CHAPTER 6
RUNOFF

Contents

Contents, Continued

Tables

Figures

1.0 OVERVIEW

Accurately quantifying stormwater runoff is critical for proper design of drainage infrastructure. Estimates of peak runoff flow rates, runoff volumes, and the time distribution of flows provide the basis for all planning, design, and construction of drainage facilities that manage and mitigate changes in hydrology as watersheds develop. Hydrology is foundational to hydraulics. Errors in hydrologic calculations or unrealistic results affect hydraulic design and can lead to infrastructure that is either undersized or oversized.

Photograph 6-1. Devastating flooding from Gregory Canyon Creek in Boulder in September 2013 emphasizes the importance of accurate flood flow projections and appropriately sized infrastructure.

While accuracy is the goal of hydrologic computations, it is important to understand that the results of the runoff analysis are engineering approximations with associated uncertainty. This chapter intends to provide reasonably dependable and consistent methods for approximating the characteristics of urban runoff for areas of Colorado and the United States that have meteorology and hydrology similar to that within the Mile High Flood District (MHFD).

Runoff computations must address a range of hydrologic conditions from small events of concern for water quality to large flood events. Runoff computations are used to define:

1. Water Quality Event (WQE) – The WQE is a design storm with a rainfall depth equal to the 80th percentile runoff-producing storm event depth. This event is based on a 1-hour point precipitation depth of 0.60 inches for the MHFD region. The WQE refers to both the volume and peak flow rate associated with the water quality design storm.

2. Water Quality Capture Volume (WQCV) – The WQCV is the runoff volume produced by the WQE. The WQCV is used for sizing stormwater control measures (SCMs) and is a component of Full Spectrum Detention (FSD). Guidance on calculating the WQCV is provided in *Calculating the WQCV and Volume Reduction* chapter of Volume 3.

3. Water Quality Peak Flow (WQPF) – The WQPF is the peak flow rate associated with the WQE. The WQPF is calculated using either the Rational Method or Colorado Urban Hydrograph Procedure as described below. The WQPF can be used for designing SCMs that are sized using a peak flow rate rather than the WQCV.

4. Excess Urban Runoff Volume (EURV) – The EURV is the difference between the developed condition runoff volume and the pre-development runoff volume across a wide range of storm events. Within the MHFD region, the EURV is relatively consistent at any given level of imperviousness for the range of storms that produce runoff. The EURV includes the WQCV. Guidance on calculating the EURV is provided in the *Storage* chapter.

5. Minor Storm Event – The minor storm event typically corresponds to a 2- to 10-year event (50% to 10% annual exceedance probabilities [AEPs]) within the MHFD region. Local governments typically establish the minor storm event return period for their communities.

6. Major Storm Event – In the MHFD region, the major storm event is typically defined as a 100-year storm event (1% AEP); although, some circumstances (critical facilities or other critical infrastructure), may warrant a less frequent major event such as the 500-year event (0.2% AEP).
7. 100-year Detention Volume – The 100-year detention volume is the storage volume needed to attenuate developed peak flow rates to allowable release rates. The 100-year detention volume includes the EURV and WQCV. Guidance on calculating the 100-year detention volume is provided in the *Storage* chapter.
8. 1% Plus Discharges – 1% Plus discharges are flows corresponding to the upper 84% confidence limit of the 100-year storm event (1% AEP). As defined by the Federal Emergency Management Agency (FEMA), the 1% Plus discharges represent one standard deviation above the 1% AEP peak flood discharges. The 1% Plus discharges can be calculated using U.S. Geological Survey (USGS) Bulletin 17C methods to analyze stream gage data or determined statistically for areas where stream flows are modeled rather than directly gaged.

All computations require imperviousness as an input for calculations. Other hydrologic parameters, such as low flows, bankfull flows, and effective discharge, are rooted in the concepts discussed in this chapter.

2.0 HYDROLOGIC METHODS AND APPLICABILITY

This chapter presents various hydrologic methods ranging from simple empirical equations to complex models. The chapter provides detailed guidance on the Rational Method and the Colorado Urban Hydrograph Procedure (CUHP) in combination with U.S. Environmental Protection Agency Storm Water Management Model (EPA SWMM) routing. It also includes guidance and criteria for evaluating the WQE, WQPF, and 1% Plus discharges.

Four methods of hydrologic analysis are commonly used for design of storm drainage infrastructure:

1. The Rational Method – Originally introduced in 1889, most engineering offices in the U.S. continue to use this method. Although this method has frequently come under academic criticism for its simplicity, no other practical drainage design method has evolved to such a level of general acceptance by the practicing engineer.
2. CUHP – CUHP is a regionally calibrated model for generating hydrographs from watersheds. Modelers often use CUHP in conjunction with EPA SWMM, using EPA SWMM to combine and route the hydrographs generated using CUHP.
3. Use of published runoff information – Hydrologic studies have been conducted for most of the major drainage systems within MHFD, and published hydrology data are available for most of these watersheds and streams from MHFD Major Drainageway Plans (MDPs), Outfall Systems Plans (OSPs), and Flood Hazard Area Delineations (FHADs) or other credible sources such as Flood Insurance Studies (FISs) and Letters of Map Revision (LOMRs).
4. Statistical analysis of stream gage data (such as USGS Bulletin 17C analysis) – This approach requires a long-term record of quality flow measurement data conforming to the assumptions of the statistical analysis methods.

The Rational Method is applicable to urban catchments that are (1) not complex, and (2) generally 90 acres or smaller. The Rational Method only calculates peak flow rates and not runoff hydrographs. Calculate peak flows using the Rational Method by hand or use the MHFD-Rational Excel workbook available at www.mhfd.org.

Since 1969, CUHP has been used extensively in this region. It has been calibrated by MHFD using regional data collected from various watersheds to develop empirical relationships between the input hyetograph and observed output flows. Many major drainageways and storm drainage systems within MHFD are designed based on hydrology calculated using CUHP and hydraulics evaluated using EPA SWMM or MHFD's UD-SWMM, an earlier adaptation of SWMM software. Use CUHP and SWMM for larger catchments and whenever a runoff hydrograph is needed for analysis.

TABLE 6-1. APPLICABILITY OF HYDROLOGIC METHODS

CATCHMENT SIZE (ACRES)	IS THE RATIONAL METHOD APPLICABLE?	IS CUHP APPLICABLE?
0 to 90	Yes	Yes
90 to 160	No	Yes
160 to 3,000	No	Yes[1]
Greater than 3,000)	No	Yes (subdividing into smaller catchments required)[1]

[1] Subdividing into smaller subcatchments and routing the resultant hydrographs using SWMM may be needed to accurately model a catchment with areas of different soil types or percentages of imperviousness.

When modeling large catchments, subcatchment discretization methods and sizes can influence results. If heterogeneous land uses are "lumped" together into large subcatchments, the models may not accurately account for the "flashy" nature of runoff from impervious surfaces, and peak rates of runoff may be underestimated. On the other hand, defining very small subcatchments can lead to complicated and unrealistic routing that can overestimate peak rates of runoff.

The size of subcatchments generally decreases as the level of design progresses from master planning to final site-level design. To discretize subcatchments, first identify design points at key locations where information will be needed for sizing storm infrastructure, then delineate contributing areas upstream of these points. Selecting design points at key hydrologic locations, such as roadway crossings is critical when discretizing catchments and developing hydrologic models to represent the natural and built environments.

The quantity of stormwater runoff from urban areas correlates to watershed characteristics (e.g., imperviousness, soil type, slope, vegetation cover) and the stormwater design practices used to mitigate runoff from the site (e.g., site grading, disconnecting impervious areas, detention facilities, buffer zones, low impact development practices, and other structural and nonstructural stormwater control measures). Implementing nature-based solutions, including Low Impact Development (LID) strategies and green stormwater infrastructure (GSI) can reduce runoff peak, runoff volume, and frequency of stormwater discharges from urban areas. These practices also can reduce erosive flows and treat stormwater prior to discharge to streams. Implementing these practices requires early planning to minimize impacts to sensitive site features, minimizing directly connected impervious areas, promoting onsite infiltration, and treating runoff at the source. Volume 3 of this manual contains additional information on SCMs, including LID and GSI practices.

Photographs 6-2 and 6-3. In natural watersheds, most of the rainfall is lost to interception, depression storage, infiltration, and evapotranspiration, resulting in infrequent runoff. Urbanization, which increases impervious surfaces and compacts pervious areas, increases the peak, volume, and frequency of stormwater runoff. Development practices and stormwater infrastructure that mimic natural watershed processes and use pervious areas for infiltration help to mitigate the adverse effects of urbanization on streams.

3.0 IMPERVIOUSNESS

Determining imperviousness is fundamental to calculating runoff whether using the Rational Method or CUHP. With the Rational Method, runoff coefficients are derived for a range of design storm return periods based on a representative imperviousness of the tributary area. CUHP uses imperviousness as one of several model input parameters and calculates runoff using a unit hydrograph procedure based on regional empirical formulas. CUHP allows users to easily vary imperviousness across different development scenarios (historic, existing, or future conditions).

There are multiple sources of mapping that can be used to estimate imperviousness depending on the stage of the project. During master planning and conceptual design before the specific layout of the development is known, it is common to use imperviousness derived from zoning or land use sources. As the project design advances, a more detailed analysis using imperviousness based on surface types is appropriate and more accurate. The hydrologic scale and resolution of analysis changes as a project progresses from master planning to final design. This requires reevaluating design methods and parametrization. For this reason, MHFD has developed two sets of recommended imperviousness values, presented in Tables 6-2 and 6-3. Table 6-2 provides imperviousness values by typical land use classifications (e.g., single-family, multi-family, industrial, etc.). These values are intended for master planning and at a conceptual design level. Table 6-3 provides imperviousness values for surface types (e.g., roads, roofs, gravel, landscaping, stormwater control measures, etc.). These values are appropriate for site-level designs when the layout of the development is known.

The accuracy of imperviousness calculations depends on the accuracy of source datasets, catchment delineation methods, and geospatial processing methods used. Within the MHFD region, source datasets are updated every few years and include:

- Impervious area or surface cover mapping from local governments or regional agencies such as the Denver Regional Council of Governments (DRCOG),
- Land use or zoning mapping from local governments,
- Land use/land cover mapping from the USGS National Land Cover Database (NLCD), and
- Aerial imagery analysis.

Depending on the underlying data, calculated imperviousness values may vary. Therefore, it is critical to verify that the information used to derive imperviousness represents the intended design conditions, typically existing and future watershed conditions. For watersheds that include substantial urban development, multiple sources of imperviousness data may be available. If this is the case, MHFD recommends that the engineer evaluate available datasets to determine which one provides the most accurate representation of the condition being evaluated.

SOURCES OF IMPERVIOUSNESS DATA FOR MASTER PLANNING

There are several data sources that can be used to determine imperviousness for master planning, ranging from impervious cover mapping for existing conditions to zoning or comprehensive plan projections for future development. There are no “standard” regional data sets because mapping sources often vary from one community to another. Therefore, the engineer must apply judgement to ensure that the data sources used realistically represent the current, future, or historical conditions being evaluated.

Site-specific conditions may vary from the representative values presented in this chapter. The engineer is responsible for assuring that the selected imperviousness values represent the imperviousness of the catchment or the proposed development. During master planning or in early stages of design, select imperviousness values that are unlikely to be exceeded as final design plans are developed to avoid the need to increase the size of infrastructure during later design stages.

TABLE 6-2. RECOMMENDED IMPERVIOUSNESS BY LAND USE

LAND USE/DENSITY	IMPERVIOUSNESS
Residential	
Single-family Housing (SFH) – Rural (0 – 3 du/ac)	35%
SFH – Low & Medium-density (3 – 5 du/ac)	55%
SFH – High-density (5 - 20 du/ac)	65%
Manufactured Housing (>= 10 du/ac)	65%
Multi-family Housing (MFH) – Medium-density (5 – 20 du/ac)	65%
MFH – High-density MFH (>20 du/ac)	70%
Commercial	
Commercial – Low-density	65%
Commercial – Medium- to High-density	80%
Commercial – Urban Core	90%
Industrial/Institutional	
Schools	55%
Office/institutional	65%
Industrial Areas	75%
Solar Fields, Gravel Cover[1,2]	60%
Solar Fields, Grass Cover[1,2]	45%
Parks and Open Space	
Open Space, Undisturbed Native Grasses	5%
Community Parks	25%
Neighborhood Parks	15%
Golf Courses	30%
Cemeteries	25%

Note: Recommended imperviousness values shown in the table are the minimum imperviousness values for a specific land use. It is the engineer's responsibility to select imperviousness values that appropriately reflect the actual density of the proposed development.

[1] Use these values at the master planning scale or when the specific layout of panels is not known. Use values from the surface type (Table 6-3) at the site planning and design stage when panel width, panel spacing, and panel orientation relative to contours are known.

