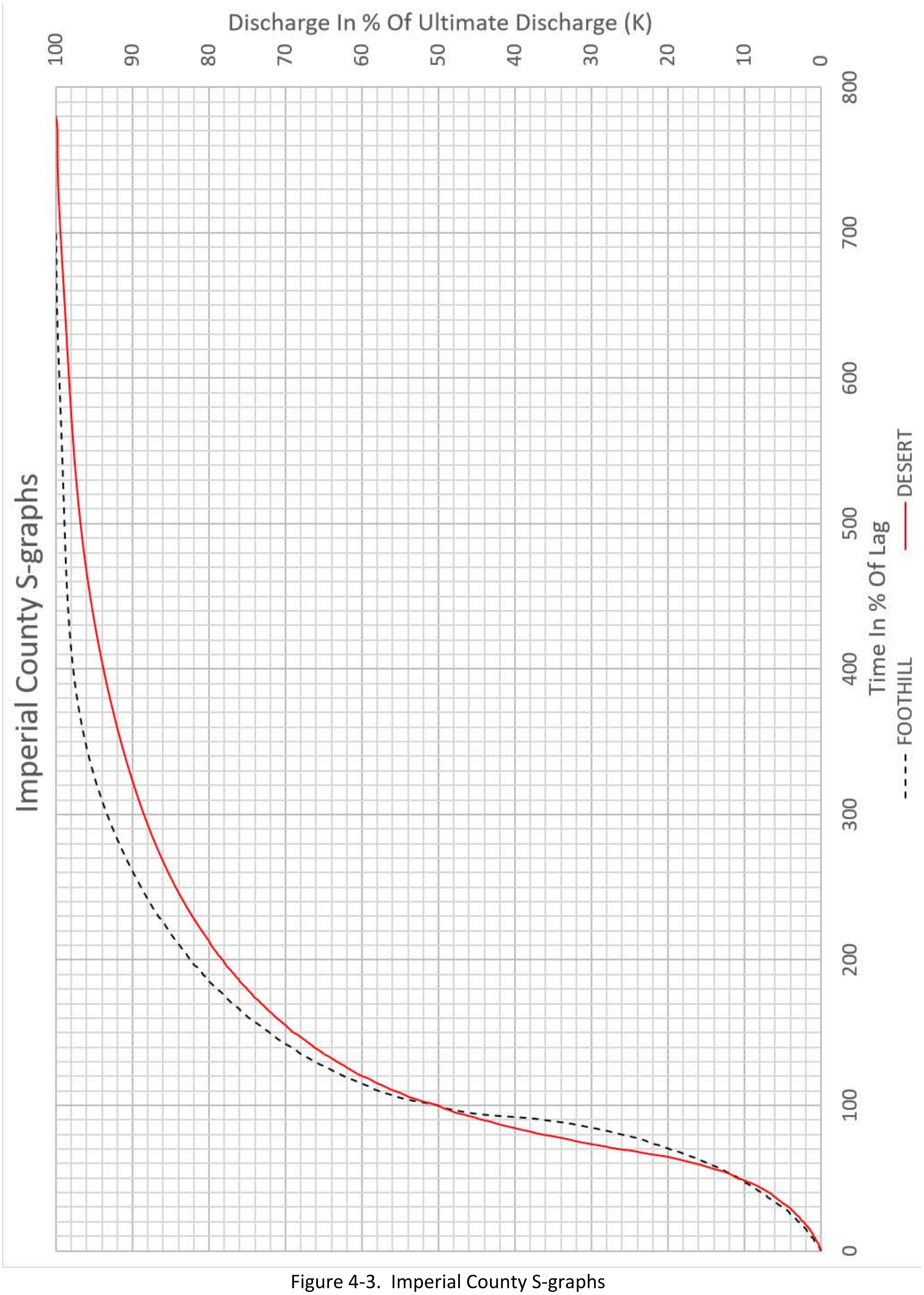


Figure 4-3. Imperial County S-graphs

4.4 Hydrograph calculation

Once an HMS project is opened, a basin hydrograph may be estimated using the following steps:

Step 1. HMS paired data creation. Use the "Components" → "Paired Data Manager" to create a "Data Type: Percentage Curves" named after the S-graph being used, as presented in Figure 4-4.

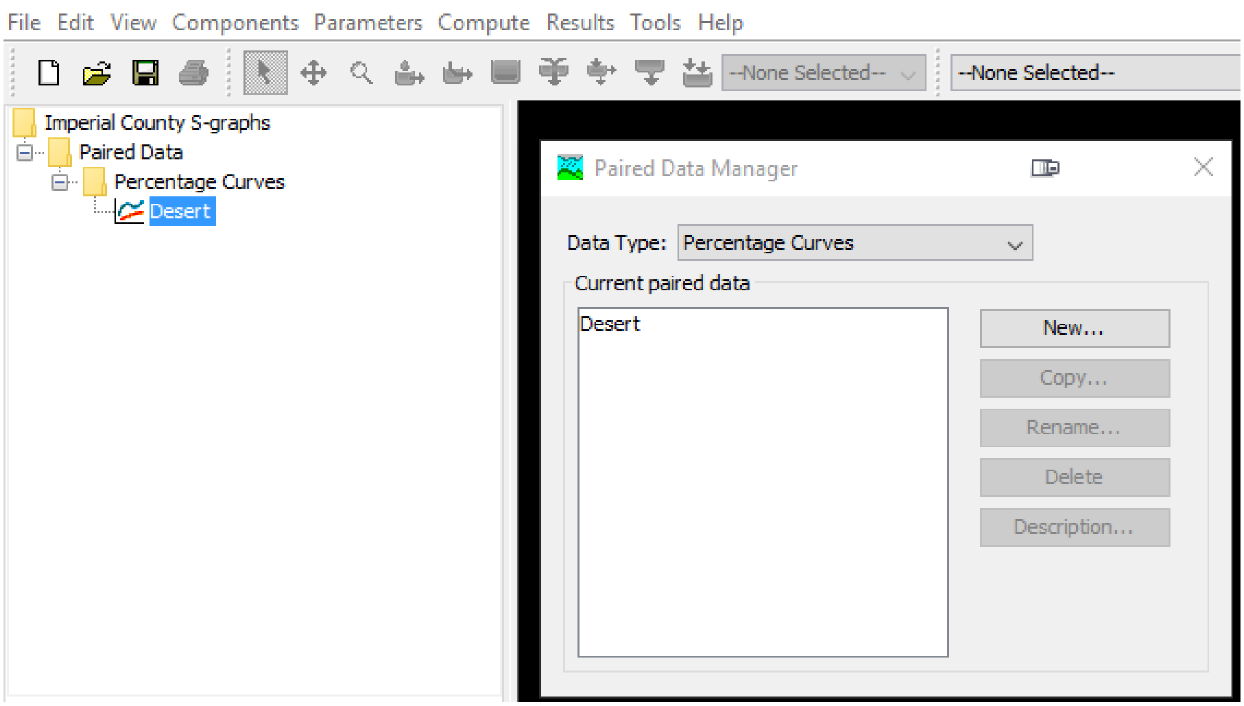


Figure 4-4. HEC-HMS paired data creation

Step 2. S-graph data entry. Select the newly created paired data type, select the "Table" data entry method and copy the proper S-graph values from Appendix A of this manual ensuring values are copied in ascending order, as presented in Figure 4-5.

