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RESOLUTION NO. 15-042

BOARD OF COUNTY COMMISSIONERS
COUNTY OF EL PASO, STATE OF COLORADO

RESOLUTION FOR ADOPTION OF PORTIONS OF THE CITY OF COLORADO SPRINGS DRAINAGE CRITERIA MANUAL VOLUME 1 DATED MAY 2014

WHEREAS, pursuant to Colorado Revised Statues, Article 28 of Title 30 and Article 20 of Title 29, the Board of County Commissioners of El Paso County, Colorado ("Board') is authorized to enact regulations in the unincorporated areas of the County through planning and building codes and other land use regulations, regulatory documents or references as necessary; and

WHEREAS, the current City/County Drainage Criteria Manual was adopted by the Board in Resolution 87-178A and amended in Resolutions 88-58, 91-334, 95-81, 01-384; and

WHEREAS, the practice of stormwater management has since advanced significantly, and the current manual does not reflect the best available practices for effective stormwater management; and

WHEREAS, the City of Colorado Springs has developed and adopted the City of Colorado Springs Drainage Criteria Manual Volume 1 (DCMV1) dated May 2014; and

WHEREAS, there have been several efforts in the last ten to fifteen years to address stormwater management comprehensively within the Pikes Peak Region and the DCMV1 is being proposed to be applied throughout the Pikes Peak Region with adoption by El Paso County and other local jurisdictions; and

WHEREAS, the City of Colorado Springs had an extensive, lengthy public participatory effort which engaged interested citizens, the development community, engineering professionals, and local, state and federal agencies and which received positive recommendations for adoption of DCMV1 from City/County Drainage Board, Housing and Building Association of Colorado Springs and approval from Colorado Department of Public Health and Environment; and

WHEREAS, El Paso County faces different challenges than the City of Colorado Springs because of its wide range in elevation, soil conditions, urban and rural development, differences in land use approval processes, differences in local policies, ordinances, resolutions and regulations, and differences in statutory authority, and as such require further refinement to DCMV1 before full adoption and implementation by El Paso County; and

WHEREAS, staff is recommending that the Board approve an initial adoption of Chapter 6 (Hydrology) and Section 3.2.1 of Chapter 13 (Full Spectrum Detention) of DCMV1 and establish a review period of approximately one year to revise the remainder of the manual for use specific to El Paso County, allowing for proposed refinements to the City of Colorado Springs DCMV1 which shall involve a subcommittee composed of County staff, private professional

consultants and other interested stakeholders to result in final revisions to be recommended to the Board; and

WHEREAS, during the review period, should any inconsistencies or conflicts be found between the aforementioned portions of DCMV1 and other adopted ordinances, resolutions or regulations of the County as well as the existing City/County Drainage Criteria Manual, Engineering Criteria Manual and Land Development Code, the provision that is more protective of public health, safety, and welfare shall control; and

WHEREAS, the purpose of the initial adoption of these portions of DCMV1 is to allow the use of new stormwater management techniques and best practices in conjunction with the current adopted City/County Drainage Criteria Manual until refinements specific to El Paso County are addressed while still ensuring the protection of the health, welfare, and safety of the citizens of El Paso County; and

WHEREAS, the responsible authority for setting these standards and providing revision to these standards is the Board of County Commissioners, who may delegate authority to apply and interpret these standards to the County Engineer or other designated staff; and

WHEREAS, the responsible authority for application of these standards for public works, capital projects, and other non-development related engineering and construction works is the Public Services Director or delegated staff and the responsible authority for application of these standards regarding all development related applications is the Development Services Director or delegated staff; and

WHEREAS, a public hearing was held by the El Paso County Planning Commission on December 2, 2014, upon which date the Planning Commission did by formal resolution recommend approval of these portions of DCMV1 while further refining the manual for use specific to El Paso County; and