[2] Assumes 1:1 ratio of panels to aisles. See MHFD's technical memorandum regarding *Determination of Solar Panel Field Runoff Coefficients and Imperviousness Values* for additional information on procedures to reflect other impervious areas such as roads and pads that may be part of a solar field and layouts with wider inter-panel spacing.

TABLE 6-3. RECOMMENDED IMPERVIOUSNESS BY SURFACE TYPES

<table>
<tr><th colspan="2">SURFACE TYPES</th><th>IMPERVIOUSNESS</th></tr>
<tr><td colspan="2">Roadways and Paved Streets</td><td>95%</td></tr>
<tr><td colspan="2">Concrete Driveways and Walks</td><td>95%</td></tr>
<tr><td colspan="2">Roofs</td><td>95%</td></tr>
<tr><td rowspan="3">Gravel</td><td>No Traffic (Pedestrian Use)</td><td>40%</td></tr>
<tr><td>Low-traffic Areas (Maintenance Paths and Substations)</td><td>60%</td></tr>
<tr><td>High-traffic Areas (Roadways and Parking)</td><td>80%</td></tr>
<tr><td colspan="2">Disturbed Soil (Including Lawns, Managed/Active Turf, Landscaped Areas with Water-Wise Vegetation, and Uncompacted Gravel/Mulch Planting Beds)</td><td>20%</td></tr>
<tr><td colspan="2">Undisturbed or Decompacted Soil (Native Grasses and Open Space Areas)</td><td>5%</td></tr>
<tr><td rowspan="2">Artificial Turfs[1]</td><td>Landscape Applications (without Subgrade Drainage Layer)</td><td>25% – 45%</td></tr>
<tr><td>Sport Fields (with Underdrain Pipe System)</td><td>60% – 80%</td></tr>
<tr><td colspan="2">Water Surfaces (Lakes/Reservoirs/Irrigation Ponds)</td><td>100%</td></tr>
<tr><td rowspan="2">Solar Fields[2]</td><td>Grass Cover (Varies with Panel Orientation Relative to Ground Contours)</td><td>10% – 45%</td></tr>
<tr><td>Gravel Cover (Varies with Panel Orientation Relative to Ground Contours)</td><td>50% – 75%</td></tr>
<tr><td colspan="2">Historic Flow Analysis, Greenbelts, Agricultural</td><td>5%</td></tr>
<tr><td colspan="2">Newly Graded Areas</td><td>65%</td></tr>
<tr><td rowspan="9">Stormwater Control Measures[3]</td><td>Retention Ponds & Constructed Wetland Ponds</td><td>100%</td></tr>
<tr><td>Rooftop Systems – Blue Roofs</td><td>95%</td></tr>
<tr><td>Rooftop Systems – Green Roofs (extensive)</td><td>65%</td></tr>
<tr><td>Rooftop Systems – Green Roofs (intensive)</td><td>50%</td></tr>
<tr><td>Permeable Pavement – CGP/PGP/RGP</td><td>55%</td></tr>
<tr><td>Permeable Pavement – PICP</td><td>45%</td></tr>
<tr><td>Extended Detention Basins</td><td>25%</td></tr>
<tr><td>Receiving Pervious Areas (incl. Grass Buffers & Grass Swales)</td><td>20%</td></tr>
<tr><td>Bioretention & Sand Filters</td><td>10%</td></tr>
</table>

[1] Consult with the manufacturer to get a recommended value.

[2] Assumes 1:1 ratio of panels to aisles. See MHFD's technical memorandum regarding *Determination of Solar Panel Field Runoff Coefficients and Imperviousness Values* for additional information on procedures for determining percent imperviousness based on panel width, panel spacing, and panel orientation relative to ground contours and how to reflect other impervious areas such as roads and pads that may be part of a solar field and layouts with wider inter-panel spacing.

[3] See MHFD's technical memorandum regarding *Evaluation of Percent Imperviousness for Stormwater Control Measures* for background information.

4.0 RATIONAL METHOD

For urban catchments that are not complex and are generally 90 acres or smaller in size, the Rational Method is acceptable for analysis. The Rational Method, when properly understood and applied, can produce satisfactory results for sizing inlets, storm drains, and small conveyances when a hydrograph is not needed for design.

4.1 RATIONAL FORMULA

The Rational Method is based on the Rational Formula:

$$Q = CIA \qquad \textbf{Equation 6-1}$$

Where:

Q = Peak rate of runoff (cfs)

C = Runoff coefficient, a non-dimensional coefficient equal to the ratio of runoff volume to rainfall volume

I = Average rainfall intensity for a duration equal to time of concentration, t_c, (inches/hour) (see *Rainfall* Chapter)

A = Tributary area (acres)

The peak rate of runoff, Q, has a rate of inches per hour per acre based on dimensional analysis of variables; however, since this rate differs from cubic feet per second (cfs) by less than one percent, the more common units of cfs are used. The time of concentration, t_c , which is used to derive rainfall intensity, represents the time required for water to flow from the most remote point of the catchment to the hydrologic design point and is determined for a path that represents the longest waterway through a rural watershed or the most representative flow path through the impervious portion of an urban catchment.

The general procedure for Rational Method calculations for a single catchment is as follows:

1. Delineate the catchment boundary and determine its area.
2. Define the flow path from the uppermost portion of the catchment to the design point. Divide the flow path into reaches of similar flow type (e.g., overland flow, shallow swale flow, gutter flow, etc.), and determine the length and slope of each reach.
3. Determine t_c for the selected flow path.
4. Find the rainfall intensity, I, for the design storm using the calculated t_c and the site-specific 1-hour point precipitation for the design storm return period using the rainfall intensity equation in the *Rainfall* chapter.
5. Determine the imperviousness of the catchment.
6. Calculate representative runoff coefficients, C, for the desired design storms.
7. Calculate the peak flow rate, Q, from the catchment using Equation 6-1.

4.2 ASSUMPTIONS

The basic assumptions for the application of the Rational Method include:

1. The computed maximum rate of runoff to the design point is a function of the average rainfall rate during the t_c to that point.
2. The hydrologic losses in the catchment are homogeneous and uniform. The runoff coefficients vary with respect to type of soils, imperviousness, and rainfall frequencies. These loss coefficients represent the average antecedent soil moisture condition.
3. The depth of rainfall used is determined from the depth-duration-frequency relationship for the project location based on the selected design storm return period and a duration equal to t_c. The design rainfall depth is converted to the average rainfall intensity over a duration of t_c.
4. The maximum runoff rate occurs when the entire area is contributing flow. This assumption is not valid when there is a more intensely developed portion of the catchment with a shorter t_c that produces a higher rate of runoff than the entire catchment with a longer t_c.

4.3 LIMITATIONS

The Rational Method is a simplistic approach for estimating the peak flow rate from a design storm event in a given catchment. Given the assumption of uniform hydrologic losses, the method is limited to analysis of catchments 90 acres or smaller. Under the condition of composite soils and land uses, use area-weighting method to determine representative catchment imperviousness and derive corresponding runoff coefficients.

The greatest drawback to the Rational Method is that it provides only one point (the peak flow rate) on the runoff hydrograph for a given return period. When drainage areas become complex or where multiple subcatchments come together, the Rational Method will tend to overestimate the actual peak flow, which can result in oversizing of drainage infrastructure. The Rational Method provides no means or methodology to generate and route hydrographs through drainage facilities. One reason the Rational Method is limited to small areas is that good design practice requires routing of hydrographs for larger catchments to achieve economically sound designs.

Another disadvantage of the Rational Method is that with typical design procedures, the engineer assumes that all the design flow is collected at the design point and that there is no overland runoff to the next design point. This is not an issue of the Rational Method but of the design procedure itself and may require additional hydrologic analysis to account for this scenario.

4.4 TIME OF CONCENTRATION

One of the basic assumptions underlying the Rational Method is that runoff is linearly proportional to the average rainfall intensity during the time required for water to flow from the most remote part of the catchment to the design point. In practice, t_c is empirically estimated along the selected drainage path through the catchment.

To calculate the time of concentration, first divide the waterway into overland flow length and channelized flow lengths, according to the drainage characteristics. For urban areas (tributary areas with imperviousness greater than 20%), t_c, consists of an initial time or overland flow time, t_i, plus the channelized flow travel time, t_t, through the storm drain, paved gutter, roadside ditch, or channel. For non-urban areas, t_c consists of an overland flow time, t_i, plus the time of travel in a defined drainage path such as a swale, channel, or stream. Estimate channelized travel time, t_t, from the hydraulic properties of the conveyance element. Initial or overland flow time varies based on factors including slope and length of the flow path, surface cover, depression storage, antecedent rainfall, and infiltration capacity of the soil. Compute the t_c for both urban and non-urban areas using Equation 6-2:

$$t_c = t_i + t_t$$ **Equation 6-2**

Where:

t_c = Time of concentration (minutes)

t_i = Overland (initial) flow time (minutes)

t_t = Channelized flow time (minutes)

4.4.1 INITIAL OR OVERLAND FLOW TIME

Calculate the initial, overland flow time, t_i, using Equation 6-3:

$$t_i = \frac{0.395(1.1 - C_5)\sqrt{L_i}}{S_o^{0.33}}$$ **Equation 6-3**

Where:

t_i = Initial, overland flow time (minutes)

C_5 = Runoff coefficient for 5-year frequency

L_i = Length of overland flow (ft)

S_o = Average slope along the overland flow path (ft/ft)

Equation 6-3 is applicable for distances up to 300 feet in urban areas and up to 500 feet in rural areas. Note that in a highly urbanized catchment, the overland flow length is typically shorter than 300 feet due to effective human-made drainage systems that collect and convey runoff. In undeveloped areas, the overland flow distance can be estimated based on field observations of the distance from the ridge of the catchment to the point where erosional rills begin to form.

4.4.2 CHANNELIZED FLOW TIME

The channelized flow time (travel time) is calculated using the hydraulic properties of the conveyance element. The channelized flow time, t_t, is estimated by dividing the length of conveyance by the velocity. Use Equation 6-4 (Guo 2013) to determine the velocity in conjunction with Table 6-4 for the conveyance factor.

$$t_t = \frac{L_t}{60K\sqrt{S_o}} = \frac{L_t}{60V_t}$$ **Equation 6-4**

Where:

t_t = Travel time of channelized flow (minutes)

K = NRCS conveyance factor (see Table 6-4)

S_o = Waterway slope (ft/ft)

L_t = Waterway length (ft)

V_t = Velocity (ft/sec) = $K\sqrt{S_o}$

TABLE 6-4. NRCS CONVEYANCE FACTORS, K

TYPE OF LAND SURFACE	CONVEYANCE FACTOR, K
Heavy Meadow	2.5
Tillage/Field	5
Short Pasture and Lawns	7
Nearly Bare Ground	10
Grassed Waterway	15
Paved Areas & Shallow Paved Swales	20

Add the initial, overland flow time, t_i, and the channelized flow time, t_t, to calculate t_c using Equation 6-2.

4.4.3 FIRST DESIGN POINT TIME OF CONCENTRATION

Equation 6-4 was developed based on hydraulic drainageway characteristics using a set of empirical formulas. A calibration study between the Rational Method and the CUHP suggests that t_c should be the lesser of the values calculated by Equation 6-2 and the regional t_c calculated using Equation 6-5 (Rapp et al. 2017).

$$t_{regional} = (26 - 17I) + \frac{L_t}{60(14I + 9)\sqrt{S_t}}$$ **Equation 6-5**

Where:

$t_{regional}$ = Regional t_c (minutes) for the Denver area. **Use this as the minimum t_c for the first design point when less than t_c from Equation 6-2.**

L_t = Length of channelized flow path (ft)

I = Imperviousness (expressed as a decimal)

S_t = Slope of the channelized flow path (ft/ft)

Equation 6-5 is the regional t_c that warrants the best agreement on peak flow predictions between the Rational Method and CUHP. It was developed using the MHFD's extensive database of catchments, which represent a broad range of imperviousness and varying areas, slopes, shape factors, and infiltration characteristics for 2-, 5-, 10-, 50, and 100-year design storm events (MacKenzie 2010). Other analysis suggests that both initial flow time and channelized flow velocity are directly related to the catchment's imperviousness (Guo and MacKenzie 2013).

The first hydrologic design point is defined as the location where surface runoff first enters the storm drain system. For example, all inlets are first hydrologic design points because inlets are designed to accept flow into the storm drain system.

Typically, but not always, Equation 6-5 will result in a lesser t_c at the first design point. For subsequent design points, add the travel time for each relevant segment downstream.

4.4.4 MINIMUM TIME OF CONCENTRATION

Use a minimum t_c value of 5 minutes for urbanized areas and a minimum t_c value of 10 minutes for non-urban areas. Use minimum values even when calculations result in a lesser t_c.

4.4.5 COMMON ERRORS IN CALCULATING TIME OF CONCENTRATION

One common mistake in hydrologic analysis of 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 when the upper portion contains grassy open land while the lower portion is more developed.