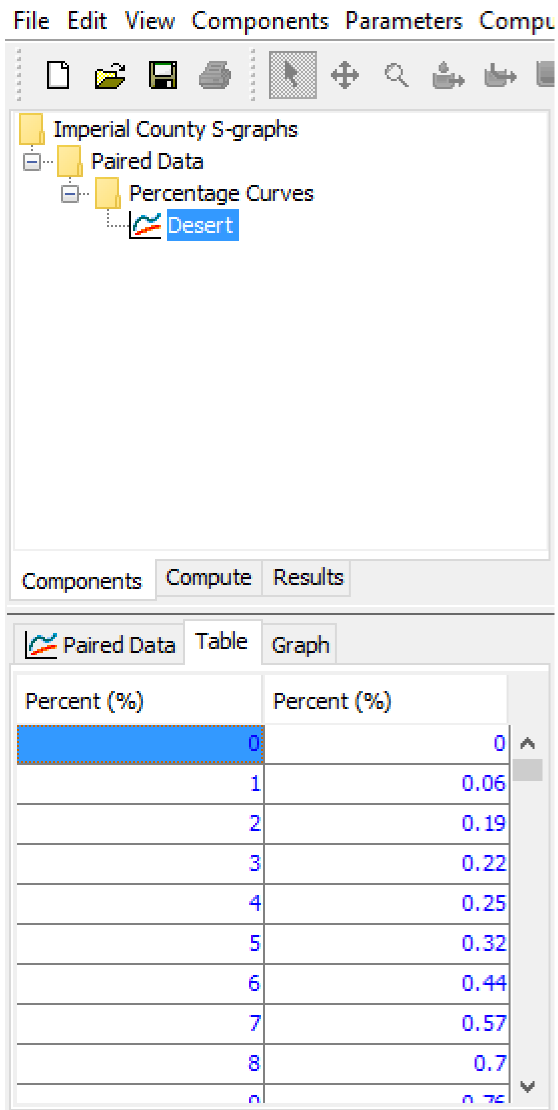


Figure 4-5. HEC-HMS S-graph data entry

Step 3. Use the "Components" → "Basin Model Manager" to create and name a basin model for the area where the hydrograph is desired. The default basin model settings as presented in Figure 4-6 are acceptable for basic hydrograph calculation.

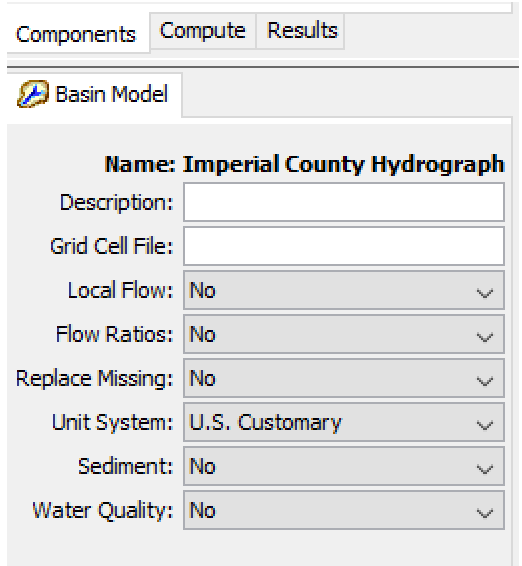


Figure 4-6. HEC-HMS basin default settings

Step 4. Using the "Subbasin Creation Tool", create and name a subbasin, enter the subbasin area, select "Loss Method" as SCS Curve Number, "Transform Method" as User-Specified S-Graph and "Baseflow Method" as –None-- as presented in Figure 4-7 and Figure 4-8.

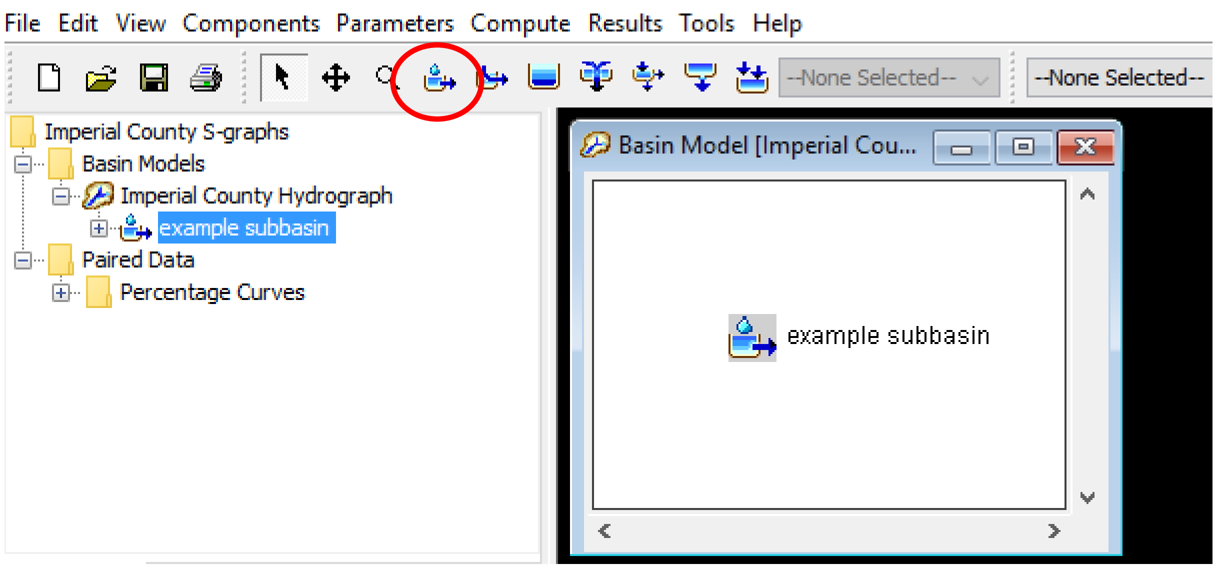


Figure 4-7. HMS subbasin creation tool

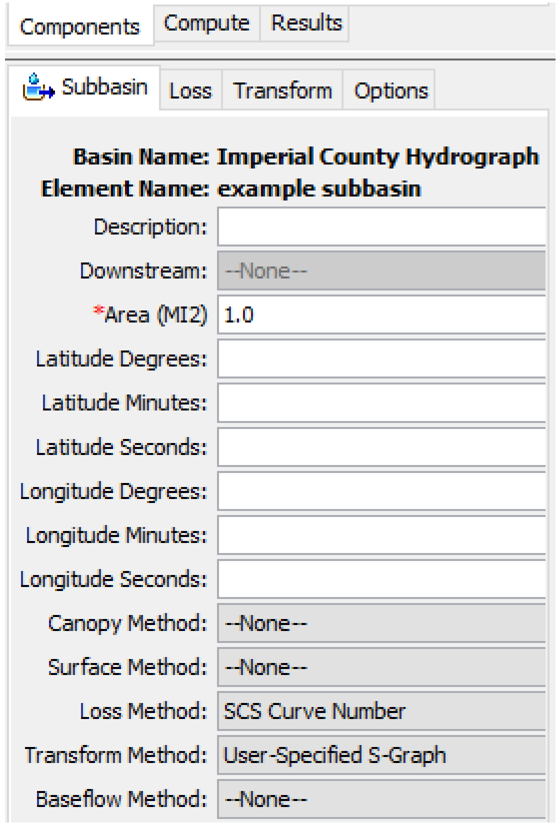


Figure 4-8. HMS subbasin area settings

Step 5. Set subbasin loss and transform parameters. As presented in Figure 4-9, select the "Loss" tab and enter a Curve Number and Impervious % as determined using the methods described in Section 4.2.2. Do not enter an Initial Abstraction (IN) value. As presented in Figure 4-10, select the "Transform" tab, select the S-graph created in Step 1 and Step 2 and enter the NRCS Lag Time determined using the Corps lag method described in Section 4.2.5.

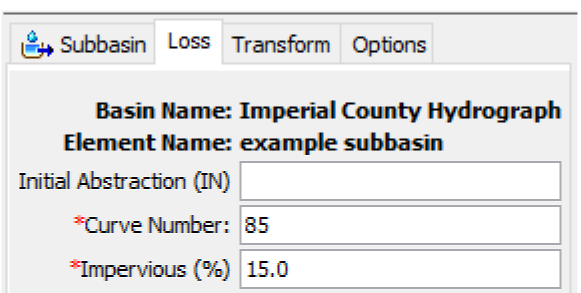


Figure 4-9. HMS S-graph loss settings

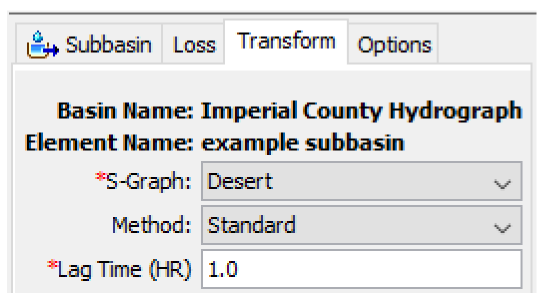


Figure 4-10. HMS S-graph transform settings

Step 6. Meteorologic Model creation. Use the "Components" → "Meteorologic Model Manager" to create and name a meteorologic model for the area where the hydrograph is desired as presented in Figure 4-11. Settings should be as presented in Figure 4-12. On the "Basins" tab, set "Include Subbasins" to "Yes" as presented in Figure 4-13.