NOW, THEREFORE, BE IT RESOLVED that the Board of County Commissioners of El Paso County, Colorado, hereby adopts Chapter 6 (Hydrology) and Section 3.2.1 of Chapter 13 (Full Spectrum Detention) of the City of Colorado Springs DCMV1 dated May 2014 effective January 31, 2015 which provisions shall be applied to all projects except for those that have submitted a completed development application prior to the effective date; and

BE IT FURTHER RESOLVED that the Board of County Commissioners of El Paso County, Colorado, hereby directs the County Engineer and appropriate staff, including but not limited to the Public Services Department and Development Services Department, to conduct a coordinated effort to further refine the DCMV1 manual for use specific to El Paso County for adoption by the El Paso County Board of Commissioners within the next year, which will include receiving recommendations by the appropriate El Paso County advisory committees that report to the El Paso County Board of Commissioners; and

BE IT FURTHER RESOLVED that staff shall continue coordination with regional partners concerning associated "spin-off" topics identified during the City's Stormwater

Management Assessment Project, and that subsequent to the adoption of DCMV1 El Paso County shall also develop a similar approach to update its Drainage Criteria Manual Volume 2 (DCMV2), which is currently included as Appendix I in the El Paso County Engineering Criteria Manual; and

BE IT FURTHER RESOLVED that Dennis Hisey, duly elected, qualified member and Chair of the Board of County Commissioners, or, Amy Lathen, duly elected, qualified member and Vice Chair of the Board of County Commissioners, be and is hereby authorized on behalf of the Board to execute any and all documents necessary to carry out the intent of the Board as described herein.

DONE THIS 27th day of January, 2015, at Colorado Springs, Colorado.

ATTEST:

By: _____
Chuck Broerman
County Clerk and Recorder

BOARD OF COUNTY COMMISSIONERS
EL PASO COUNTY, COLORADO

By: _____
Dennis Hisey, Chair

Drainage Criteria Manual
Volume 1

May 2014

CITY OF COLORADO SPRINGS
30 S. Nevada Ave.
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Drainage Criteria Manual
Volume 1

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1.0 Introduction

This chapter describes the basis for determining design flows and volumes for the planning and design of stormwater management facilities using various hydrologic methods and summarizes these methods. The development of appropriate hydrology for a project is often one of the first steps in the planning or design process. Accurately estimating design flows and volumes is critical to the proper functioning of facilities. Depending on the size of the drainage basin and the type of project different hydrologic methods may be appropriate. In many cases flow estimates may be available from previous studies and these should be considered and evaluated, especially when new hydrologic analysis results in significant changes to previous estimates. A general discussion of the overall process of developing plans and analyzing drainage systems can be found in the UDFCD Storm Drainage Criteria Manual (UDFCD Manual) Volume 1, Planning chapter.[1]

Design flows for stormwater facilities are primarily based on rainfall events, and a range of possible storm events must be considered. Other sources of runoff such as snowmelt and nuisance flows can affect a project design and must also be identified. Anticipated changes to basin conditions, especially due to development, must also be fully considered for facilities to function as intended.

Rainstorms in the Fountain Creek watershed and El Paso County have been identified as being of two primary types with significantly different spatial and temporal characteristics. Short-duration thunderstorms (cloud bursts) occur frequently and can be very intense (produce significant rainfall depths rapidly), but are localized and not expected to produce widespread flooding. Longer-duration storms occur less frequently, may be part of a much larger storm system influenced by regional conditions, and are generally less intense; however, they can affect much larger areas and produce widespread flooding. The type of storm that must be evaluated depends on the size of the drainage basin, the type of project, and other site- and basin-specific factors. In some cases, both types of storms may need to be evaluated. Rainstorms of any significance typically occur between the months of May and September.