4.5 RAINFALL INTENSITY

The calculated rainfall intensity, I, is the average rainfall rate in inches per hour over a duration equal to t_c. Obtain 1-hour point precipitation depths from National Oceanic and Atmospheric Administration (NOAA) Atlas 14 for the average return periods of interest and apply Equation 5-1 in the *Rainfall* chapter using t_c as the storm duration, t_d. Use the centroid of the catchment to determine the 1-hour point precipitation depths. The MHFD-Rational and MHFD-Inlet Excel workbooks automatically calculate rainfall intensity based on 1-hour point precipitation depths for a specified location.

4.6 RUNOFF COEFFICIENTS

Any watershed can be conceptualized as a combination of pervious and impervious surfaces. Pervious surfaces allow water to infiltrate into the ground, while impervious surfaces do not allow for infiltration. In urban hydrology, the relationships between pervious and impervious surfaces is important. Urbanization increases impervious area, causing rainfall-runoff relationships to change significantly. In the absence of stormwater management controls that infiltrate or detain runoff, urbanization increases peak runoff rates, volumes, and frequency of runoff and decreases the time to peak.

When analyzing a catchment for planning or design purposes, estimates of the existing and probable future imperviousness of the drainage area are needed. In some cases, the pre-development (i.e., historic) condition also must be analyzed. Table 6-2 provides recommended imperviousness values based on land use types and is appropriate for master planning analysis and conceptual design. Note that the land use classifications in Table 6-2 incorporate roads that are included within the land use. Table 6-3 provides recommended imperviousness values for different surface types and is appropriate for use during later stages of design when the layout of different types of impervious and pervious areas on the site is known and the area of each surface type can be quantified.

The runoff coefficient, C, represents the integrated effects of infiltration, evaporation, depression storage, and interception, all of which affect the rate and volume of runoff. Determining representative runoff coefficients requires judgment based on the experience and expertise of the engineer.

Volume-based runoff coefficients were derived to improve consistency between CUHP and the Rational Method for peak flow predictions (Guo 2013; Guo and Urbonas 2013). The coefficients developed by Dr. Guo were recalibrated using CUHP Version 2.0.0 (Rapp et al. 2017). Using imperviousness, expressed as a decimal, and the Natural Resources Conservation Service (NRCS) Hydrologic Soil Group (HSG), the equations in Table 6-5 can be used to calculate runoff coefficients for design storm return periods for the Rational Method.

TABLE 6-5. RUNOFF COEFFICIENT EQUATIONS BASED ON NRCS HSG AND STORM RETURN PERIOD

NRCS HSG	STORM RETURN PERIOD						
	WQE & 2-Year	5-Year	10-Year	25-Year	50-Year	100-Year	500-Year
A	$C_A = 0.840I^{1.302}$	$C_A = 0.861I^{1.276}$	$C_A = 0.873I^{1.232}$	$C_A = 0.884I^{1.124}$	$C_A = 0.854I + 0.025$	$C_A = 0.779I + 0.110$	$C_A = 0.645I + 0.254$
B	$C_B = 0.835I^{1.169}$	$C_B = 0.857I^{1.088}$	$C_B = 0.807I + 0.057$	$C_B = 0.628I + 0.249$	$C_B = 0.558I + 0.328$	$C_B = 0.465I + 0.426$	$C_B = 0.366I + 0.536$
C/D	$C_{C/D} = 0.834I^{1.122}$	$C_{C/D} = 0.815I + 0.035$	$C_{C/D} = 0.735I + 0.132$	$C_{C/D} = 0.560I + 0.319$	$C_{C/D} = 0.494I + 0.393$	$C_{C/D} = 0.409I + 0.484$	$C_{C/D} = 0.315I + 0.588$

Where:

I = Weighted imperviousness of catchment **expressed as a decimal**

C_A = Runoff coefficient for NRCS HSG A soils

C_B = Runoff coefficient for NRCS HSG B soils

$C_{C/D}$ = Runoff coefficient for NRCS HSG C and D soils

The values for various catchment imperviousness and storm return periods are tabulated in Tables 6-6 through 6-8 and presented graphically in Figures 6-1 through 6-3. These coefficients were developed for the Denver region to work in conjunction with the t_c criteria in Section 4.4. Use of these coefficients and this procedure outside of the semi-arid climate found in the Denver region may not be valid. The MHFD-Rational Excel workbook performs calculations to determine the runoff coefficient based on the HSG, the design storm return period, and imperviousness and is available at www.mhfd.org.

See Examples 13.1 and 13.2 for application of the Rational Method.

TABLE 6-6. RUNOFF COEFFICIENTS, *C*, NRCS HSG A

TOTAL OR EFFECTIVE % IMPERVIOUS	NRCS HSG A						
	WQE & 2-Year	5-Year	10-Year	25-Year	50-Year	100-Year	500-Year
2%	0.01	0.01	0.01	0.01	0.04	0.13	0.27
5%	0.02	0.02	0.02	0.03	0.07	0.15	0.29
10%	0.04	0.05	0.05	0.07	0.11	0.19	0.32
15%	0.07	0.08	0.08	0.10	0.15	0.23	0.35
20%	0.10	0.11	0.12	0.14	0.20	0.26	0.38
25%	0.14	0.15	0.16	0.19	0.24	0.30	0.42
30%	0.18	0.19	0.20	0.23	0.28	0.34	0.45
35%	0.21	0.23	0.24	0.27	0.32	0.38	0.48
40%	0.25	0.27	0.28	0.32	0.37	0.42	0.51
45%	0.30	0.31	0.33	0.36	0.41	0.46	0.54
50%	0.34	0.36	0.37	0.41	0.45	0.50	0.58
55%	0.39	0.40	0.42	0.45	0.49	0.53	0.61
60%	0.43	0.45	0.47	0.50	0.54	0.57	0.64
65%	0.48	0.50	0.51	0.54	0.58	0.61	0.67
70%	0.53	0.55	0.56	0.59	0.62	0.65	0.71
75%	0.58	0.60	0.61	0.64	0.67	0.69	0.74
80%	0.63	0.65	0.66	0.69	0.71	0.73	0.77
85%	0.68	0.70	0.71	0.74	0.75	0.76	0.80
90%	0.73	0.75	0.77	0.79	0.79	0.80	0.83
95%	0.79	0.81	0.82	0.83	0.84	0.84	0.87
100%	0.84	0.86	0.87	0.88	0.88	0.88	0.90

TABLE 6-7. RUNOFF COEFFICIENTS, *C*, NRCS HSG B

TOTAL OR EFFECTIVE % IMPERVIOUS	NRCS HSG B						
	WQE & 2-Year	5-Year	10-Year	25-Year	50-Year	100-Year	500-Year
2%	0.01	0.01	0.07	0.26	0.34	0.44	0.54
5%	0.03	0.03	0.10	0.28	0.36	0.45	0.55
10%	0.06	0.07	0.14	0.31	0.38	0.47	0.57
15%	0.09	0.11	0.18	0.34	0.41	0.50	0.59
20%	0.13	0.15	0.22	0.37	0.44	0.52	0.61
25%	0.17	0.19	0.26	0.41	0.47	0.54	0.63
30%	0.20	0.23	0.30	0.44	0.50	0.57	0.65
35%	0.24	0.27	0.34	0.47	0.52	0.59	0.66
40%	0.29	0.32	0.38	0.50	0.55	0.61	0.68
45%	0.33	0.36	0.42	0.53	0.58	0.64	0.70
50%	0.37	0.40	0.46	0.56	0.61	0.66	0.72
55%	0.42	0.45	0.50	0.59	0.63	0.68	0.74
60%	0.46	0.49	0.54	0.63	0.66	0.71	0.76
65%	0.50	0.54	0.58	0.66	0.69	0.73	0.77
70%	0.55	0.58	0.62	0.69	0.72	0.75	0.79
75%	0.60	0.63	0.66	0.72	0.75	0.77	0.81
80%	0.64	0.67	0.70	0.75	0.77	0.80	0.83
85%	0.69	0.72	0.74	0.78	0.80	0.82	0.85
90%	0.74	0.76	0.78	0.81	0.83	0.84	0.87
95%	0.79	0.81	0.82	0.85	0.86	0.87	0.88
100%	0.84	0.86	0.86	0.88	0.89	0.89	0.90

TABLE 6-8. RUNOFF COEFFICIENTS, *C*, NRCS HSG C/D

TOTAL OR EFFECTIVE % IMPERVIOUS	NRCS HSG C/D						
	WQE & 2-Year	5-Year	10-Year	25-Year	50-Year	100-Year	500-Year
2%	0.01	0.05	0.15	0.33	0.40	0.49	0.59
5%	0.03	0.08	0.17	0.35	0.42	0.50	0.60
10%	0.06	0.12	0.21	0.38	0.44	0.52	0.62
15%	0.10	0.16	0.24	0.40	0.47	0.55	0.64
20%	0.14	0.20	0.28	0.43	0.49	0.57	0.65
25%	0.18	0.24	0.32	0.46	0.52	0.59	0.67
30%	0.22	0.28	0.35	0.49	0.54	0.61	0.68
35%	0.26	0.32	0.39	0.52	0.57	0.63	0.70
40%	0.30	0.36	0.43	0.54	0.59	0.65	0.71
45%	0.34	0.40	0.46	0.57	0.62	0.67	0.73
50%	0.38	0.44	0.50	0.60	0.64	0.69	0.75
55%	0.43	0.48	0.54	0.63	0.66	0.71	0.76
60%	0.47	0.52	0.57	0.66	0.69	0.73	0.78
65%	0.51	0.56	0.61	0.68	0.71	0.75	0.79
70%	0.56	0.61	0.65	0.71	0.74	0.77	0.81
75%	0.60	0.65	0.68	0.74	0.76	0.79	0.82
80%	0.65	0.69	0.72	0.77	0.79	0.81	0.84
85%	0.69	0.73	0.76	0.80	0.81	0.83	0.86
90%	0.74	0.77	0.79	0.82	0.84	0.85	0.87
95%	0.79	0.81	0.83	0.85	0.86	0.87	0.89
100%	0.84	0.86	0.87	0.88	0.89	0.89	0.90