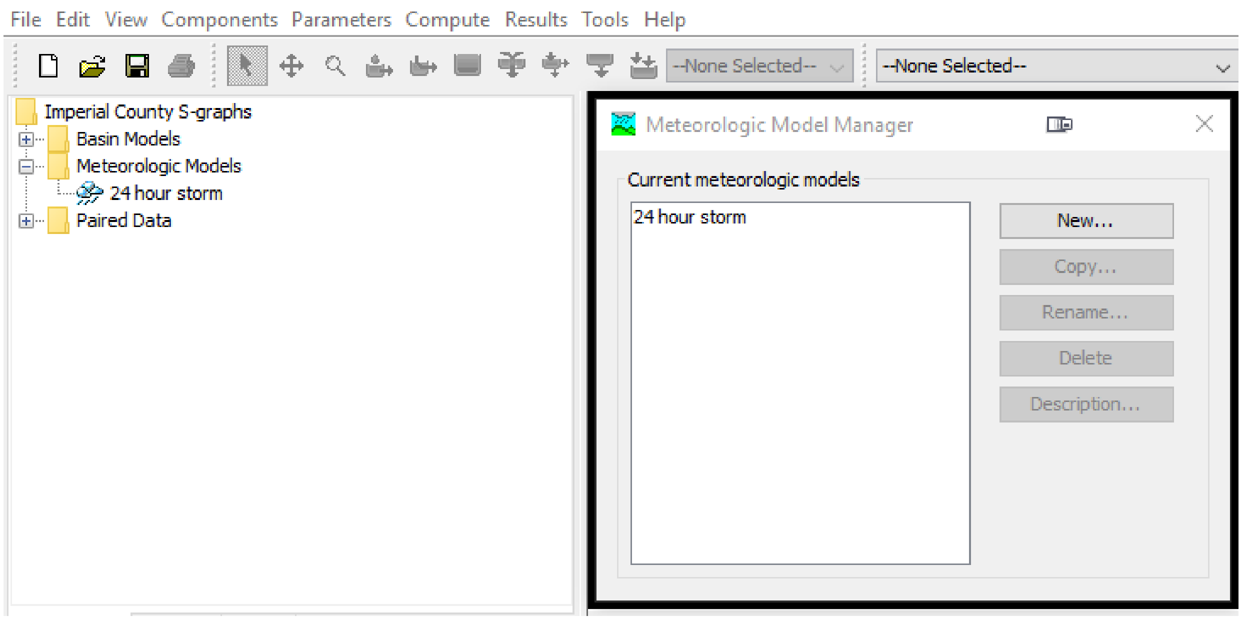
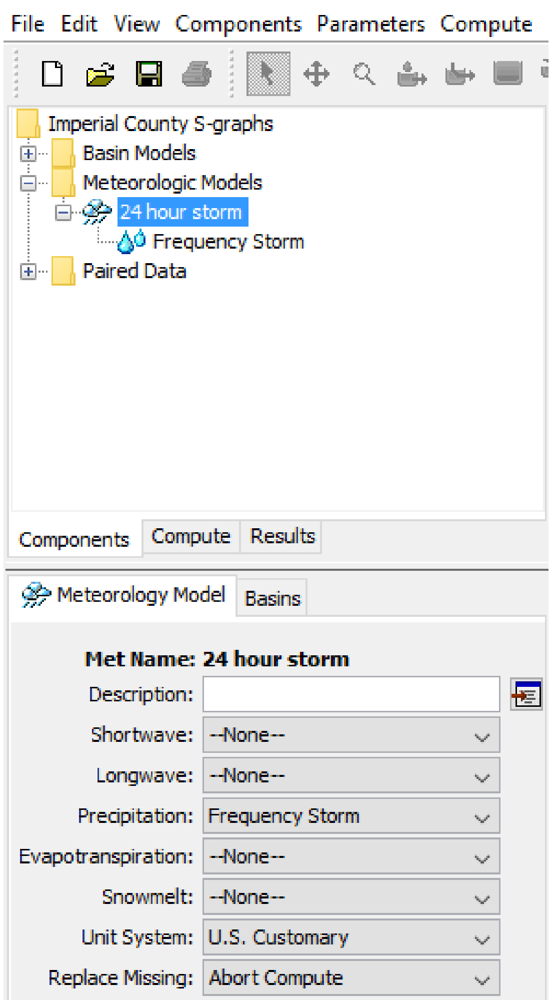


Figure 4-11. HMS Meteorologic Model creation



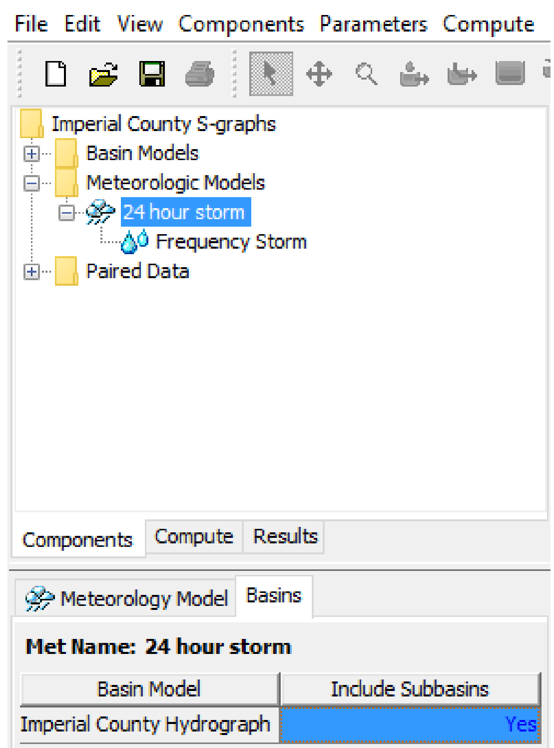


Figure 4-12. HMS Meteorologic Model settings

Figure 4-13. HMS Meteorologic Model subbasins

Step 7. Create the 1/3, 2/3 balanced hyetograph for the storm. Select “Frequency Storm” under the “Meteorologic Model” created in Step 6. Set “Storm Duration” to the design storm duration (24 hours in this example), “Intensity Position” to 67 Percent and “Storm Area (MI2)” to 1 (regardless of the watershed area.) *It is important to set “Storm Area (MI2)” to 1, otherwise the HEC-HMS default depth-area-reduction factor will be applied in addition to the area reduction already applied using the methodology in Section 2.3*. Under “Partial-Duration Depth (IN)”, enter the appropriate rainfall depths for the site as determined using the methods in Section 2.2 and Section 2.3 – these values should include any appropriate depth-area-reduction. Settings should be as presented in Figure 4-14, with the exception of the Partial-Duration Depth values, which will be site and storm duration specific.