Urbanization can have a significant impact on runoff by increasing peak flow rates, runoff volumes, and the frequency of runoff. This increase in runoff can lead to severe stream erosion, habitat degradation, and increased pollutant loading. However, with proper planning, increased runoff can be managed to create or supplement existing wetland areas or riparian habitats, which may provide significant benefits to the watershed. The increase in runoff from development is especially pronounced when drainage systems are designed to quickly and "efficiently" convey runoff from paved areas and roofs directly into inlets and storm sewers, discharging eventually into drainageways that are typically designed to convey flows at maximum acceptable velocities. Whether for one site or for a whole watershed, this increase in runoff and acceleration of flood peaks can be estimated by the hydrologic methods discussed herein.

In addition to increased runoff, the reduction of available sediment due to urbanization also has the potential to destabilize downstream channels. When the natural sources of sediment are eliminated by paving, building structures or stabilized channels, runoff will tend to replace the natural sediment supply by satisfying its capacity for sediment transport with new sources. Therefore, even an effective reduction in developed runoff to levels approximating historic rates will probably not eliminate the need for the stabilization of downstream systems.

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[1] Some of the terminology used in the UDFCD Manual varies from the terminology in Colorado Springs (e.g., Drainage Basin Planning Study[DBPS] in Colorado Springs versus Major Drainageway Planning Studies/Outfall Systems Planning Studies in the UDFCD Manual). Additionally, the UDFCD Manual does not address requirements for site-level drainage report and design preparation because these requirements are usually explicitly defined by the municipalities that make up UDFCD.

As discussed in Volume 2 of this Manual, which addresses stormwater quality, effective stormwater management seeks to disconnect impervious surfaces, decrease flow velocities, and convey runoff over vegetated ground surfaces, leading to filtering, infiltration, and attenuation of flows. These principles can also be reflected in the hydrologic variables discussed in this chapter, yielding longer times of concentration and reduced runoff peaks and volumes.

1.1 Design Flows

A broad range of events pass through stormwater facilities and natural drainageways. These range from those producing little or no runoff prior to development to extensive and extreme storm events that produce life threatening and destructive floods. To effectively and efficiently analyze even a small percentage of all possible events is time and cost prohibitive. Therefore, to efficiently plan and design stormwater facilities, "design flows" have been established to represent events that are typical or representative of the range of runoff events that can occur. In most cases, projects can be adequately designed using estimates of these representative flows. Depending on the type of project, design flows may include "baseflows," "low flows," "minor flows," "major flows" and "flood flows." A description of each of these types of flows is provided below and methods for estimating these design flows are described later in this chapter.

1. **Baseflows**: Baseflow estimates (sometimes referred to as "trickle flows") are used to account for flows that may not be directly related to storm events but may be created by groundwater recharge of streams, wastewater return flows, excess irrigation, water system losses, and other urban water uses. Baseflows are the flows that can be observed in streams and engineered drainageways during dry weather. Methods for directly identifying the source and quantity of these flows are not generally available for drainage basins. Where available, such as on Monument and Fountain Creeks, baseflows may be estimated from gage data and possibly from projections of future return flows. The *Fountain Creek Watershed Hydrology Report* (USACE 2006) provides some baseflow data. Channel improvements must account for these flows to address erosion potential in the lower portion of channel sections. The presence of these flows in historically dry basins can also interfere with the growth of certain types of vegetation. However, these same flows can provide water to sustain vegetation along low-flow channel banks or in channels with wetland bottoms where vegetation was not previously supported.

2. **Low Flows**: Low flows are used primarily for open channel design and are defined as those flows resulting from relatively frequent storm events that are contained within a well-defined or main channel portion of the floodplain (sometimes these are referred to as "bankfull flows" or "channel forming flows" for natural streams). Flows greater than the low-flow event begin to flow beyond the main channel into the overbank or floodplain portion of natural channels.

   In natural channels, the capacity of the low-flow portion of the channel can be represented by the "bankfull" flow. It is difficult to relate this flow to a particular return period since it represents the combined influence of a wide variety of storm events occurring over a long period of time in the upstream drainage basin. However, it is generally accepted that the bankfull discharge has a return period that is in the 1-year to 2-year range, but this value can change significantly in different hydrologic regions, especially when there is urbanization in a basin. "Low flows" should not be confused with "minor flows" which are associated with a specific recurrence interval, as described below, and are generally greater than "low flows" for natural channels.