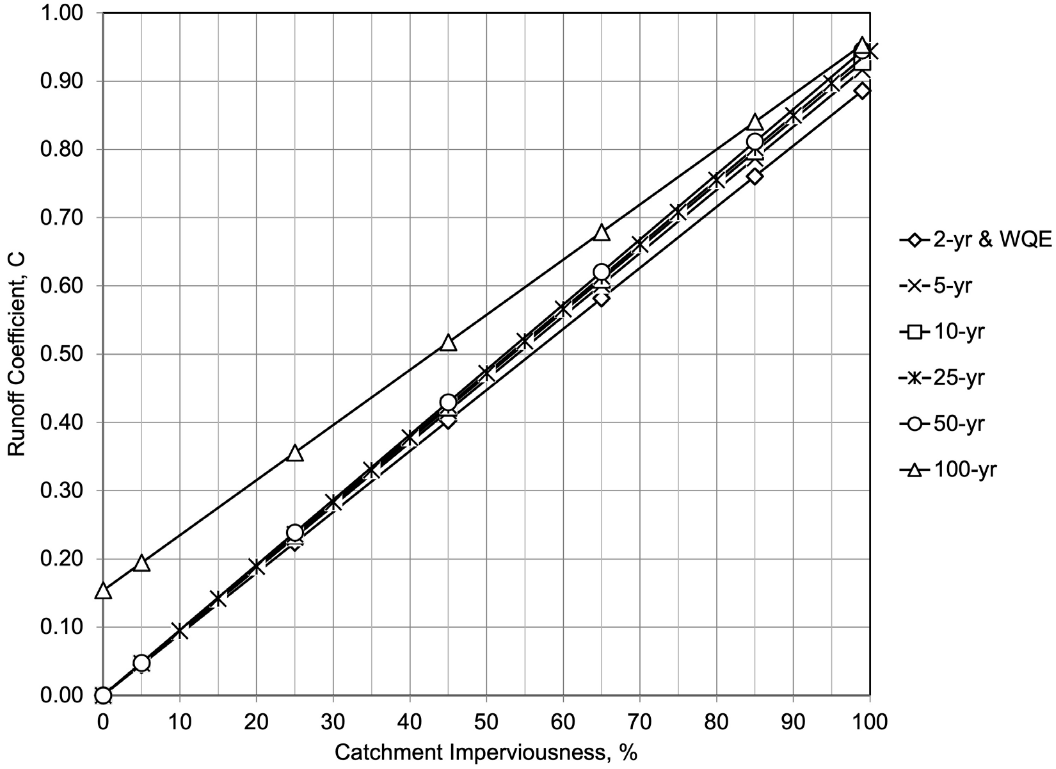

FIGURE 6-1. RUNOFF COEFFICIENT VS. CATCHMENT IMPERVIOUSNESS NRCS HSG A

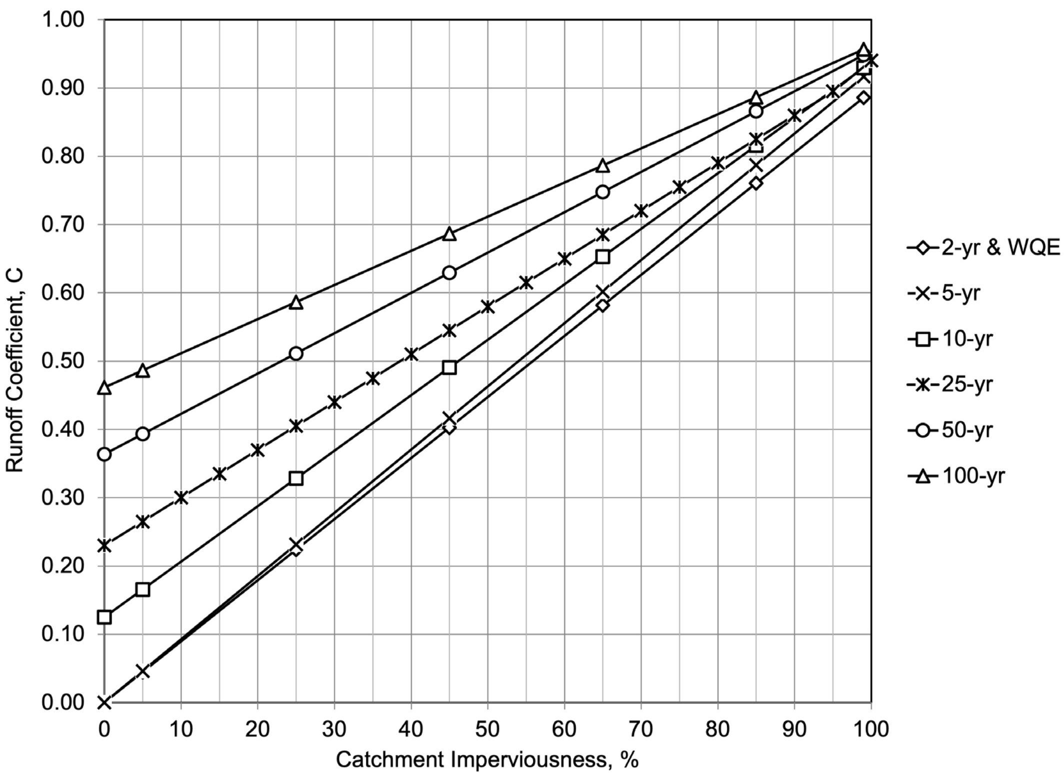

FIGURE 6-2. RUNOFF COEFFICIENT VS. CATCHMENT IMPERVIOUSNESS NRCS HSG B

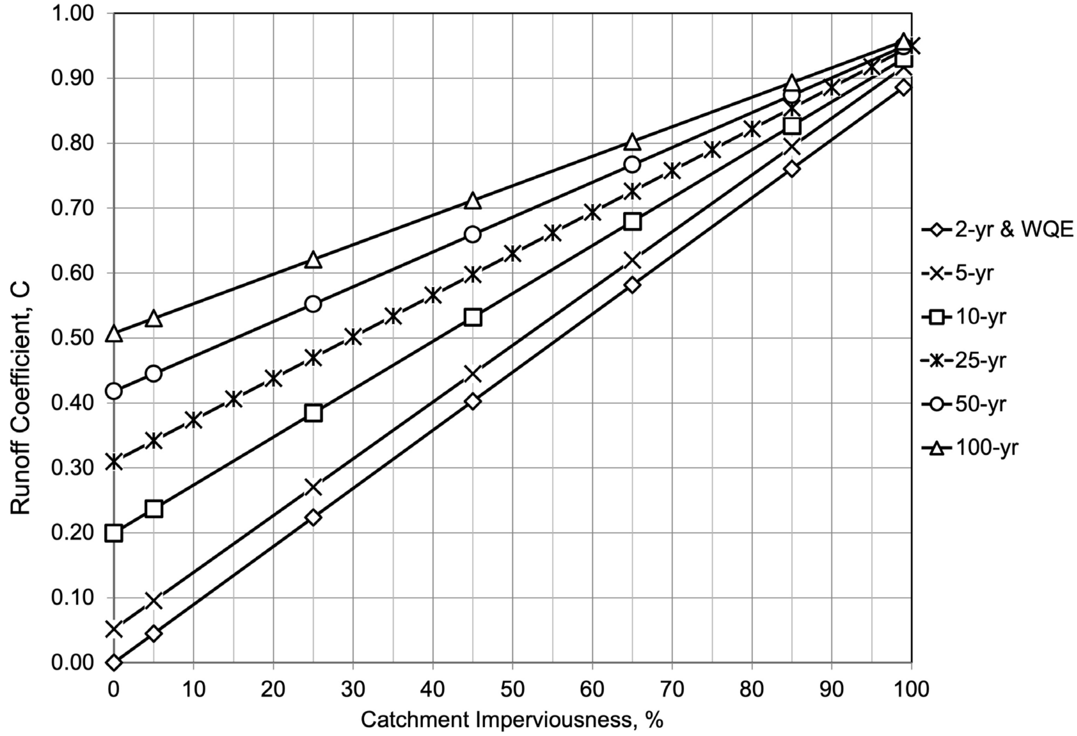

FIGURE 6-3. RUNOFF COEFFICIENT VS. CATCHMENT IMPERVIOUSNESS NRCS HSG C/D

5.0 COLORADO URBAN HYDROGRAPH PROCEDURE (CUHP)

5.1 BACKGROUND

CUHP is a method of hydrologic analysis based on unit hydrograph principles. A unit hydrograph is defined as the hydrograph of one inch of direct runoff from the tributary area resulting from a storm of a given duration. The unit hydrograph approach incorporates the integrated effects of factors such as tributary area, shape, street pattern, channel capacities, and street and land slopes. The basic premise of the unit hydrograph is that individual hydrographs resulting from the successive increments of excess rainfall that occur throughout a storm period are proportional in discharge throughout their runoff period. Thus, the hydrograph of total storm discharge can be obtained by summing the ordinates of the individual sub-hydrographs.

CUHP has been developed and calibrated using rainfall-runoff data collected in Colorado (mostly in the greater Denver metropolitan area). This section provides general background on the use of the computer version of CUHP to perform stormwater runoff calculations. A detailed description of the CUHP method and the assumptions and equations used, including a hand calculation example, are provided in the CUHP User Manual. The latest version of the CUHP 2005 macro-enabled Excel workbook and User Manual are available for download from www.mhfd.org.

5.2 EFFECTIVE RAINFALL FOR CUHP

Effective rainfall, or excess rainfall, is the portion of precipitation during a storm event that reaches the outlet of a catchment as runoff. Precipitation that does not reach the catchment outlet is “lost” to hydrologic processes, including interception, depression storage, evaporation, and infiltration. Use NOAA Atlas 14 point-precipitation depths for a given location (discussed in more detail in the *Rainfall* chapter) for CUHP input.

5.2.1 PERVIOUS-IMPERVIOUS AREAS

As described in Section 4.6, the urban landscape consists of pervious and impervious surfaces. Imperviousness is a primary variable affecting the volumes and rates of runoff calculated using CUHP. When analyzing a catchment for design purposes, the existing and probable future percent of imperviousness must be estimated. In some cases, historic (pre-development) conditions also must be evaluated. References to imperviousness and all calculations in this chapter are based on the input of **total** impervious areas (i.e., directly connected and unconnected impervious area, combined).

The pervious-impervious area relationships in CUHP are based on:

- **Directly Connected Impervious Area (DCIA):** Impervious area that drains directly to the drainage system.
- **Unconnected Impervious Area (UIA):** Impervious area that drains onto or across pervious surfaces.
- **Receiving Pervious Area (RPA):** Pervious area that receives runoff from unconnected impervious area.
- **Separate Pervious Area (SPA):** Pervious area that does not receive runoff from impervious surfaces.

The CUHP User Manual and Volume 3 provide more detail on pervious-impervious area relationships and effects on runoff.

5.2.2 DEPRESSION STORAGE

Precipitation collected and held in small depressions that does not become part of surface runoff is called depression storage. Most of this water eventually infiltrates or evaporates. Table 6-9 can be used as a guide in estimating the amount of depression storage (retention) losses to be used with CUHP. The depression storage parameters in Table 6-9 also account for water intercepted by trees, bushes, vegetation, and other surfaces. CUHP requires input of depression storage to calculate the effective rainfall.

TABLE 6-9. TYPICAL DEPRESSION STORAGE FOR VARIOUS LAND COVERS OR SURFACE TYPES (ALL VALUES IN WATERSHED INCHES FOR USE WITH THE CUHP)

LAND COVER/SURFACE TYPE	DEPRESSION STORAGE (INCHES)	RECOMMENDED (INCHES)
Impervious		
Large, Paved Areas	0.05 – 0.15	0.10
Roofs – Flat	0.10 – 0.30	0.10
Roofs – Sloped	0.05 – 0.10	0.05
Pervious		
Lawn Grass	0.20 – 0.50	0.35
Wooded Areas and Open Fields	0.20 – 0.60	0.40

When an area is analyzed for depression storage, consider the pervious and impervious depression storage values for all parts of the watershed as an area-weighted average in proportion to the percent of aerial coverage for each representative surface type.

5.2.3 INFILTRATION

Infiltration is the process of rainfall penetrating the ground surface into the soil. In urban hydrology, much of the infiltration occurs in areas covered with vegetation. Urbanization can increase or decrease the total amount of infiltration depending on how runoff is managed, historical and proposed uses of the area, and other factors, but generally, infiltration decreases as areas urbanize.

Soil characteristics are the most important factor in determining infiltration rates. When the soil has a large percentage of well-graded fines, the infiltration rate is low. In some cases of extremely tight soil or highly compacted areas, there may be essentially no infiltration from a practical standpoint. If the soil has several layers or horizons, the least permeable layer near the surface will control the maximum infiltration rate. Soil cover also plays an important role in determining the infiltration rate. Vegetation, grass in particular, tends to increase infiltration by loosening the soil near the surface via root systems. Other factors affecting infiltration rates include the ground slope, temperature, water quality, landscaping characteristics, and soil compaction.

During a storm event, the infiltration rate decreases with time. When rainfall occurs in an area that has little antecedent moisture and the ground is dry, the infiltration rate can be much higher than it is with high antecedent moisture resulting from previous storms or landscaping irrigation. Although antecedent precipitation is important when calculating runoff from smaller storms in non-urbanized areas, runoff data from urbanized watersheds indicate that antecedent precipitation has a smaller effect on runoff peaks and volumes in highly-urbanized areas of MHFD.

There are many infiltration models in use by hydrologists that vary significantly in complexity. Because of the semi-arid climate conditions in the MHFD region and because runoff from urban watersheds is not very sensitive to infiltration refinements, the infiltration model proposed by Horton was found to provide a good balance between simplicity and reasonable physical description of the infiltration process for use in CUHP. Equation 6-6 describes Horton's infiltration model.

$$f = f_o + (f_i - f_o)e^{-at}$$ **Equation 6-6**

Where:

f = Infiltration rate at any given time t from start of rainfall (in/hr)

f_o = Final infiltration rate (in/hr)

f_i = Initial infiltration rate (in/hr)

e = Natural logarithm base

a = Decay coefficient (1/second)

t = Time (seconds)

In developing Equation 6-6, Horton observed that infiltration is high early in the storm and eventually decays to a steady state constant value as the pores in the soil become saturated. The coefficients and initial and final infiltration values are site-specific and depend on the underlying soils and vegetative cover. With sufficient rainfall-runoff observations, it is possible to develop these values for a specific site.

Since 1977, MHFD has collected and analyzed considerable rainfall-runoff data across the Front Range. Based on this work, MHFD recommends using the values in Table 6-10 when applying CUHP. The NRCS HSGs C and D occur most frequently within MHFD; however, areas of NRCS HSG A and B soils also exist. Consult NRCS soil surveys for appropriate HSGs for the project location.

TABLE 6-10. RECOMMENDED HORTON'S EQUATION PARAMETERS

NRCS HYDROLOGIC SOIL GROUP	INFILTRATION (INCHES PER HOUR)		DECAY COEFFICIENT – a
	INITIAL – f_i	FINAL – f_o	
A	5.0	1.0	0.0007
B	4.5	0.6	0.0018
C	3.0	0.5	0.0018
D	3.0	0.5	0.0018

To calculate the maximum infiltration depths that may occur at each time increment, Equation 6-6 must be integrated. Very little accuracy is lost if, instead of integrating Equation 6-6, the infiltration rate is calculated at the center of each time increment. This "central" value can then be multiplied by the unit time increment to estimate the infiltration depth. Although Table 6-10 provides recommended values for various Horton equation parameters, these recommendations are made specifically for the urbanized or urbanizing watersheds in the Denver metropolitan area and may not be valid in different meteorological and climatic regions.