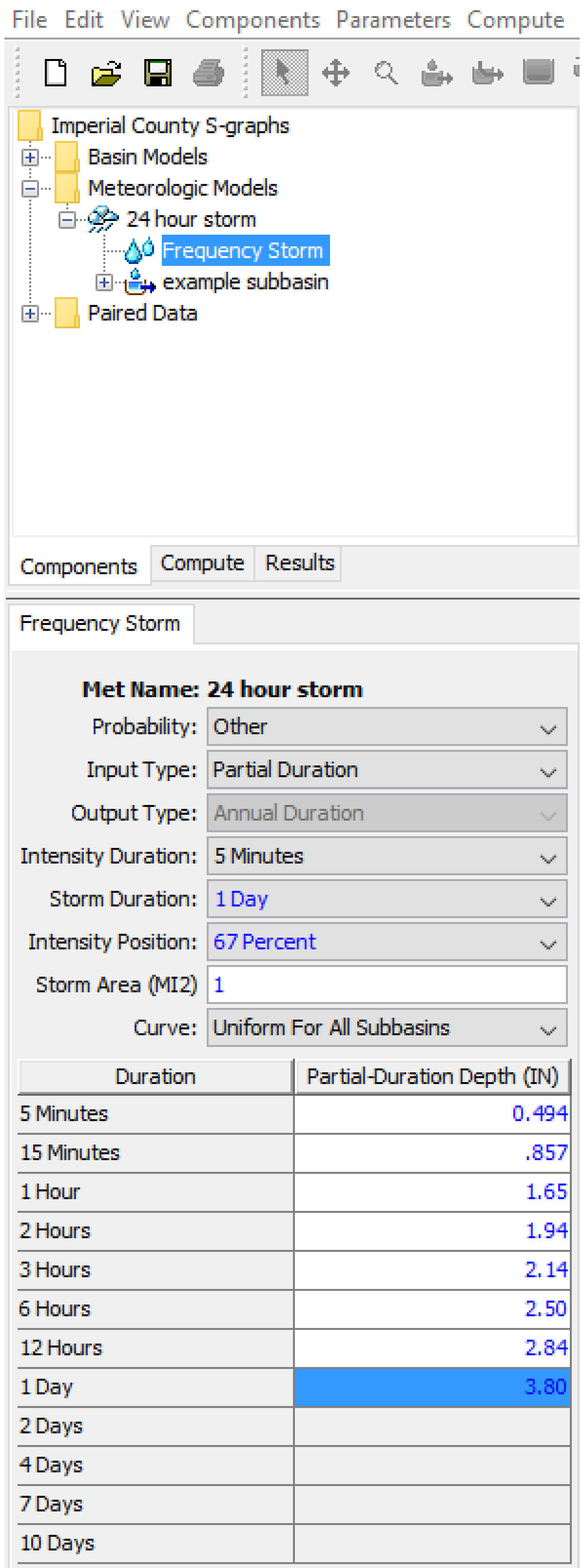


Figure 4-14. HMS 1/3, 2/3 balanced storm setup

Step 8. Control specification creation. Use the "Components" → "Control Specification Manager" to create a simulation time window for the hydrograph creation as presented in Figure 4-15.

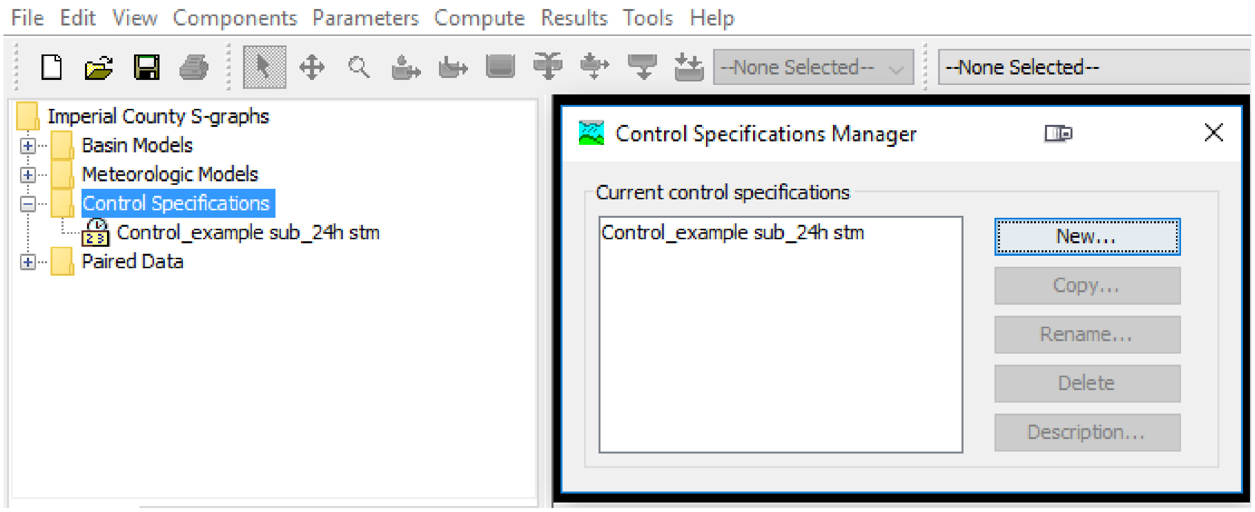


Figure 4-15. HMS Control Specifications creation

Step 9. Control Specifications settings. The start and end dates and times should be selected to provide enough time to capture the entire hydrograph. The "Time Interval" setting of the Control Specifications should be set no greater than the "Intensity-Duration" in Step 7 (5 minutes in this example.). In the example shown, a time interval of 5 minutes is selected. When peak discharge is of primary importance, a short time interval should be utilized. Settings should be as presented in Figure 4-16. .

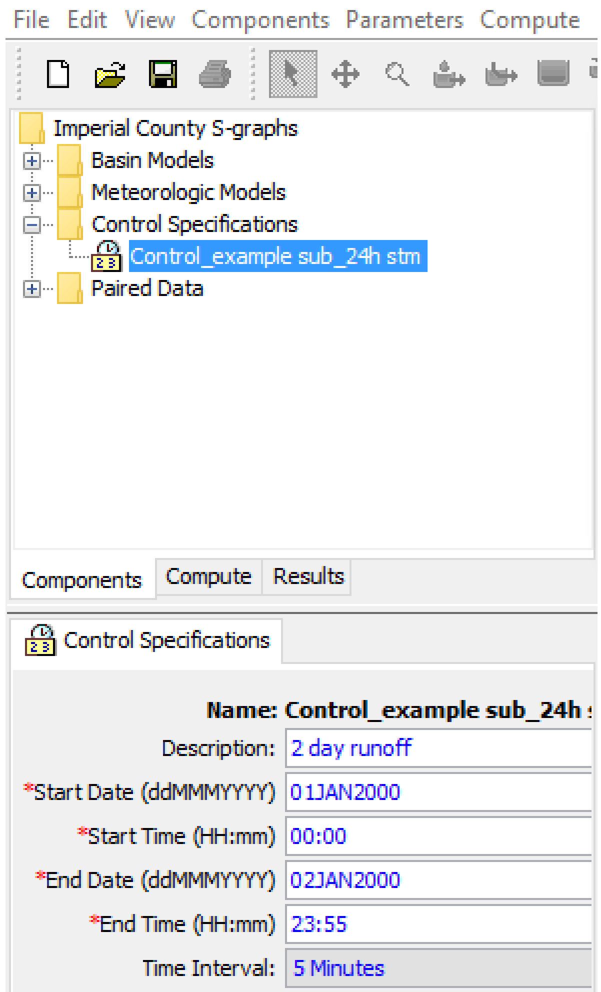


Figure 4-16. Control Specifications settings

Step 10. Create a Simulation Run. Use the “Compute” → “Create Compute” → “Simulation Run …” to prepare a model run. Follow the prompts to name the model run, select the basin model created in Step 3, Meteorologic Model created in Step 6 and the Control Specifications created in Step 9.

Step 11. Calculate the hydrograph. Select the “Compute” tab, select Simulation Runs and right click the simulation run created in Step 10. Click compute as presented in Figure 4-17.