3. **Minor Flow**: Minor flows are defined as those flows resulting from relatively frequent storm events that are contained within a portion of the conveyance system such as gutters and storm sewers and are typically defined by a specific return period. Flows greater than the minor flow

event begin to flow beyond the normal acceptable limits, like street gutters, onto adjacent improvements, like sidewalks, and begin to interfere with human activity, such as traffic and pedestrian access. For the purposes of this Manual, the minor flow is defined by the 5-year storm runoff event.

4. **Major Flow**: Major flows must be conveyed to avoid safety hazards, undue interference with human activity, damage to adjacent structures and damage to conveyance systems. The 100-year runoff event has been identified as the major flow that must be safely conveyed according to this Manual. This design flow is typically used to determine maximum street capacities and to size certain facilities such as culverts.

5. **Flood Flows**: In this Manual the term "flood flows" is used to refer to any flows that exceed the low-flow channel, whether natural or engineered. Flood flows must be conveyed to avoid safety hazards, damage to adjacent structures and damage to conveyance systems. Flood flows are typically used to design open channels, size detention ponds and to delineate floodplains. Flooding is often associated with rather extreme events, but is actually defined by any event that causes flows to spill from the low-flow channel onto the overbank or floodplain area of a channel. The 100-year runoff event has been identified as the major flood event that defines the regulatory floodplain according to this Manual. However, flood studies typically include evaluation of other events such as the 10-, 25-, 50- and 500-year events. In some cases, it may be necessary to evaluate lesser flows, such as the 2-year or 5-year flow, to consider critical hydraulic conditions. In some situations, it may be appropriate to address more severe flows, such as the 500-year flow. For instance, drop structures may be largely submerged during the 100-year event, with critical hydraulic conditions occurring during lesser floods. Also, where critical infrastructure, such as hospitals, power plants or emergency response facilities may be at risk, it may State and local floodplain regulations may require evaluation of less frequent events such as the 500-year flood.

Prudent management of upstream land uses and the implementation of runoff reducing practices such as "Low Impact Development" and/or "Full-Spectrum Detention" have the potential to reduce the volume and rate of runoff for design flows received by downstream systems. These effects generally are most significant for frequently occurring events. How design flows are affected by upstream basin conditions (under future "build out" conditions) must be fully considered.

1.2 Sources of Design Flows and Types of Hydrologic Analyses

Estimates of runoff are required for a variety of purposes in stormwater management analyses. In many cases, previously completed analyses may be available for the project area. Given adequate periods of record and relatively static basin conditions, gage data may be the most reliable source of flow estimates. Designers should first investigate the existence of relevant studies that can be applied to the project at hand. However, it is often necessary to complete new analyses that more accurately represent project conditions and provide estimates where they are needed to complete the project. Whenever new analyses are needed, they shall be completed based on the methodologies described in this Manual.

To provide plans and designs that are appropriate for current and future conditions, hydrologic analyses must include various scenarios. Scenarios will typically include multiple runoff events, changes in land uses and alternative system plans such as for transportation. Each scenario must be identified and properly described so that the drainage system plan and possible alternatives can be adequately evaluated.

1.2.1 Published Hydrologic Information

Drainage master plans have been prepared for many of the Colorado Springs drainage basins. These reports may contain information regarding peak flow and runoff volume from the 2-year through 100-year storm events at numerous design points within the study watersheds. These studies contain information about watershed and sub-watershed boundaries, soil types, percent imperviousness, and rainfall. If there are published flow rate values available, these values shall be used for design unless they are considered to be inaccurate or unreliable due to physical changes in the drainage basin or in criteria. The need for additional evaluation and the use of other values shall be approved in writing in advance of any related planning or design work.