In some cases, including forensic reconstructions of flood events and evaluations of runoff from frequently occurring storm events where infiltration is a large fraction of rainfall, site-specific evaluations of soil types and infiltration characteristics may be needed to accurately represent runoff, beyond the simplified HSG-based rates in Table 6-10.

5.3 CUHP PARAMETER SELECTION

5.3.1 RAINFALL

CUHP requires input of rainfall precipitation depths to develop design storms, either as program-generated hyetographs using 1-hour and 6-hour rainfall depths or as user-defined hyetographs to evaluate historical storm events. CUHP generates a hyetograph based on a 1-hour point precipitation depth and the standard 2-hour temporal distribution recommended in the *Rainfall* chapter for different return periods. In addition, the program can generate a 6-hour storm distribution with area corrections applied in cases where larger catchments are studied to account for spatial variability in storms.

When catchments are large enough to require application of Depth Reduction Factors (DRFs), multiple CUHP/SWMM model runs are required to correctly apply the DRF to the tributary area draining to each design point. The average rainfall over a large catchment is generally lower than point rainfall from NOAA Atlas 14; therefore, the DRF is applied to reduce point rainfall depths to area-average rainfall depths over the entire catchment in accordance with the *Rainfall* chapter. This modeling approach attempts to account for variability within a storm where intensity may be greater over a portion of the catchment at any given time. This avoids overestimating the total rainfall volume for large catchments since the highest intensity rainfall is not uniform over the entire drainage area. However, it is still necessary to evaluate the critical storm intensity within individual portions of the catchment. This requires evaluation of peak discharges with varying DRF adjustments at key design points in the catchment. For example, subcatchments in the headwaters will not require application of DRFs; however, as multiple subcatchments are combined moving downstream, DRFs must be applied to determine the appropriate peak discharges to use at various locations along the major drainageway. This process requires careful organization of model results to be sure that peak flows at each design point represent the effects of the DRFs for the drainage area tributary to that design point. See the example in Section 13.3.

5.3.2 CATCHMENT PARAMETERS

The following catchment parameters are required for CUHP to generate unit and storm hydrographs.

- **Area:** Catchment area in square miles (or acres). See Table 6-1 for catchment size limits. Typically, a 5-minute unit hydrograph is used in CUHP. However, for catchments smaller than approximately 90 acres, MHFD recommends using a 1-minute unit hydrograph time step, particularly if there are significant differences between the “excess precipitation” and “runoff hydrograph” volumes listed in the summary output. For very small catchments (i.e., smaller than 10 acres) and those with high imperviousness, a 1-minute time step may be needed to preserve runoff volume integrity.
- **Length:** The length in miles (or feet) from the downstream design point of the catchment along the main flow path to the furthest point on its respective catchment boundary (i.e., the longest flow path in the catchment). When subdividing a catchment into a series of subcatchments, the subcatchment length shall include the distance required for runoff to reach the major drainageway from the farthest point in the subcatchment.
- **Length to Centroid:** Distance in miles (or feet) from the design point of the catchment along the longest flow path to a point perpendicular to its respective catchment centroid. Length to centroid does not include the distance from the longest flow path to the centroid itself.
- **Slope:** The length-weighted, corrected average catchment slope in feet per foot (ft/ft).
    - » There are natural processes at work that limit the time to peak of a unit hydrograph as a natural stream or vegetated channel becomes steeper. To account for this phenomenon, adjust the slope used in CUHP for streams and vegetated channels using Figure 6-4.
    - » When a riprap channel is evaluated, use the measured (i.e., uncorrected) average channel invert slope.
    - » In concrete-lined channels and buried conduits, the velocities can be very high. For this reason, MHFD recommends using the average ground slope (i.e., not flow-line slope) where concrete-lined channels and/or storm drains dominate. There is no correction factor or upper limit recommended for the slope of concrete-lined channels and buried conduits.
    - » When drop structures are present in channels, use the average slope of the channel between drop structures to account for the effects of drop structures in reducing slope and velocities.

Figure 6-4, which applies to natural channels and vegetated channels without grade controls, shows a linear relationship for slopes of 0.04 ft/ft and less, and no slope correction is needed for this range of measured slopes. For measured slopes greater than 0.04 ft/ft, calculate the slope correction factor using Equation 6-7.

$$S_A = 61.713\ S_M^{\ 3} - 18.517\ S_M^{\ 2} + 1.9376\ S_M - 0.0117\ S_M$$ **Equation 6-7**

Where:

S_M = Measured slope (ft/ft)

S_A = Adjusted slope (ft/ft)

Note that Equation 6-7 is only applicable for measured slopes ranging from 0.04 to 0.12 feet per feet. **If the measured slope exceeds 0.12 feet/feet, use an adjusted slope of 0.06 feet/feet.**

Where the flow-line slope varies along a concentrated flow path or channel, calculate a length-weighted corrected average catchment slope for use with CUHP. Do this by segmenting the major drainageway into reaches having similar longitudinal slopes, and calculate the weighted slope using Equation 6-8.

$$S = \left[\frac{L_1 S_1^{\ 0.24} + L_2 S_2^{\ 0.24} + ... + L_n S_n^{\ 0.24}}{L_1 + L_2 + L_3 + ...L_n} \right]^{4.17}$$ **Equation 6-8**

Where:

S = Weighted basin waterway slopes (ft/ft)

$S_1, S_2, ... S_n$ = Slopes of individual reaches, after adjustments using Figure 6-4 (ft/ft)

$L_1, L_2, ... L_n$ = Lengths of corresponding reaches (ft)

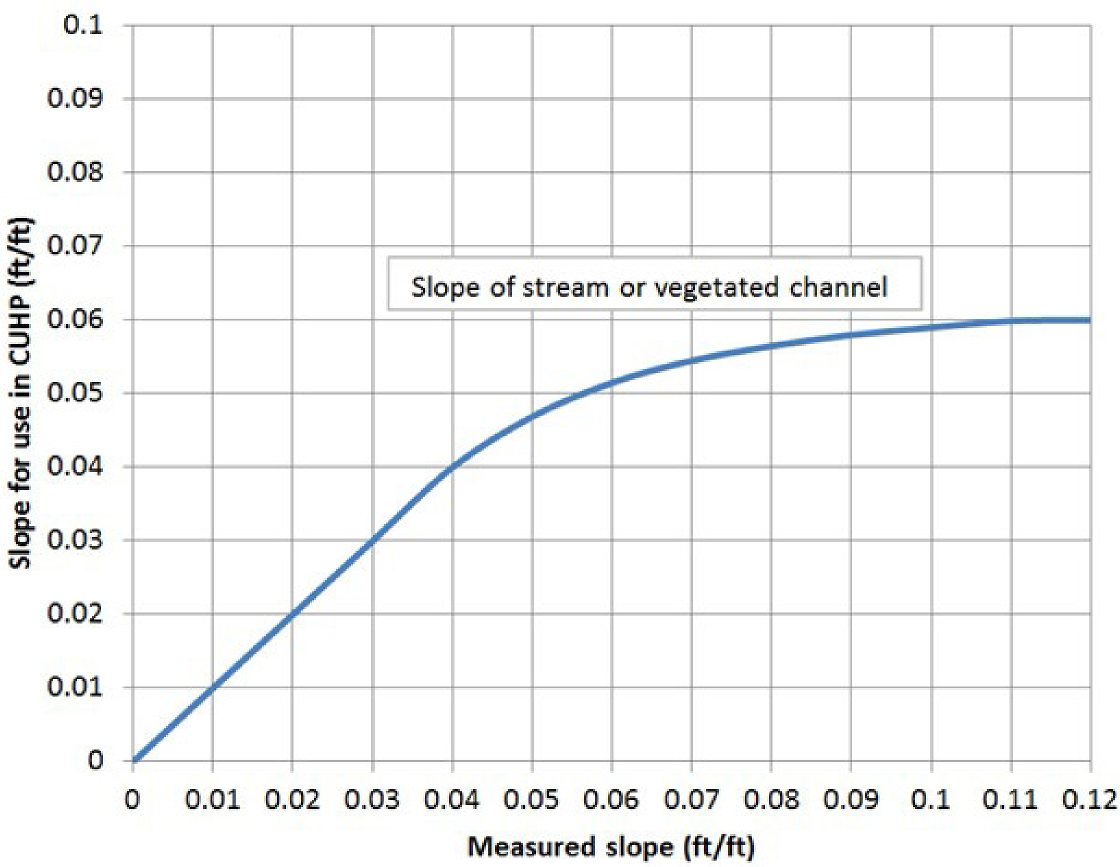

FIGURE 6-4. SLOPE CORRECTION FOR STREAMS AND VEGETATED CHANNELS

- **Percent Imperviousness:** The portion of the catchment's total surface area that is impervious, expressed as a percentage between 0 and 100. (See Section 3.0 for more details).
- **Maximum Pervious Depression Storage:** Maximum depression storage on pervious surfaces in inches. (See Table 6-9).
- **Maximum Impervious Depression Storage:** Maximum depression storage on impervious surfaces in inches. (See Table 6-9).
- **Initial Infiltration Rate:** Initial infiltration rate for pervious surfaces in inches per hour. When entered without a decay coefficient and final infiltration rate, this value becomes a constant infiltration rate throughout the storm (not recommended). (See Table 6-10).
- **Horton's Decay Coefficient:** Exponential decay coefficient in Horton's equation in "per second" units (1/sec). (See Table 6-10).
- **Final Infiltration Rate:** Final infiltration rate in Horton's equation in inches per hour. (See Table 6-10).

The following catchment parameters are optional inputs available to the user to evaluate effects of directly connected and unconnected impervious areas:

- **DCIA Level:** Specifies the DCIA level of practice as defined in Volume 3. The user may specify 0, 1, or 2 for the level of DCIA to model.
- **Directly Connected Impervious Fraction:** Defines the fraction of the total impervious area directly connected to the drainage system. Values range from 0.01 to 1.0.
- **Receiving Pervious Fraction:** Defines the fraction of overall site pervious area that receives runoff from the unconnected impervious areas. Values range from 0.01 to 1.0.

To assist in the determination of the time to peak and peak runoff for the unit hydrograph, CUHP computes the coefficients C_T and C_p; however, the user can also enter override values for these parameters.

The algorithm described in the CUHP 2005 User Manual develops the unit hydrograph.

- C_T: An unmodified time to peak coefficient that relates the total imperviousness of a catchment to the time to peak.
- C_p: Peak runoff rate coefficient determined from C_t and the peaking parameter, P.

The unit hydrograph shaping also relies on proportioning the widths at 50% and 75% of the unit hydrograph peak. The proportioning is based on 0.35 of the width at 50% of peak being ahead of the time to peak and 0.45 of the width at 75% of peak being ahead of the time to peak. These proportioning factors were selected after observing a number of unit hydrographs derived from the rainfall-runoff data collected by the USGS and MHFD. The user can override the unit hydrograph widths and the proportioning of these widths built into the program. For drainage and flood studies within MHFD, use the default program values. If the user has derived unit hydrographs from reliable rainfall-runoff data for a study catchment, the user can develop a "calibrated" unit hydrograph for this catchment and reshape the unit hydrograph accordingly. See the CUHP User Manual for more information on applying these adjustment factors.

5.3.3 CATCHMENT DELINEATION CRITERIA

MHFD recommends an average catchment size of approximately 100 acres for master planning purposes. As engineering progresses from master planning to more detailed design, smaller catchment sizes are used based on locations of design points where peak flows or hydrographs are needed for sizing of infrastructure. See Table 6-1 for a description of catchment size limitations for CUHP.

The catchment shape can have a profound effect on the results and, in some instances, can result in underestimates of peak flows. Experience with earlier versions of CUHP showed that whenever catchment length is increased faster than its area, the storm hydrograph peak tended to decrease disproportionately. Although hydrologic routing is an integral part of runoff analysis, the data used to develop CUHP are insufficient to say that the observed CUHP response with disproportionately increasing basin length is valid. For this reason, subdivide irregularly shaped or very long catchments (i.e., catchment length to width ratio of four or more) into more regularly shaped subcatchments. A composite catchment storm hydrograph can then be developed using appropriate routing to combine the individual subcatchment storm hydrographs.

GEOPROCESSING SOFTWARE

When using geoprocessing software for delineating subcatchments in master plans, it is often easier to start by defining smaller catchments (30 – 50 acres) that can be aggregated into larger catchments as needed based on design points.

5.3.4 COMBINING AND ROUTING SUBCATCHMENT CUHP HYDROGRAPHS

When analyzing numerous subcatchments, hydrographs from subcatchments must be combined and routed through the drainage system. CUHP allows the modeler to specify the target node in EPA SWMM where each subcatchment hydrograph will be linked. CUHP then generates an output text file that SWMM recognizes as an external inflow file. The CUHP User Manual provides a detailed description of these features and more.