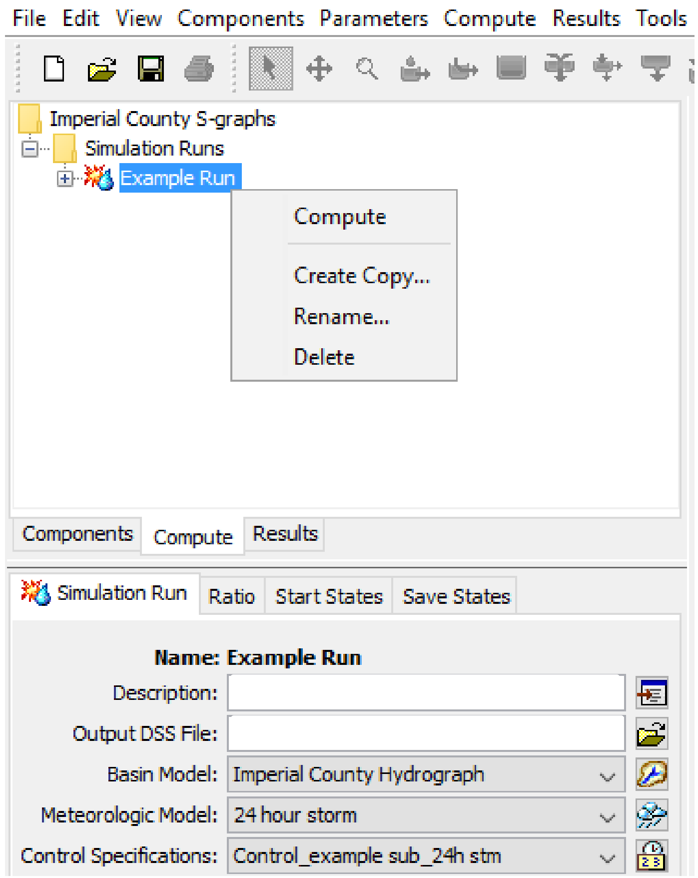


Figure 4-17. HMS hydrograph calculation

Step 12. View the results. The resulting hydrograph may be viewed by selecting the "Results" tab, clicking "Simulation Runs", clicking the simulation run created in Step 10, clicking the subbasin created in Step 4 and selecting "Graph" as presented in Figure 4-18.

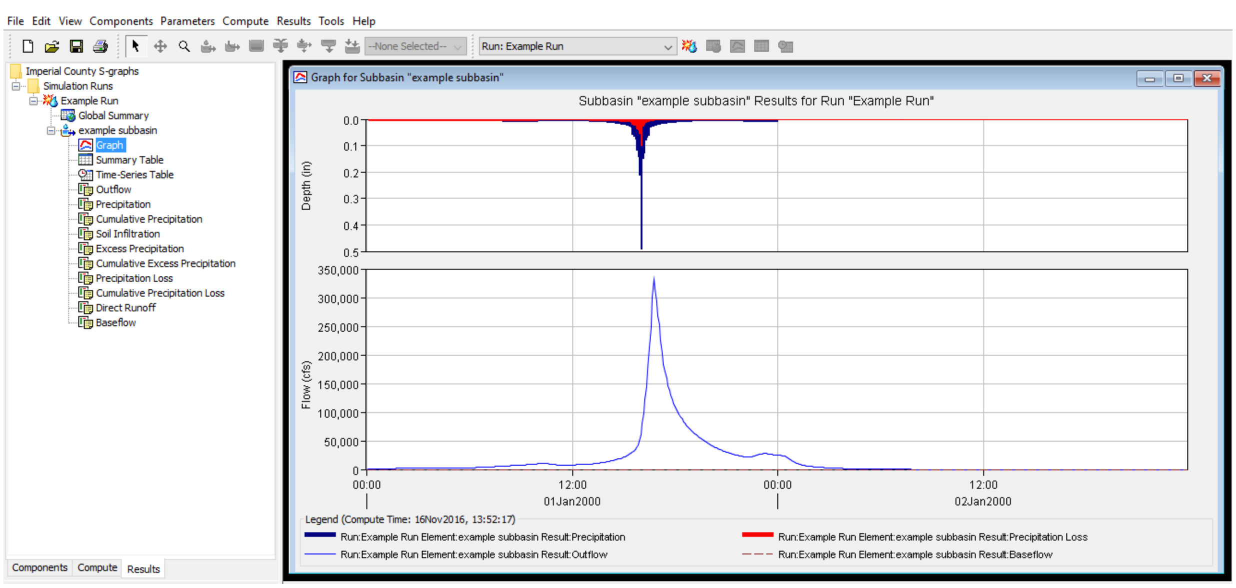


Figure 4-18. Viewing hydrograph results

By visual inspection, it may be concluded the time window chosen for simulation is sufficient to capture the rising and falling hydrograph limbs. (In fact, the time could be shortened by returning to Step 9, changing the end time and re-running the model.) Results such as peak discharge, time of peak discharge, runoff volume, etc. are available by clicking "Summary Table" below the "Graph" icon previously selected. An example Summary Results window is presented as Figure 4-19. Detailed output for each time step is also available by selecting "Time-Series Table" below "Summary Table" in the hierarchical list. An example of more detailed output is presented in Figure 4-20.

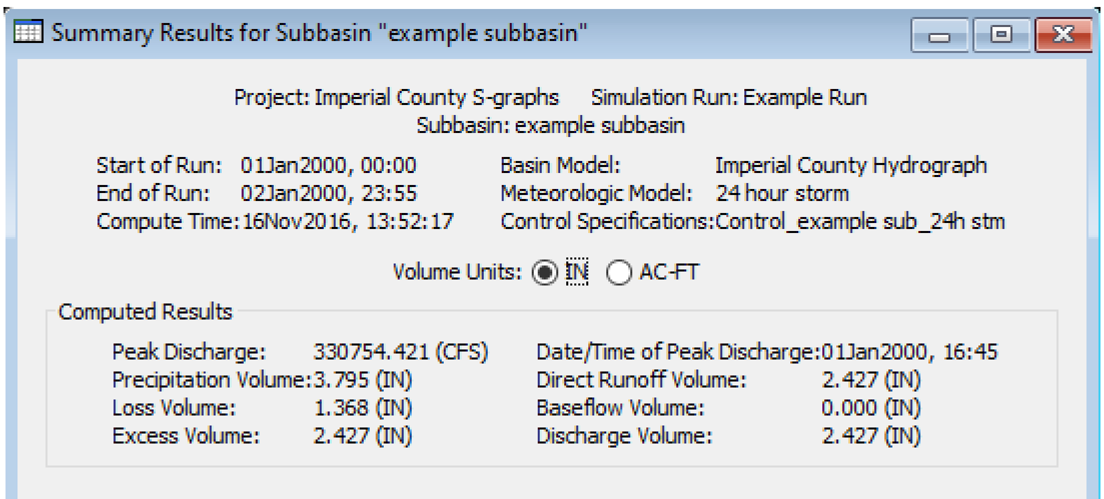


Figure 4-19. Hydrograph summary results

Time-Series Results for Subbasin "example subbasin"

Project: Imperial County S-graphs Simulation Run: Example Run
Subbasin: example subbasin

Start of Run: 01Jan2000, 00:00 Basin Model: Imperial County Hydrograph
End of Run: 02Jan2000, 23:55 Meteorologic Model: 24 hour storm
Compute Time: 16Nov2016, 13:52:17 Control Specifications: Control_example sub_24h stm

Date	Time	Precip (IN)	Loss (IN)	Excess (IN)	Direct Flow (CFS)	Baseflow (CFS)	Total Flow (CFS)
01Jan2000	00:00				0.000	0.000	0.000
01Jan2000	00:05	0.006	0.005	0.001	15.386	0.000	15.386
01Jan2000	00:10	0.006	0.005	0.001	36.984	0.000	36.984
01Jan2000	00:15	0.006	0.005	0.001	68.883	0.000	68.883
01Jan2000	00:20	0.006	0.005	0.001	108.993	0.000	108.993
01Jan2000	00:25	0.006	0.005	0.001	155.429	0.000	155.429
01Jan2000	00:30	0.006	0.005	0.001	231.184	0.000	231.184
01Jan2000	00:35	0.006	0.005	0.001	325.905	0.000	325.905
01Jan2000	00:40	0.006	0.005	0.001	480.074	0.000	480.074
01Jan2000	00:45	0.006	0.005	0.001	679.697	0.000	679.697
01Jan2000	00:50	0.006	0.005	0.001	842.895	0.000	842.895
01Jan2000	00:55	0.006	0.005	0.001	975.639	0.000	975.639
01Jan2000	01:00	0.006	0.005	0.001	1082.621	0.000	1082.621
01Jan2000	01:05	0.006	0.005	0.001	1187.521	0.000	1187.521
01Jan2000	01:10	0.006	0.005	0.001	1269.334	0.000	1269.334
01Jan2000	01:15	0.006	0.005	0.001	1339.196	0.000	1339.196
01Jan2000	01:20	0.006	0.005	0.001	1397.524	0.000	1397.524
01Jan2000	01:25	0.006	0.005	0.001	1452.061	0.000	1452.061
01Jan2000	01:30	0.006	0.005	0.001	1503.888	0.000	1503.888
01Jan2000	01:35	0.006	0.005	0.001	1546.057	0.000	1546.057
01Jan2000	01:40	0.006	0.005	0.001	1587.484	0.000	1587.484
01Jan2000	01:45	0.006	0.005	0.001	1628.105	0.000	1628.105
01Jan2000	01:50	0.006	0.005	0.001	1661.687	0.000	1661.687
01Jan2000	01:55	0.006	0.005	0.001	1694.440	0.000	1694.440
01Jan2000	02:00	0.006	0.005	0.001	1723.902	0.000	1723.902
01Jan2000	02:05	0.006	0.005	0.001	1755.620	0.000	1755.620
01Jan2000	02:10	0.006	0.005	0.001	1780.838	0.000	1780.838
01Jan2000	02:15	0.006	0.005	0.001	1807.912	0.000	1807.912
01Jan2000	02:20	0.006	0.005	0.001	1833.191	0.000	1833.191
01Jan2000	02:25	0.006	0.005	0.001	1857.018	0.000	1857.018
01Jan2000	02:30	0.006	0.005	0.001	1879.635	0.000	1879.635
01Jan2000	02:35	0.006	0.005	0.001	1900.934	0.000	1900.934