Published hydrologic information for major drainageways can also be found in Federal Emergency Management Agency (FEMA) Flood Insurance Studies (FIS). For all FEMA-related projects, the FEMA hydrologic data shall be consulted. Flow rates published in FEMA FIS studies typically represent existing conditions at the time the study was completed and generally do not incorporate any future development. The analysis and design of stormwater facilities must be on future development flow rates; therefore, FEMA flow rates shall not be used without written approval.

1.2.2 Statistical Methods

In some situations, statistical analysis of measured stream flow data provides an acceptable means of determining design flows. Statistical analyses for larger, less-frequent storm events are to be limited to drainageways with a long period of reliable flow data that had no significant changes occur in land uses within the tributary watershed during the flow record. A minimum period of record of 30 years is recommended for statistical analysis of events up to and including the 100-year event; however, when performing statistical analysis for frequently occurring events, such as a 2- or 5-year event, a shorter period of record may be acceptable. Statistical methods may be useful in calibrating a hydrologic model for existing development conditions, but these methods are not suited for estimating the flow for expected future watershed development conditions.

Gage data available for Fountain Creek have been analyzed in a report prepared for FEMA titled “Flood Hydrology for Fountain Creek” (Michael Baker Jr. 2010). The results of this study shall apply for analyses of flood flows on Fountain Creek.

Statistical analyses of gage data should be completed using the Log-Pearson Type III analysis as performed by programs such as the U.S. Geological Survey (USGS) PeakFQ analysis tool. The gage identification and location should be indicated on all calculation sheets and model output.

Statistical methods can be used where a long-term record of flows is available and where the upstream basin characteristics are not expected to change significantly through urbanization or through the construction of flow altering structures or inflows. Limitations of these methods are:

- It is not uncommon for stream gages to be relocated over time. If the relocation is relatively near the former gaging station, an adjustment based on ratios of contributing drainage areas can be used to adjust gage data for a more direct comparison.

- Stream gage data should not be used for statistical analysis unless it has gone through a Quality Assurance/Quality Control (QA/QC) check to verify that the data are reasonable. QA/QC checks to verify that data are reasonable to use for statistical analysis and to assure that the reported data are being interpreted correctly typically include the following:

- Contact the agency that operates and maintains the gage to learn how flows are measured, how different rating curves may be applied to low and high flows, how rating curves have "shifted" over time and other information on the quality of the data from the gage operator. Ask the operators of the gage for the estimated accuracy of flow measurements during low flow and peak flow conditions.

- Plot data and look for unusual "jumps" or "drops" in the data, which could potentially reflect changes in rating curves used to determine flow from stage data. It is common for one rating curve to be used for the low flow range and a separate curve used for the high flow range. Identifying when flow measurements switch from the low flow range to the high flow range can be important for understanding data.

- Evaluate the period of record and the types and amount of data reported within the period of record. For example, if the user is interested in annual peak flow values for a stream, but the period of record has many unreported values during spring runoff months, this would be an indication that the peak flow estimates may not be reliable due to spotty data.

- Evaluate the statistical distribution of the data and perform statistical tests for outliers if any data points appear to be extreme. While most rainfall and streamflow data are log-normally distributed, it is important to verify this assumption, either by plotting data, looking at model output statistics or calculating simple statistical parameters on log-transformed data using a normality test.

- Availability of peak flow data can be a challenge for statistical hydrology. Do not run Log-Pearson Type III analysis on average daily flows to determine flood flows. Annual peak flows are needed for this purpose but unfortunately are not available at all stream gages.

1.2.3 Rainfall/Runoff Methods

It is often necessary to estimate runoff for a project when no previous estimates have been provided. The most common method for making these estimates is by converting rainfall (using intensity, depth and temporal and spatial characteristics) to runoff by representing basin characteristics that affect the volume and rate of runoff expected. There are numerous methods that can be applied, but only a few have been adopted for this Manual. The method selected will depend on the purpose of the analysis and the size of the drainage basin.