6.0 EPA SWMM AND HYDROGRAPH ROUTING

EPA SWMM is a rainfall-runoff simulation model used for a single event or long-term (continuous) simulation of runoff and routing through a drainage system. The rainfall-runoff component of SWMM calculates runoff hydrographs from subcatchments, and the routing component of SWMM translates these runoff hydrographs through a system of storm drains, channels, and storage/treatment facilities. The procedures described in this chapter are limited to using SWMM only for the routing component and relying on CUHP for the rainfall-runoff component. Originally the routing component was performed using UDSWMM, a modified version of EPA SWMM runoff calculations designed to work directly with CUHP. In 2005, MHFD adopted EPA SWMM 5.0 and recommended using the most current version of EPA SWMM for all future hydrology studies.

The discussion in this chapter provides general background on using SWMM in conjunction with CUHP to perform more runoff calculations. Additional details about this model's use and specifics of data formats are provided in the SWMM User Manual (Rossman 2015). The SWMM software, user manual, and background information about EPA SWMM may be downloaded from EPA's SWMM website.

6.1 SOFTWARE DESCRIPTION

SWMM represents a catchment as an aggregate of idealized runoff planes, channels, gutters, pipes, and specialized units such as storage nodes, outlets, pumps, etc. The program can accept rainfall hyetographs and perform step-by-step accounting of rainfall infiltration losses in pervious areas, surface retention, overland flow, and gutter flow leading to the generation of hydrographs. However, this portion of the model is normally not used by MHFD because the resulting peak flows and runoff volumes are not calibrated to MHFD regional rainfall-runoff observations in the same way CUHP storm hydrographs are. Instead, generation of hydrographs for each subcatchment is carried out using CUHP. If the user wants to use SWMM to calculate runoff, the model must be calibrated against CUHP runoff hydrographs for each subcatchment being studied.

After CUHP software is used to generate hydrographs from a number of subcatchments, the resulting hydrographs from these subcatchments are combined and routed through a series of links (i.e., channels, gutters, pipes, dummy links, etc.) and nodes (i.e., junctions, storage, diversion, etc.) in SWMM to compute the resultant hydrographs and related design information (flooding of nodes, depths and velocities in conduits, etc.) at all design point within the catchment.

6.1.1 SURFACE FLOWS AND FLOW ROUTING FEATURES

Stormwater runoff hydrographs generated using CUHP are routed through a system of stormwater conveyance, diversion, and storage elements in a complex urban drainage system. In setting up SWMM, establish design points at locations where hydrologic and hydraulic information will be needed for design. To evaluate combined flow in storm drains or other conveyances along streets, use a diversion junction and parallel links representing the storm drain or channel and the road. This allows the user to model the common situation that occurs when pipes and/or channels do not have the capacity to convey higher flows and allow excess flows to be diverted to overflow channels (often streets). This method avoids flooded nodes and other related errors in the calculated peak flow values downstream.

There are several types of conveyance elements used in SWMM. One option is a user-defined irregular channel cross-section, similar to cross-sections as they are defined in HEC-RAS. This makes the model very flexible in modeling natural waterways and composite man-made channels. For a complete description of the routing elements and junction types available for modeling, see the SWMM User's Manual (Rossman 2015).

6.1.2 FLOW ROUTING METHOD OF CHOICE

MHFD recommends the kinematic wave routing option in SWMM for planning purposes because flood flows are generally dominated by kinematic waves (USACE 1993). Dynamic wave routing is appropriate when inertial and pressure forces are important and when evaluating complex existing elements within a larger system. Dynamic wave routing is an option that can also offer some advantages in final design, as it provides hydraulic grade lines and accounts for backwater effects by solving the complete St. Venant flow equations.

For most applications, kinematic wave routing is recommended due to the detailed information needed to accurately model dynamic wave conditions and the tendency for models to become unstable when analyzing more complex elements and/or junctions. Much of the required detail for an accurate dynamic wave model is not typically available during the master planning and conceptual design phases of a project.

6.2 DATA PREPARATION FOR THE SWMM SOFTWARE

Use of SWMM requires three basic steps:

Step 1: Identify design points, discretize subcatchments, and the determine the geometric characteristics of subcatchments and conveyance/storage elements.

Step 2: Estimate roughness coefficients and functional/tabular relationships for storage and other special elements.

Step 3: Prepare input data for the model.

CHANNEL SLOPE AND ROUTING FOR NETWORKS OF SMALL SUBCATCHMENTS

When smaller subcatchments are used to represent details of a drainage system, the smaller subcatchments will tend to produce more runoff (cfs/acre) than larger catchments. The following recommendations can help eliminate the effect of this increase and provide more realistic hydraulic routing through the conveyance network in EPA SWMM:

- Carefully estimate the effective longitudinal channel slope instead of relying on the elevations at the two ends of each routing element. If there are drop structures or other forms of vertical offsets in the channel reach, the effective channel slope between drops should be used rather than an overall reach slope between endpoints.
- Select the irregular natural channel option in the SWMM conduit cross-section editor to accurately represent actual channel cross-sections rather than selecting generic geometric channels. This provides a more accurate wetted perimeter.
- Use appropriate Manning's n values that are reflective of the nuances in channel geometry and other flow controls along its reaches, namely those recommended in Section 7.2.3 of the *Open Channels* Chapter of the USDCM by following these guidelines:
  - » For lined channels and pipes, increase Manning's n value by 25% over what would normally be used for the design as described in Section 6.2.2 below.
  - » For grass-lined channels, riprap-lined channels, and natural channels, use the higher range of the values for the appropriate type of channel reach as recommended in FHWA Publication HDS-4, Introduction to Highway Hydraulics (Table B.2. Manning's Roughness Coefficients for Various Boundaries).

Whenever HEC-RAS sections are available, use the roughness coefficients for the main channel and overbanks from those studies unless the values obtained from Table B.2 of FHWA HDS-4 are higher.

6.2.1 STEP 1: METHOD OF DISCRETIZATION

Discretization is the procedure used to define catchments and subcatchments. Discretization begins with defining design points and drainage area boundaries and identifying locations and hydrologic connectivity of storm drains, streets, and channels to be represented in the model. For computation and routing of hydrographs, the overall catchment is represented as a network of hydraulic elements (i.e., subcatchments, gutters, pipes, storage nodes, etc.). The hydraulic properties of each element are then characterized by various parameters such as size, slope, and roughness coefficients.

6.2.2 STEP 2: ESTIMATE COEFFICIENTS AND FUNCTIONAL/TABULAR CHARACTERISTICS OF STORAGE AND OUTLET

For hydrologic routing through conveyance elements such as pipes, gutters, and channels, the resistance (Manning's n) coefficients should not necessarily be the same as those used in performing hydraulic design calculations. As a general rule, it was found that increasing the "typical" values of Manning's n by approximately 25% was appropriate when using UDSWMM in the past and should be appropriate for use in SWMM as well. Thus, if a pipe is estimated to have $n = 0.013$ for hydraulic calculations, it is appropriate to use $n = 0.016$ in SWMM.

When modeling the hydrologic routing of natural streams, grass-lined channels, or riprap-lined channels in Colorado, estimate Manning's n for SWMM using Equation 6-9 (Jarrett 1984 and 1985).

$$n = 0.393\ S^{0.38}\ R^{-0.16}$$ **Equation 6-9**

Where:

n = Manning's roughness coefficient

S = friction slope (ft/ft)

R = hydraulic radius (ft)

To estimate the hydraulic radius of a natural, grass-lined, or riprap-lined channel for Equation 6-9, use one-half of the estimated hydrograph peak flow to account for the variable depth of flow during a storm event.

SWMM has several options for defining gutters and street geometry. The user can select the irregular transect option to define the geometry of the gutter and street or use default geometric options if they adequately represent the conveyance element. For storage nodes, the user must define relationships for stage versus surface area versus storage volume using mathematical functions or tables generated by the MHFD-Detention workbook or other hydraulic calculations. For storage unit outlets or downstream outfalls, the user must develop tables or functions to define stage-discharge characteristics. The MHFD-Detention workbook can be used to develop a stage-discharge relationship for multi-stage outlet structures and overflow weirs. Alternatively, the user can define geometries and characteristics for weirs and orifices directly in SWMM and let the program calculate the discharge relationship. The use of the weirs can sometimes be troublesome in SWMM when using dynamic wave routing.

6.2.3 STEP 3: PREPARATION OF DATA FOR COMPUTER INPUT

A major effort in developing a CUHP-SWMM hydrologic routing model is defining the dendritic network of all the runoff and conveyance elements and dividing the catchment into subcatchments. Develop the conveyance elements using a catchment map and subdivision plans, and record drawings of the drainage system. Define pipes with little or no backwater effects, channels, reservoirs, or flow dividers as conveyance elements for computation by SWMM. Once the conveyance element system is set and labeled, use CUHP to generate an output text file that contains runoff hydrographs for all subcatchments. SWMM uses the CUHP output file as an external inflow interface file to assign the hydrograph from CUHP to a target node in SWMM. Refer to the SWMM User's Manual (Rossman 2015) for additional details about input data preparation.

7.0 WATER QUALITY EVENT AND WATER QUALITY PEAK FLOW RATE

MHFD defines the WQE as a design storm with a rainfall depth equal to the 80^{th} percentile runoff-producing storm event for the Denver metropolitan area, 0.6 inches. The WQE can be applied to calculate both a volume (WQCV) and a peak flow (WQPF). The WQPF is the peak flow produced by application of the WQE to a project site. The WQCV, initially developed in 1989 and revised in 1996, is the basis for determining the desired long-term volume capture efficiency (between 80 – 90%) of a stormwater quality facility (Urbonas et al. 1990; Guo & Urbonas 1996). However, the WQCV method does not produce the information needed to design some types of SCMs that require peak flow rates rather than volumes. For example, hydrodynamic separators (HDSs) and other types of manufactured treatment devices (MTDs) require flow-based designs rather than volume-based design approaches (Wilson et al. 2008).

Two methods are available to calculate the WQPF:

1. The recommended method for analyzing the WQE to determine the WQPF is to apply CUHP using a 1-hour point precipitation depth of 0.6 inches and the 2-year, 2-hour temporal storm distribution. This is the preferred method because it generates a hydrograph for the WQE in addition to providing the WQPF.

2. The WQPF can also be calculated using the Rational Method for small sites with time of concentration between 5 and 15 minutes. Calculate the WQPF using the Rational Method as follows:

    a. Determine the weighted imperviousness of the area draining to the design point and calculate (or look up) runoff coefficients using methods in Section 4.6.

    b. Calculate the time of concentration using equations in Section 4.4.

    c. For rainfall intensity, apply Equation 5-1 from the *Rainfall* chapter using 0.6 inches for the 1-hour point rainfall depth with the storm duration equal to the time of concentration.

    d. Calculate the WQPF using Equation 6-1.

See the technical memorandum titled *Investigation of the Water Quality Event and Recommendations for Calculating Water Quality Peak Flows* (Zivkovich and Piza 2024) for additional information on the basis of the WQE and WQPF. See Section 13.4 for an example of WQE and WQPF calculations.

8.0 1% PLUS DISCHARGES

The 1% Plus discharges are calculated to provide upper 84-percent confidence limits for 1% AEP discharges developed as a part of MHFD FHADs. The 1% Plus discharges provide a statistical bound (one standard deviation) on the results of deterministic modeling results and help to communicate the uncertainty of estimates of 1% AEP peak discharges to engineers, local floodplain administrators, and the public. Based on analysis used to develop the 1% Plus method for MHFD, the 1% Plus discharges typically fall between the 200- to 500-year peak flow rates.

FEMA (2019) provides the following guidance on the 1% Plus peak discharges and calculation methodology:[1]

> *The 1-percent-plus flood elevation for a study utilizing rainfall-runoff methodology is defined as a flood elevation derived by using discharges at the upper 84-percent confidence limit for the 1-percent-annual-chance flood. 1-percent + discharges can be estimated using methods outlined in Bulletin 17C appendix 7 (Expected Moments Algorithm), and Chapter 4 of the USACE document Risk-Based Analysis for Flood Damage Reduction Studies (EM 1110-2-1619, USACE 1996). Equations in Appendix 5 are used to determine synthetic logarithmic skew coefficients, standard deviation, and mean. These values paired with equivalent record length of the rainfall-runoff model estimated based on methods shown in Table 4-5 of Chapter 4 of EM 110-2-1619, are used in equations in Appendix 7 of Bulletin 17C to calculate the upper confidence limit discharge. The equivalent record length of the rainfall-runoff model is estimated based on the source data and the amount of detail and calibration that was provided with the model inputs as outlined in Table 4-5 of Chapter 4 of the USACE document Risk-Based Analysis for Flood Damage Reduction Studies.* (EM 1110-2-1619, USACE 1996)

If annual peak streamflow data are available from a stream gage, the preferred approach to calculating 1% Plus discharges is to apply the methods in USGS Bulletin 17C using flow frequency analysis software such as HEC-SSP (USACE 2022). In many cases, however, gage data will not be available for the location of interest, and the procedures developed by MHFD following FEMA guidance provide a way to calculate the 1% Plus discharges based on modeled peak flows for the 2-, 10-, and 100-year return periods.