Figure 4-20. Hydrograph detailed output

Of course, the simulation results from the example are for the 1 square mile watershed used to apply the proper depth-area-reduction. Final results are obtained by multiplying simulation results by the actual square mile area of the watershed. The abscissa and ordinate values of the hydrograph are available in the detailed output. The procedure described for determining a runoff hydrograph is applicable to a single basin. Analysis of more complicated watersheds requiring subbasins should follow a similar overall approach and may require the use of junctions, routing reaches, reservoirs, etc. Refer to the HEC-HMS User's Manual for further information regarding the use of multiple subbasins.

4.5 Transition from Rational Method to NRCS Hydrologic Method

As discussed in Section 3.1, the engineer should only use the RM or MRM for drainage areas up to approximately 1 square mile. The NRCS hydrologic method should be used for study areas approximately 1 square mile and greater in size. For study areas greater than approximately 1 square mile, the NRCS hydrologic method may be used for the entire study area, or the RM or MRM may be used for approximately 1 square mile of the study area with results then transitioned to the NRCS hydrologic method solutions using the procedure described below:

1) Stop RM calculations at approximately 1 square mile;

2) Freeze RM peak discharge, Q_p, at approximately 1 square mile;

3) Begin NRCS hydrograph calculations at the next point of interest. Estimate the travel time, T_t, from the MRM calculations along the reach to the point of interest, and increase the T_c from the MRM calculations by T_t. Determine T_p based on T_c using McCuen (1982):

$$T_P = 0.67\ T_C \qquad (4\text{-}6)$$

   Perform NRCS calculations as described in Section 4.4 and the total watershed area to the point of interest.

If $Q_{MRM} > Q_{NRCS}$ then use Q_{MRM}.

If $Q_{MRM} < Q_{NRCS}$ then use Q_{NRCS}.

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Appendix

Imperial County S-graph Coordinates

Table A-1. Foothill S-graph values

% Lag	% Peak q	% Lag	% Peak q	% Lag	% Peak q	% Lag	% Peak q
0	0.00	40	7.74	80	25.92	120	62.26
1	0.03	41	8.00	81	26.63	121	62.65
2	0.16	42	8.32	82	27.41	122	63.03
3	0.22	43	8.58	83	28.25	123	63.42
4	0.42	44	8.90	84	29.15	124	63.81
5	0.61	45	9.16	85	30.12	125	64.16
6	0.80	46	9.48	86	31.09	126	64.45
7	0.86	47	9.87	87	32.12	127	64.84
8	0.93	48	10.13	88	33.09	128	65.35
9	1.06	49	10.45	89	34.57	129	65.74
10	1.31	50	10.83	90	35.99	130	66.06
11	1.38	51	10.96	91	37.28	131	66.38
12	1.57	52	11.35	92	39.48	132	66.77
13	1.70	53	11.73	93	42.13	133	67.09
14	1.83	54	12.19	94	44.39	134	67.54
15	1.95	55	12.51	95	45.87	135	67.87
16	2.13	56	12.83	96	46.97	136	68.12
17	2.34	57	13.22	97	47.92	137	68.38
18	2.53	58	13.67	98	48.91	138	68.57
19	2.60	59	14.18	99	49.64	139	68.89
20	2.89	60	14.51	100	50.00	140	69.22
21	3.11	61	15.02	101	50.69	141	69.47
22	3.24	62	15.54	102	51.74	142	69.86
23	3.50	63	15.92	103	52.87	143	70.05
24	3.82	64	16.38	104	53.88	144	70.31
25	4.01	65	16.96	105	54.71	145	70.76
26	4.08	66	17.47	106	55.36	146	71.08
27	4.20	67	17.99	107	55.94	147	71.28
28	4.40	68	18.44	108	56.52	148	71.53
29	4.65	69	19.08	109	57.17	149	71.79
30	4.91	70	19.73	110	57.75	150	72.05
31	5.23	71	20.18	111	58.26	151	72.24
32	5.56	72	20.70	112	58.71	152	72.56
33	5.81	73	21.28	113	59.04	153	72.89
34	6.13	74	22.12	114	59.42	154	73.01
35	6.39	75	22.57	115	59.98	155	73.30
36	6.65	76	22.95	116	60.33	156	73.59
37	6.97	77	23.60	117	60.84	157	73.85
38	7.10	78	24.24	118	61.29	158	74.04
39	7.42	79	25.08	119	61.87	159	74.30

% Lag	% Peak q		% Lag	% Peak q		% Lag	% Peak q		% Lag	% Peak q
160	74.69		201	82.59		242	88.01		283	91.99
161	74.94		202	82.65		243	88.12		284	92.07
162	75.14		203	82.85		244	88.23		285	92.15
163	75.33		204	82.91		245	88.34		286	92.23
164	75.52		205	83.08		246	88.44		287	92.31
165	75.85		206	83.30		247	88.55		288	92.39
166	75.97		207	83.36		248	88.66		289	92.47
167	76.17		208	83.55		249	88.76		290	92.55
168	76.42		209	83.68		250	88.87		291	92.63
169	76.62		210	83.81		251	88.97		292	92.71
170	76.87		211	84.00		252	89.08		293	92.79
171	77.13		212	84.07		253	89.18		294	92.87
172	77.32		213	84.19		254	89.28		295	92.94
173	77.52		214	84.39		255	89.39		296	93.02
174	77.71		215	84.58		256	89.49		297	93.09
175	77.94		216	84.71		257	89.59		298	93.17
176	78.03		217	84.84		258	89.69		299	93.24
177	78.22		218	85.03		259	89.79		300	93.31
178	78.42		219	85.09		260	89.89		301	93.38
179	78.67		220	85.22		261	89.99		302	93.46
180	78.93		221	85.35		262	90.08		303	93.53
181	79.06		222	85.48		263	90.18		304	93.60
182	79.32		223	85.60		264	90.28		305	93.67
183	79.51		224	85.73		265	90.37		306	93.74
184	79.70		225	85.79		266	90.47		307	93.80
185	79.92		226	85.99		267	90.56		308	93.87
186	79.96		227	86.18		268	90.66		309	93.94
187	80.13		228	86.24		269	90.75		310	94.00
188	80.41		229	86.44		270	90.84		311	94.07
189	80.54		230	86.63		271	90.93		312	94.14
190	80.79		231	86.69		272	91.03		313	94.20
191	80.86		232	86.89		273	91.12		314	94.26
192	81.05		233	86.95		274	91.21		315	94.33
193	81.24		234	87.11		275	91.30		316	94.39
194	81.37		235	87.23		276	91.38		317	94.45
195	81.50		236	87.34		277	91.47		318	94.51
196	81.76		237	87.45		278	91.56		319	94.57
197	82.01		238	87.57		279	91.65		320	94.64
198	82.08		239	87.68		280	91.73		321	94.69
199	82.14		240	87.79		281	91.82		322	94.75
200	82.27		241	87.90		282	91.90		323	94.81