Rainfall/runoff methods are based on approximations of parameters that can vary significantly from basin to basin or between climatic zones. Whenever flow data are available they should be compared to the calculated estimates of flow to confirm the reasonableness of the estimates.

1.2.4 Paleo-flood Analysis

By assessing geologic conditions and remnants of flood flows within a stream valley evidence of past floods can be observed and evaluated to determine their limits and approximate timing. The application of these methods requires special expertise, but may provide additional insight into the flood potential within a project reach, especially for large flood events that may not be captured in the modern stream gage record. These methods are documented in USGS publications, but must be approved prior to their application.

1.3 Data Requirements

Prior to commencing a hydrologic analysis the designer must research and collect the necessary data to provide inputs for the hydrologic method to be used. These data may be available from existing sources or may need to be created for the project at hand. These data will typically include: topographic mapping, existing and future land use conditions for each scenario to be evaluated, an inventory of existing and proposed structures (in waterways and other structures associated with development) within the study area, soil types, ground cover types, groundwater conditions, site location information (horizontally and vertically), previous studies, and any other documents that can provide needed background information. It is the responsibility of the designer to identify and collect the most appropriate and accurate data available to complete the analysis. Some useful sources of information include previous major drainage planning studies, Natural Resources Conservation Service (NRCS) Soil Surveys, USGS mapping (detailed survey data are needed for design), the USGS StreamStats program, nearby rain gages and stream gages, storm sewer mapping maintained by most communities with a municipal NPDES stormwater permit, the City's FIMS data, historic and current aerial photography, and other sources.

1.4 Selecting Methods for Estimating Design Flows

The approved methods for estimating design flows and volumes are:

- Gage analysis
- Rational Method
- NRCS Curve Number Loss Method and Unit Hydrograph as implemented in USACE HEC-HMS model
- U.S. Environmental Protection Agency Stormwater Management Model (EPA SWMM)
- Bankfull regression equation (low flows)

Gage analyses can only be performed where a sufficiently long and reliable set of data is available. The application of this method depends on an understanding of the basin conditions over the period of time that the data were collected. Significant changes in the basin conditions, such as the construction of reservoirs or diversions or significant development, can make the data less reliable. This method is typically not useful for projecting future estimates unless changes in the drainage basin have been well documented over time.

When a rainfall/runoff methodology is used for hydrologic analyses, the Rational Method, the EPA SWMM method, or the NRCS Unit Hydrograph (Curve Number) method shall be applied. Alternative methods may be proposed on a case by case basis; however, these may be used only after careful consideration and with adequate justification and documentation that the results will be consistent with approved methods or locally available recorded data. The application of these methods is described in the UDFCD Manual, Volume 1, Runoff chapter and in this chapter. The Rational Method is a relatively simple approach used for smaller watersheds where only peak flows are required and a hydrograph is not required. For more complex drainage basins and routing requirements, the HEC-HMS model or the EPA SWMM method is better suited, but requires more experience and expertise to properly apply. The EPA SWMM method also provides hydrographs, reservoir routing, and the ability to evaluate runoff reduction practices in detail. For larger or complex drainage basins, the NRCS method may be used.

The bankfull regression equations can be used for estimating low flows for channel designs in major drainageways downstream of detention storage ponds. This method only provides a peak flow estimate and cannot provide an estimate of runoff volume.

The following considerations may help the user to select an appropriate method:

- If no detention facilities are planned or if detention facilities are to be sized using simplified methods, hydrograph information is not required, and the Rational Method would be the simpler of the methods. This applies only to small drainage basins without complex routing. The Rational Method is most commonly used for sizing inlets and storm sewers.

- If detention facilities are to be sized based on hydrograph routing, or if hydrograph information is desired for any other reason, the EPA SWMM or the NRCS method must be used.

- If more detailed information on time to peak, duration of flow, rainfall losses, and/or infiltration is desired, the EPA SWMM or the NRCS method (HEC-HMS model) offers this information.