MHFD developed an Excel spreadsheet, MHFD-1% Plus, to calculate 1% Plus discharges using modeled 2-year, 10-year, and 100-year flows following FEMA's methodology. The spreadsheet calculates synthetic statistics that are used to determine the shape of the flow frequency curve and associated confidence limits. This approach is documented in the MHFD technical memorandum titled *One-Percent-Plus Flow Frequency Analysis* (Earles et al. 2022). The memorandum and software developed by MHFD are available at www.mhfd.org.

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1. The approach used by MHFD for determining 1% Plus discharges uses 2-, 10-, and 100-year peak flow output from CUHP/SWMM. An alternative approach used by FEMA in two-dimensional hydrologic modeling applications is to adjust the rainfall to the upper 84-percent confidence limit and then run hydrologic and hydraulic models using the 1% Plus rainfall to generate the 1% Plus discharges (FEMA 2021).

9.0 PUBLISHED HYDROLOGIC INFORMATION

MHFD and their project partners have prepared hydrologic studies for the majority of watersheds and major drainageways within MHFD. These studies contain hydrologic information including peak flow rates and runoff volumes for a range of return periods at numerous design points within studied watersheds. These studies also contain information regarding catchment and subcatchment boundaries, soil types, imperviousness assumptions, and rainfall. Hydrology studies are available at www.mhfd.org. When published flow values are available from MHFD, use these values for design unless there are compelling reasons to modify the published values.

10.0 STATISTICAL METHODS

10.1 STREAM GAGE ANALYSIS

Statistical analysis of measured streamflow data is another acceptable means of hydrologic analysis in certain situations where long-term flow records are available from reliable stream gages (such as South Platte River, Clear Creek, and Cherry Creek). Statistical analysis should be limited to streams with a long period of flow data (30 years as a recommended minimum) and where there have been no significant land use changes in the tributary area during the period of record (stationarity). Statistical analysis should follow procedures in USGS Bulletin 17C (England et al. 2019), which can be implemented using software such as the USACE's HEC-SSP program (USACE 2022). When conducting statistical analysis, check for the availability of paleoflood data as well as other historic flood data that may not be characterized in the systematic record and available for some streams in the MHFD region through the USGS Colorado Water Science Center Flood Database for Colorado (Kohn et al. 2013). Note that there is no generally accepted and widely used approach to extrapolate calculated flow from a statistical analysis to estimate the flow for expected future watershed development conditions.

10.2 STREAMSTATS

The USGS StreamStats tool (USGS 2019) is a web-based geographic information system application that provides users with access to an assortment of analytical hydrologic assessment tools that are useful for a variety of water resources planning and management purposes and engineering design purposes. StreamStats implements regression equations developed through statistical analysis of stream gages within different hydrologic regions of Colorado (Capesius and Stephens 2009; Kohn et al. 2016). StreamStats has many potential applications in hydrology, ranging from estimating peak flows for a range of AEPs for streams that do not have published flow frequency data to obtaining initial estimates of bankfull and low flow statistics for stream design.

To use StreamStats, the user navigates to the location of interest and selects a point on a stream line using the watershed delineation tool. Once the watershed is delineated, the user can select regression-based scenarios to evaluate and watershed characteristics to calculate and summarize. Regression-based scenarios include peak flow statistics; flood volume statistics; low flow statistics; flow duration statistics; annual, monthly, and bankfull flow statistics; and maximum probable flood statistics. Note that in some locations, only a subset of these options for different hydrologic assessment scenarios may be available. Watershed characteristics include topographic information, as well as information on land cover, rainfall, and soil characteristics. StreamStats extracts selected watershed characteristics, applies regression equations, and creates a summary report.

When interpreting StreamStats analysis, it is important to recognize that the results are regression-based estimates of flow parameters with a quantifiable level of uncertainty. StreamStats output includes information on the average standard error percentages for flow estimates that should be considered by the analyst when interpreting the results. StreamStats also compares regression equation input parameters with the minimum and maximum limits of application intended for the regression equation and flags values that fall outside of the range of applicability. Data generated by StreamStats when input parameters fall outside of the range of applicability of the regression equations should be used with extreme caution.

StreamStats is not applicable on regulated streams and should be applied with caution in urban areas that have complex underground drainage networks. StreamStats should generally only be used for initial hydrologic estimates in the Denver metropolitan area. For final design of stormwater infrastructure, use published flow data from MHFD master planning studies or the other hydrologic methods described in this chapter.

11.0 SOFTWARE

MHFD has developed a collection of calculation workbooks and software to help with the design procedures in the USDCM. Instructional videos are available for many of these tools at www.mhfd.org.

- CUHP, which is periodically updated, is a macro-enabled Excel workbook that calculates runoff hydrographs based on catchment characteristics and other inputs described in Section 5. CUHP generates output hydrographs that can be input to EPA SWMM nodes for hydrologic routing and combination of hydrographs. The latest release of EPA SWMM is available for download from EPA's SWMM website.
- MHFD-Detention is an Excel-based workbook used to size full spectrum detention (FSD) facilities. MHFD-Detention calculates the WQCV, EURV, and 100-year storage volumes for detention facilities. The spreadsheet performs hydrologic calculations using CUHP to generate runoff hydrographs for inflows to the FSD facility and uses the Modified Puls reservoir routing method to size the facility, comparing calculated release rates to predevelopment discharges for the 2-, 5-, 10-, 25-, 50-, 100-, and 500-year events. MHFD-Detention allows analysis of a variety of SCMs, including extended detention basins, bioretention, sand filters, and other stormwater facilities that may or may not be full spectrum. The *Storage* chapter includes an example of applying the MHFD-Detention workbook.
- MHFD-Rational is an Excel workbook that performs runoff calculations using the Rational Method. Inputs include subcatchment area, runoff coefficient, 1-hour point precipitation depth from NOAA Atlas 14, and flow reach characteristics (length, slope, and type of ground surface). The workbook then calculates the time of concentration, rainfall intensity, and peak flow rate for all subcatchments.
- MHFD-Inlet is an Excel workbook that uses the Rational Method to calculate runoff to storm inlets and evaluates the capture efficiency of various inlet types. Inputs include the Rational Method inputs described above, street geometry parameters (longitudinal slope, gutter depth, local depression, etc.), and inlet properties (inlet type, grate and curb opening dimensions, and clogging factors).
- SCM Design (formerly UD-BMP) is an Excel workbook used for sizing SCMs included in Volume 3. SCM Design performs WQCV calculations for storage-based SCMs and calculates the WQPF for flow through SCMs like hydrodynamic separators. Several components of SCMs can be sized in the workbook, including inflow features (level spreaders, curb openings, and forebays), minimum filter areas, basin geometry, and outlet orifice plate openings. The RPA worksheet performs runoff reduction calculations for grass buffers using regression equations derived from extensive CUHP analysis to quantify runoff volume reduction when routing impervious areas across RPAs.

Users of these software packages should check for updates on a regular basis. Updates and enhancements are constantly under development to incorporate new and modified criteria and improve functionality.

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13.0 EXAMPLES

13.1 RATIONAL METHOD EXAMPLE 1

Find the 100-year peak flow rate for a 60-acre catchment in an undeveloped grassland area located in Brighton. The upper 400 feet of the catchment is sloped at 2%, the lower 1,500 feet is grassed waterway that is sloped at 1%. The area has HSG C soils.

From NOAA Atlas 14, the 1-hour point precipitation value is 2.55 inches. The imperviousness is 5% (or 0.05) based on Table 6-3 and using the category “Undisturbed or Decompacted Soils (Native Grasses and Open Space Areas).”

Determine C_5 from Table 6-5:

$C_5 = 0.815I + 0.035$

$C_5 = 0.815(0.05) + 0.035$

$= 0.08$

Determine t_i from Equation 6-3:

$$t_i = \frac{0.395(1.1 - C_5)\sqrt{L_i}}{S_o^{\;0.33}}$$

$$t_i = \frac{0.395(1.1 - 0.08)\sqrt{400}}{(0.02)^{0.33}}$$

$t_i = 29.3$ minutes

Find t_t from Equation 6-4:

$$t_t = \frac{L_t}{60V_t} = \frac{L_t}{60K\sqrt{S_o}}$$

From Table 6-4, $K = 15$ (grassed waterway); $S_o = 0.01$ and $L = 1{,}500$ feet from problem statement

$$t_t = \frac{1500}{60(15\sqrt{0.01})}$$

$t_t = 16.7$ minutes

From Equation 6-2:

$t_c = t_i + t_t$

$t_c = 29.3 + 16.7$

$t_c = 46$ minutes

Note: The first design point time of concentration, Equation 6-5, does not apply for this example because the tributary area is undeveloped and less than 20% impervious.

Determine C_{100} from Table 6-5:

$C_{100} = 0.409I + 0.484$

$C_{100} = 0.409(0.05) + 0.484$

$C_{100} = 0.50$

Determine rainfall intensity, I, from Equation 5-1 (from the *Rainfall* chapter):

$$I = \frac{28.5P_1}{(10+t_c)^{0.786}}$$

$$I = \frac{28.5(2.55)}{(10+46)^{0.786}}$$

I = 3.07 in/hr

Determine Q from Equation 6-1:

Q= (0.50)(3.07)(60)

Q = 92 cfs

Alternately, use the MHFD-Rational Excel workbook to calculate the peak flow rate.

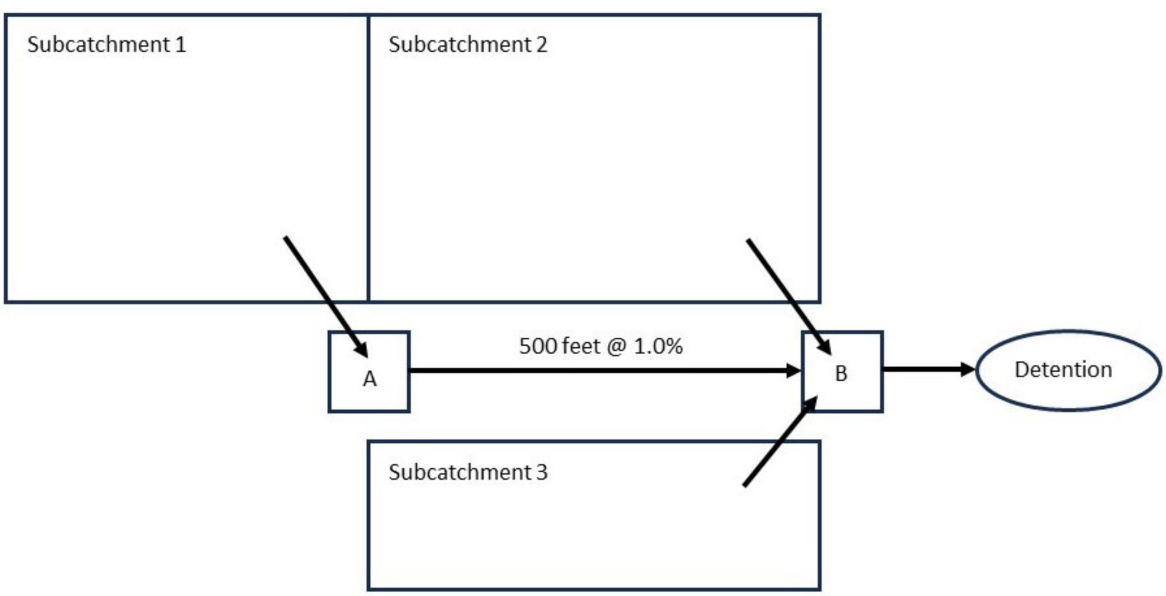
13.2 RATIONAL METHOD EXAMPLE 2

A project located in the City of Denver is represented by three subcatchments. The drainage system collects Subcatchment 1 at Point A, and Subcatchments 2 and 3 at Point B, and then drains into a detention system. Determine the 10-year peak discharge at Point B using the watershed parameters summarized in the table below.