% Lag	% Peak q	% Lag	% Peak q	% Lag	% Peak q	% Lag	% Peak q
324	94.87	365	96.76	406	97.88	447	98.49
325	94.93	366	96.80	407	97.90	448	98.50
326	94.98	367	96.83	408	97.92	449	98.51
327	95.04	368	96.87	409	97.94	450	98.52
328	95.10	369	96.90	410	97.96	451	98.53
329	95.15	370	96.94	411	97.98	452	98.54
330	95.20	371	96.97	412	98.00	453	98.55
331	95.26	372	97.00	413	98.01	454	98.56
332	95.31	373	97.03	414	98.03	455	98.57
333	95.36	374	97.07	415	98.05	456	98.58
334	95.42	375	97.10	416	98.07	457	98.59
335	95.47	376	97.13	417	98.08	458	98.60
336	95.52	377	97.16	418	98.10	460	98.61
337	95.57	378	97.19	419	98.12	461	98.62
338	95.62	379	97.22	420	98.13	462	98.63
339	95.67	380	97.25	421	98.15	463	98.64
340	95.72	381	97.28	422	98.16	464	98.65
341	95.76	382	97.31	423	98.18	465	98.66
342	95.81	383	97.33	424	98.20	467	98.67
343	95.86	384	97.36	425	98.21	468	98.68
344	95.91	385	97.39	426	98.23	469	98.69
345	95.95	386	97.42	427	98.24	470	98.70
346	96.00	387	97.44	428	98.25	472	98.71
347	96.04	388	97.47	429	98.27	473	98.72
348	96.09	389	97.50	430	98.28	474	98.73
349	96.13	390	97.52	431	98.30	476	98.74
350	96.17	391	97.55	432	98.31	477	98.75
351	96.22	392	97.57	433	98.32	478	98.76
352	96.26	393	97.60	434	98.34	480	98.77
353	96.30	394	97.62	435	98.35	481	98.78
354	96.34	395	97.64	436	98.36	482	98.79
355	96.38	396	97.67	437	98.37	484	98.80
356	96.42	397	97.69	438	98.39	485	98.81
357	96.46	398	97.71	439	98.40	487	98.82
358	96.50	399	97.73	440	98.41	488	98.83
359	96.54	400	97.76	441	98.42	490	98.84
360	96.58	401	97.78	442	98.43	491	98.85
361	96.62	402	97.80	443	98.44	493	98.86
362	96.65	403	97.82	444	98.46	494	98.87
363	96.69	404	97.84	445	98.47	496	98.88
364	96.73	405	97.86	446	98.48	498	98.89

% Lag	% Peak q		% Lag	% Peak q		% Lag	% Peak q		% Lag	% Peak q
499	98.90		544	99.18		595	99.54		624	99.74
501	98.91		546	99.19		597	99.55		625	99.75
502	98.92		547	99.20		598	99.56		627	99.76
504	98.93		549	99.21		599	99.57		628	99.77
506	98.94		550	99.22		601	99.58		630	99.78
507	98.95		552	99.23		602	99.59		632	99.79
509	98.96		553	99.24		603	99.60		633	99.80
511	98.97		555	99.25		605	99.61		635	99.81
512	98.98		556	99.26		606	99.62		637	99.82
514	98.99		558	99.27		608	99.63		638	99.83
515	99.00		559	99.28		609	99.64		640	99.84
517	99.01		561	99.29		610	99.65		642	99.85
519	99.02		562	99.30		612	99.66		644	99.86
520	99.03		575	99.39		613	99.67		646	99.87
522	99.04		576	99.40		615	99.68		648	99.88
524	99.05		577	99.41		616	99.69		651	99.89
525	99.06		579	99.42		618	99.70		653	99.90
527	99.07		580	99.43		619	99.71		655	99.91
528	99.08		582	99.44		563	99.31		658	99.92
530	99.09		583	99.45		565	99.32		661	99.93
532	99.10		584	99.46		566	99.33		667	99.94
533	99.11		586	99.47		568	99.34		674	99.95
535	99.12		587	99.48		569	99.35		683	99.96
536	99.13		588	99.49		570	99.36		687	99.97
538	99.14		590	99.50		572	99.37		692	99.98
540	99.15		591	99.51		573	99.38		697	99.99
541	99.16		592	99.52		621	99.72		700	100.00
543	99.17		594	99.53		622	99.73			

Table A-2. Desert S-graph values

% Lag	% Peak q	% Lag	% Peak q	% Lag	% Peak q	% Lag	% Peak q
0	0.00	40	6.66	80	36.20	120	59.83
1	0.06	41	7.02	81	37.17	121	60.25
2	0.19	42	7.31	82	37.81	122	60.63
3	0.22	43	7.69	83	38.72	123	60.95
4	0.25	44	8.01	84	39.56	124	61.28
5	0.32	45	8.34	85	40.33	125	61.60
6	0.44	46	8.79	86	41.17	126	61.92
7	0.57	47	9.24	87	41.94	127	62.18
8	0.70	48	9.75	88	42.59	128	62.50
9	0.76	49	10.14	89	43.17	129	62.88
10	0.96	50	10.72	90	44.01	130	63.21
11	1.02	51	11.23	91	44.78	131	63.46
12	1.08	52	11.49	92	45.30	132	63.78
13	1.21	53	11.88	93	46.07	133	64.11
14	1.34	54	12.33	94	46.78	134	64.36
15	1.46	55	12.84	95	47.62	135	64.81
16	1.59	56	13.60	96	48.13	136	65.07
17	1.79	57	14.26	97	48.58	137	65.33
18	1.98	58	14.91	98	49.22	138	65.58
19	2.11	59	15.49	99	49.64	139	65.91
20	2.23	60	16.18	100	50.00	140	66.23
21	2.49	61	16.97	101	50.59	141	66.42
22	2.68	62	17.75	102	51.31	142	66.68
23	2.75	63	18.52	103	52.13	143	66.93
24	2.94	64	19.29	104	52.65	144	67.19
25	3.20	65	20.20	105	53.23	145	67.45
26	3.39	66	21.40	106	53.87	146	67.71
27	3.52	67	22.65	107	54.26	147	67.96
28	3.77	68	23.68	108	54.64	148	68.22
29	3.90	69	24.65	109	55.09	149	68.48
30	4.07	70	26.20	110	55.67	150	68.86
31	4.41	71	27.36	111	56.19	151	69.16
32	4.67	72	28.20	112	56.58	152	69.31
33	4.99	73	29.36	113	56.96	153	69.51
34	5.19	74	30.46	114	57.35	154	69.76
35	5.51	75	31.49	115	57.80	155	69.89
36	5.70	76	32.33	116	58.25	156	70.15
37	5.96	77	33.10	117	58.57	157	70.40
38	6.21	78	34.07	118	58.89	158	70.60
39	6.41	79	35.04	119	59.28	159	70.85

% Lag	% Peak q	% Lag	% Peak q	% Lag	% Peak q	% Lag	% Peak q
160	71.11	201	78.35	242	83.51	283	87.25
161	71.30	202	78.48	243	83.62	284	87.33
162	71.50	203	78.67	244	83.72	285	87.40
163	71.69	204	78.87	245	83.83	286	87.48
164	71.88	205	78.93	246	83.93	287	87.56
165	72.14	206	79.12	247	84.03	288	87.63
166	72.33	207	79.25	248	84.14	289	87.71
167	72.52	208	79.44	249	84.24	290	87.78
168	72.71	209	79.51	250	84.34	291	87.85
169	72.89	210	79.70	251	84.44	292	87.93
170	73.16	211	79.76	252	84.54	293	88.00
171	73.29	212	79.89	253	84.63	294	88.07
172	73.48	213	79.97	254	84.73	295	88.15
173	73.68	214	80.11	255	84.83	296	88.22
174	73.93	215	80.24	256	84.92	297	88.29
175	74.19	216	80.38	257	85.02	298	88.36
176	74.25	217	80.51	258	85.11	299	88.43
177	74.45	218	80.65	259	85.21	300	88.50
178	74.64	219	80.78	260	85.30	301	88.57
179	74.83	220	80.91	261	85.39	302	88.64
180	75.02	221	81.04	262	85.48	303	88.71
181	75.09	222	81.17	263	85.57	304	88.77
182	75.34	223	81.30	264	85.66	305	88.84
183	75.54	224	81.42	265	85.75	306	88.91
184	75.71	225	81.55	266	85.84	307	88.97
185	75.92	226	81.67	267	85.93	308	89.04
186	76.05	227	81.79	268	86.02	309	89.11
187	76.11	228	81.91	269	86.10	310	89.17
188	76.37	229	82.04	270	86.19	311	89.24
189	76.56	230	82.15	271	86.27	312	89.30
190	76.63	231	82.27	272	86.36	313	89.37
191	76.82	232	82.39	273	86.44	314	89.43
192	77.01	233	82.51	274	86.52	315	89.49
193	77.14	234	82.62	275	86.61	316	89.56
194	77.33	235	82.74	276	86.69	317	89.62
195	77.52	236	82.85	277	86.77	318	89.68
196	77.71	237	82.96	278	86.85	319	89.74
197	77.78	238	83.07	279	86.93	320	89.81
198	77.97	239	83.18	280	87.01	321	89.87
199	78.03	240	83.29	281	87.09	322	89.93
200	78.16	241	83.40	282	87.17	323	89.99

% Lag	% Peak q	% Lag	% Peak q	% Lag	% Peak q	% Lag	% Peak q
324	90.05	365	92.25	406	94.02	447	95.44
325	90.11	366	92.30	407	94.06	448	95.47
326	90.17	367	92.35	408	94.10	449	95.50
327	90.23	368	92.39	409	94.14	450	95.53
328	90.29	369	92.44	410	94.18	451	95.56
329	90.34	370	92.49	411	94.22	452	95.59
330	90.40	371	92.54	412	94.25	453	95.62
331	90.46	372	92.58	413	94.29	454	95.65
332	90.52	373	92.63	414	94.33	455	95.67
333	90.58	374	92.67	415	94.36	456	95.70
334	90.63	375	92.72	416	94.40	457	95.73
335	90.69	376	92.77	417	94.44	458	95.76
336	90.74	377	92.81	418	94.47	459	95.79
337	90.80	378	92.86	419	94.51	460	95.82
338	90.86	379	92.90	420	94.55	461	95.84
339	90.91	380	92.95	421	94.58	462	95.87
340	90.97	381	92.99	422	94.62	463	95.90
341	91.02	382	93.03	423	94.65	464	95.93
342	91.08	383	93.08	424	94.69	465	95.95
343	91.13	384	93.12	425	94.72	466	95.98
344	91.18	385	93.16	426	94.76	467	96.01
345	91.24	386	93.21	427	94.79	468	96.03
346	91.29	387	93.25	428	94.83	469	96.06
347	91.34	388	93.29	429	94.86	470	96.09
348	91.40	389	93.34	430	94.89	471	96.11
349	91.45	390	93.38	431	94.93	472	96.14
350	91.50	391	93.42	432	94.96	473	96.16
351	91.55	392	93.46	433	94.99	474	96.19
352	91.60	393	93.50	434	95.03	475	96.22
353	91.66	394	93.55	435	95.06	476	96.24
354	91.71	395	93.59	436	95.09	477	96.27
355	91.76	396	93.63	437	95.12	478	96.29
356	91.81	397	93.67	438	95.16	479	96.31
357	91.86	398	93.71	439	95.19	480	96.34
358	91.91	399	93.75	440	95.22	481	96.36
359	91.96	400	93.79	441	95.25	482	96.39
360	92.01	401	93.83	442	95.28	483	96.41
361	92.06	402	93.87	443	95.31	484	96.44
362	92.11	403	93.91	444	95.35	485	96.46
363	92.15	404	93.95	445	95.38	486	96.48
364	92.20	405	93.99	446	95.41	487	96.51

% Lag	% Peak q	% Lag	% Peak q	% Lag	% Peak q	% Lag	% Peak q
488	96.53	529	97.34	570	97.94	611	98.43
489	96.55	530	97.36	571	97.96	612	98.44
490	96.57	531	97.37	572	97.97	613	98.45
491	96.60	532	97.39	573	97.98	614	98.46
492	96.62	533	97.41	574	97.99	615	98.47
493	96.64	534	97.42	575	98.01	616	98.48
494	96.66	535	97.44	576	98.02	617	98.49
495	96.69	536	97.46	577	98.03	618	98.50
496	96.71	537	97.47	578	98.04	619	98.52
497	96.73	538	97.49	579	98.06	620	98.53
498	96.75	539	97.50	580	98.07	621	98.54
499	96.77	540	97.52	581	98.08	622	98.55
500	96.79	541	97.53	582	98.09	623	98.56
501	96.81	542	97.55	583	98.10	624	98.57
502	96.83	543	97.57	584	98.12	625	98.58
503	96.85	544	97.58	585	98.13	626	98.59
504	96.88	545	97.60	586	98.14	627	98.60
505	96.90	546	97.61	587	98.15	628	98.61
506	96.92	547	97.63	588	98.16	629	98.63
507	96.94	548	97.64	589	98.18	630	98.64
508	96.96	549	97.66	590	98.19	631	98.65
509	96.98	550	97.67	591	98.20	632	98.66
510	96.99	551	97.68	592	98.21	633	98.67
511	97.01	552	97.70	593	98.22	634	98.68
512	97.03	553	97.71	594	98.23	635	98.69
513	97.05	554	97.73	595	98.25	636	98.70
514	97.07	555	97.74	596	98.26	637	98.71
515	97.09	556	97.76	597	98.27	638	98.72
516	97.11	557	97.77	598	98.28	639	98.74
517	97.13	558	97.78	599	98.29	640	98.75
518	97.15	559	97.80	600	98.30	641	98.76
519	97.16	560	97.81	601	98.31	642	98.77
520	97.18	561	97.82	602	98.33	643	98.78
521	97.20	562	97.84	603	98.34	644	98.79
522	97.22	563	97.85	604	98.35	645	98.80
523	97.24	564	97.86	605	98.36	646	98.81
524	97.25	565	97.88	606	98.37	647	98.82
525	97.27	566	97.89	607	98.38	648	98.83
526	97.29	567	97.90	608	98.39	649	98.85
527	97.31	568	97.92	609	98.40	650	98.86
528	97.32	569	97.93	610	98.42	651	98.87

% Lag	% Peak q	% Lag	% Peak q	% Lag	% Peak q	% Lag	% Peak q
652	98.88	679	99.18	706	99.48	733	99.76
653	98.89	680	99.19	707	99.49	734	99.77
654	98.90	681	99.20	708	99.50	735	99.78
655	98.91	682	99.22	709	99.51	736	99.79
656	98.92	683	99.23	710	99.52	737	99.80
657	98.94	684	99.24	711	99.53	738	99.81
658	98.95	685	99.25	712	99.54	739	99.82
659	98.96	686	99.26	713	99.55	740	99.83
660	98.97	687	99.27	714	99.56	741	99.84
661	98.98	688	99.28	715	99.57	742	99.85
662	98.99	689	99.29	716	99.59	743	99.86
663	99.00	690	99.30	717	99.60	744	99.87
664	99.01	691	99.32	718	99.61	745	99.88
665	99.02	692	99.33	719	99.62	746	99.89
666	99.04	693	99.34	720	99.63	747	99.90
667	99.05	694	99.35	721	99.64	748	99.91
668	99.06	695	99.36	722	99.65	749	99.92
669	99.07	696	99.37	723	99.66	750	99.93
670	99.08	697	99.38	724	99.67	751	99.94
671	99.09	698	99.39	725	99.68	755	99.95
672	99.10	699	99.40	726	99.69	760	99.96
673	99.11	700	99.41	727	99.70	765	99.97
674	99.13	701	99.42	728	99.71	770	99.98
675	99.14	702	99.43	729	99.72	775	99.99
676	99.15	703	99.45	730	99.73	780	100.00
677	99.16	704	99.46	731	99.74		
678	99.17	705	99.47	732	99.75		