RATIONAL METHOD EXAMPLE WATERSHED PARAMETERS

SUBCATCHMENT	DRAINAGE AREA A (ACRES)	RUNOFF COEFFICIENT C	TIME OF CONCENTRATION t_c
1	2.00	0.55	15
2	5.00	0.65	22
3	1.50	0.81	12

As shown in the figure below, there are three flow paths to reach Point B. Their flow times are:



RATIONAL METHOD EXAMPLE LAYOUT

From Subcatchment 1: The flow time includes the time of concentration of Subcatchment 1, and the flow time from Point A to Point B through the street. The flow time from Subcatchment 1 to Point B is the sum of the time of concentration of Subcatchment 1 and the flow time through the 500-foot gutter:

$$t_c = t_{c1} + t_{tA\ to\ B}$$

$$t_{c1} = 15 \text{ minutes}$$

$$t_t = \frac{L_t}{60V_t} = \frac{L_t}{60K\sqrt{S_o}}$$

$$t_c = 15 + \frac{500}{60(20)\sqrt{0.01}} = 19.2 \text{ minutes}$$

From problem statement for Subcatchment 2: $t_{c2} = 22$ minutes

From problem statement for Subcatchment 3: $t_{c3} = 12$ minutes

At Point B, the design rainfall duration $t_d = \max(t_{c1}, t_{c2}, t_{c3}) = 22$ minutes

The 10-year design rainfall intensity for Denver is (from Equation 5-1 in the *Rainfall* chapter, using $P_1 = 1.33$ inches):

$$I = \frac{28.5P_1}{(10 + t_c)^{0.786}}$$

$$I = \frac{28.5(1.33)}{(10 + 22)^{0.786}} = 2.49 \text{ in/hr}$$

Area-weighted runoff coefficient, $C_{composite}$ calculation shown below for all of the areas that drain to Point B:

$$C_{composite} = \frac{(C_1A_1 + C_2A_2 + C_3A_3)}{(A_1 + A_2 + A_3)}$$

$$C_{composite} = \frac{((0.55)(2) + (0.65)(5) + (0.81)(1.5))}{(2 + 5 + 1.5)} = 0.65$$

The 10-year peak discharge is:

$$Q = CIA = (0.65)(2.49)(8.5) = 13.8 \text{ cfs}$$

13.3 EXAMPLE OF CUHP AND SWMM FILE ORGANIZATION FOR APPLYING DEPTH REDUCTION FACTORS

For a total watershed area of 14 mi^2, a CUHP/SWMM model is set up to calculate 2-, 5-, and 10-year events with depth reduction factor (DRF) corrections applied for areas of 0 – 2 mi^2, 2 – 5 mi^2, 5 – 10 mi^2, and 10 – 14 mi^2 (four area correction categories), using the mid-range of the each area category to select a DRF in accordance with the *Rainfall* chapter. For this example there is one SWMM input file because the geometry of the routing remains the same.

In additional to the single runs of 25-, 50-, 100-, and 500-year events (because DRFs do not apply to these return periods until the watershed area reached 15 mi^2), twelve (12) runs of CUHP hydrograph files are routed through the single SWMM input file to generate 12 SWMM output/report files for the 2-, 5-, and 10-year events (3 return events x 4 area corrections). Although the same SWMM network is used for the different CUHP runs, it is important to name the SWMM output file in a way that reflects the area correction used in the CUHP run. The table below illustrates the file organization and naming conventions that can be applied to the CUHP and SWMM files.

EXAMPLE CUHP/SWMM FILE ORGANIZATION FOR ANALYSIS OF MULTIPLE RUNS INCLUDING DEPTH REDUCTION FACTORS

SCENARIO	CUHP			TIME OF CONCENTRATION t_c
	RETURN PERIOD (YEAR)	CORRECTION AREA (MI²)	CUHP INTERFACE FILE	SWMM REPORT FILE
1	2	0	1_Ex_2yr_0mi^2_CUHP.txt	1_Ex_2yr_0mi^2_SWMM.rpt
2	5	0	2_Ex_5yr_0mi^2_CUHP.txt	2_Ex_5yr_0mi^2_SWMM.rpt
3	10	0	3_Ex_10yr_0mi^2_CUHP.txt	3_Ex_10yr_0mi^2_SWMM.rpt
4	2	3.5	4_Ex_2yr_3.5mi^2_CUHP.txt	4_Ex_2yr_3.5mi^2_SWMM.rpt
5	5	3.5	5_Ex_5yr_3.5mi^2_CUHP.txt	5_Ex_5yr_3.5mi^2_SWMM.rpt
6	10	3.5	6_Ex_10yr_3.5mi^2_CUHP.txt	6_Ex_10yr_3.5mi^2_SWMM.rpt
7	2	7.5	7_Ex_2yr_7.5mi^2_CUHP.txt	7_Ex_2yr_7.5mi^2_SWMM.rpt
8	5	7.5	8_Ex_5yr_7.5mi^2_CUHP.txt	8_Ex_5yr_7.5mi^2_SWMM.rpt
9	10	7.5	9_Ex_10yr_7.5mi^2_CUHP.txt	9_Ex_10yr_7.5mi^2_SWMM.rpt
10	2	12	10_Ex_2yr_12mi^2_CUHP.txt	10_Ex_2yr_12mi^2_SWMM.rpt
11	5	12	11_Ex_5yr_12mi^2_CUHP.txt	11_Ex_5yr_12mi^2_SWMM.rpt
12	10	12	12_Ex_10yr_12mi^2_CUHP.txt	12_Ex_10yr_12mi^2_SWMM.rpt
13	25	0	13_Ex_25yr_0mi^2_CUHP.txt	13_Ex_25yr_0mi^2_SWMM.rpt
14	50	0	14_Ex_50yr_0mi^2_CUHP.txt	14_Ex_50yr_0mi^2_SWMM.rpt
15	100	0	15_Ex_100yr_0mi^2_CUHP.txt	15_Ex_100yr_0mi^2_SWMM.rpt
16	500	0	16_Ex_500yr_0mi^2_CUHP.txt	16_Ex_500yr_0mi^2_SWMM.rpt

For design points with tributary areas of less than 2 mi^2, peak flows are pulled from the SWMM reports generated for Scenarios 1, 2, and 3; for tributary areas of 2 – 5 mi^2, report flows from Scenarios 4 through 6; for tributary areas of 5 – 10 mi^2, report flows from Scenarios 7 through 9; and for tributary area greater than 10 mi^2, report flows from Scenarios 10 through 12.

Reporting peak flows in a watershed with multiple area corrections requires the total tributary area to each design point to correlate the appropriate SWMM report results. A design point with 1.2 mi^2 reports flows from SWMM runs for Scenarios 1 through 3 and 13 through 16. Whereas, a design point with 7 mi^2 reports flows from SWMM runs for Scenarios 7 through 9 and 13 through 16.

13.4 WATER QUALITY EVENT AND WATER QUALITY PEAK FLOW EXAMPLE

Find the WQPF for a 5-acre catchment with imperviousness of 80% and the following characteristics:

- Length to Centroid = 0.2 mile
- Length = 0.33 mile
- Slope = 0.02 ft/ft
- Maximum Pervious Depression Storage = 0.35 inches
- Maximum Impervious Depression Storage = 0.1 inches
- Soil Type for Horton's Infiltration Parameters: HSG C
- 1-minute time step between computations

Enter catchment parameters into CUHP:

CUHP SUBCATCHMENTS

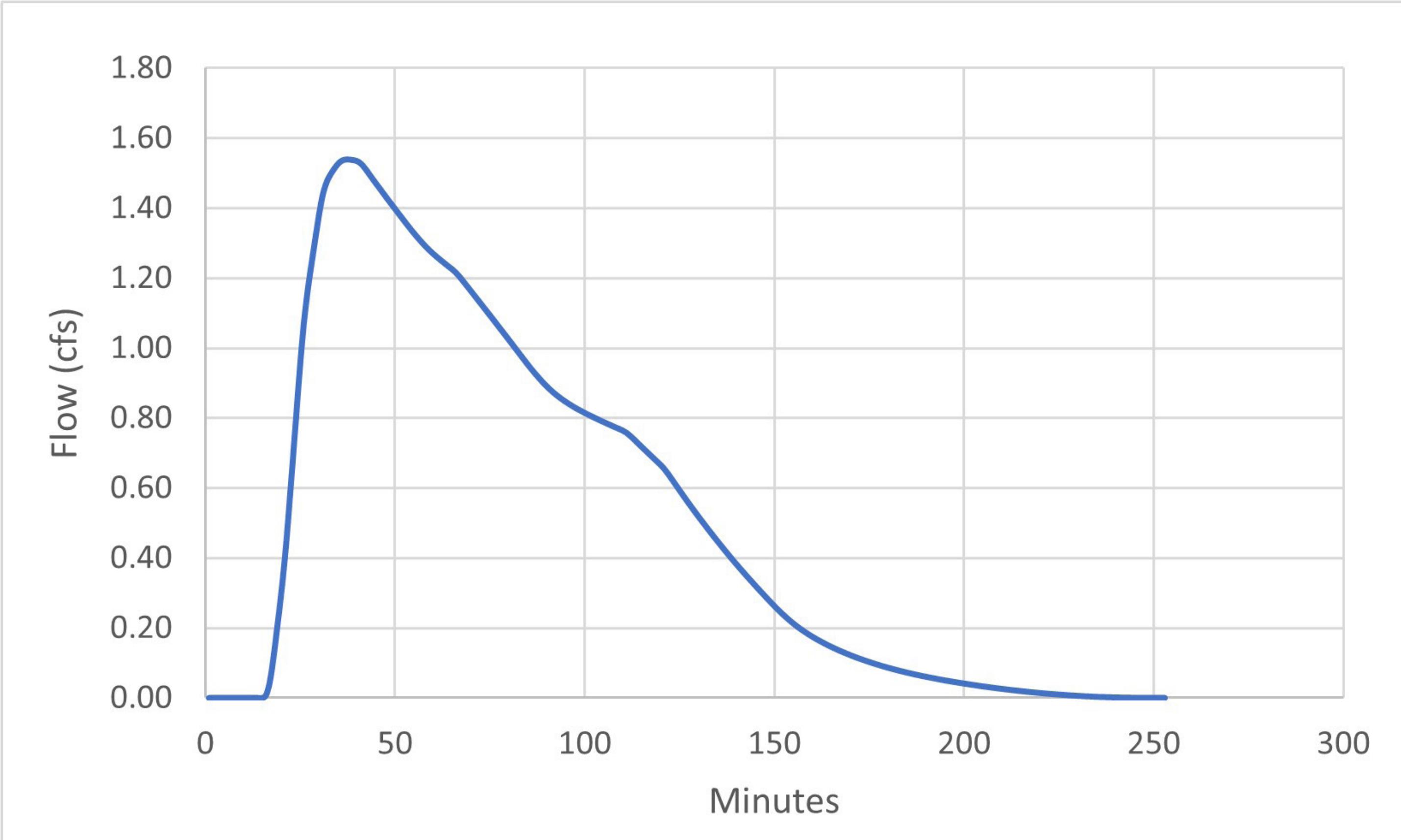
Columns with this color heading are for required user-input
Columns with this color heading are for optional override values
Columns with this color heading are for program-calculated values

								Maximum Depression Storage (Watershed inches)		Horton's Infiltration Parameters			DCIA
Subcatchment Name	EPA SWMM Target Node	Raingage	Area (mi^2)	Length to Centroid (mi)	Length (mi)	Slope (ft/ft)	Percent Imperviousness	Pervious	Impervious	Initial Rate (in/hr)	Decay Coefficient (1/seconds)	Final Rate (in/hr)	Level 0, 1, or 2
Subcatchment 1		WQE	0.0078125	0.2	0.33	0.02	80	0.35	0.1	3	0.0018	0.5	0

Set up a rain gage in CUHP using the 2-year hyetograph and a 1-hour point precipitation depth of 0.6 inches:

Comment	Water Quality Event	
1Hr Depth	0.6	NOAA Atlas 14 Point Precipitation Frequency Estimates: CO (Note: Use 60-minute recurrence interval depth)
Return Period	2	Years
Time	Depth	CurveValue
0:05	0.012	0.02
0:10	0.024	0.04
0:15	0.05	0.084
0:20	0.096	0.16
0:25	0.15	0.25
0:30	0.084	0.14
0:35	0.038	0.063
0:40	0.03	0.05
0:45	0.018	0.03
0:50	0.018	0.03
0:55	0.018	0.03
1:00	0.018	0.03
1:05	0.018	0.03
1:10	0.012	0.02
1:15	0.012	0.02
1:20	0.012	0.02
1:25	0.012	0.02
1:30	0.012	0.02
1:35	0.012	0.02
1:40	0.012	0.02
1:45	0.012	0.02
1:50	0.012	0.02
1:55	0.006	0.01
2:00	0.006	0.01
2:05	0	

Run CUHP and plot the storm hydrograph to check for reasonableness:



Determine the peak flow rate of the hydrograph from the tabular output in the CUHP workbook:

Summary of Unit Hydrograph Parameters Used By Program and Calculated Results (Version 2.0.1)

	Unit Hydrograph Parameters and Results									Excess Precip.		Storm Hydrograph			
Catchment Name/ID	CT	Cp	W50 (min.)	W50 Before Peak	W75 (min.)	W75 Before Peak	Time to Peak (min.)	Peak (cfs)	Volume (c.f)	Excess (inches)	Excess (c.f.)	Time to Peak (min.)	Peak Flow (cfs)	Total Volume (c.f.)	Runoff per Unit Area (cfs/acre)
Subcatchment 1	0.078	0.072	35.2	2.24	18.3	1.58	3.7	6.7	18,150	0.43	7,745	38.0	1.5	7,745	0.31

The WQPF for this watershed is 1.5 cfs.