- If the effects of runoff reduction practices need to be considered, each of the rainfall/runoff methods can be applied, but with varying levels of detail. This can be accomplished through the application of "effective imperviousness" values with each of these methods.

- Public domain software, including the USACE HEC-HMS model and EPA SWMM, is preferred to proprietary software because reviewers may not have the ability to open and inspect input and output files using proprietary models and because documentation of proprietary software packages is not always freely available. However, users of this Manual may use proprietary software and submittals will be allowed based on proprietary software provided that; (1) the proprietary software uses the methods accepted by this Manual, (2) the user provides a full listing of all input and output files in an easy to understand format that clearly shows that the model results comply with applicable criteria and (3) the results are comparable to what would be obtained using accepted software, such as HEC-HMS or EPASWMM. To confirm that the proprietary software produces similar results it be necessary to provide documentation of the methodologies used and sample comparisons of the results from each program for conditions specific to the project being evaluated.

Regardless of the method used, the maximum sub-watershed size for basin planning studies shall be approximately 130 acres. This is to reduce discrepancies in peak flow predictions between master plan hydrology and flow estimates based on single sub-watersheds significantly larger than 130 acres and to provide consistent guidance on sub-watershed delineation.

The selected method must be applied to calculate the flows corresponding to the return period of the design storms. In most cases, this will require calculations for both the "minor" and "major" storm events, at a minimum. Table 6-1 summarizes each method for estimating design flows.

Table 6-1. Methods for Estimating Design Flows

Method	Drainage Basin Area	Runoff Type	Routing Effects	System Complexity	BMP/Runoff Reduction
Gage Analysis	Any	Peak flow	NA	NA	NA
Rational Method	<130 acres	Peak flow	Simple	Simple	Effective imperviousness
NRCS/ HEC-HMS	Not typically applied to basins < 10 acres	Peak flow/volume/ /hydrograph	Simple to complex	Moderate to complex	Effective imperviousness
EPA SWMM	<640 acres (most commonly applied to urbanized watersheds)	Peak flow/volume/ /hydrograph	Simple to complex	Complex	Effective imperviousness, cascading planes or individual feature modeling
Bankfull Eq. (Eq. 6-24)	≥130 acres	Low flow peak only	NA	NA	NA

2.0 Rainfall

This section describes rainfall characteristics for use with the hydrologic methods in determining design flows and volumes. Rainfall data to be used are based on two sources:

- National Oceanic and Atmospheric Administration, *Precipitation-Frequency Atlas of the Western United States, Volume III-Colorado* (NOAA Atlas 2), published in 1973. Precipitation depth maps shown in the NOAA Atlas were used to determine representative 6-hour and 24-hour point rainfall values. Following the guidelines in the NOAA Atlas, these point values can be used to develop point rainfall values for various storm durations and frequencies. The NOAA Atlas is also used to provide Depth-Area Reduction Factor (DARF) curves for longer-duration (24-hour) events.

- *Fountain Creek Rainfall Characterization Study*, Carlton Engineering, Inc., prepared for the City of Colorado Springs, January, 2011. This study evaluated rainfall gage and gage-adjusted NEXRAD data within the Fountain Creek watershed and eastern Colorado. The results of this study have been evaluated and incorporated into this Manual.

2.1 Rainfall Depths

Rainfall depths must be determined based on the duration and return period of the design storm and the size of the drainage basin being evaluated. Depths can be derived by the methods described in the NOAA Atlas. The depths reported in the NOAA Atlas represent probable total depths for each duration and return period at a point on the ground. An extensive evaluation of available rain gage data was completed with the Carlton Study. While some increase in recorded depths was noted from the airport gage data, the other long-term gage locations showed that depths consistent with the NOAA Atlas can be expected. Since the NOAA Atlas is in the process of being updated, it was determined that the published atlas should continue to be used as the source of rainfall depths until this publication is revised or replaced.

The methods described in this Manual require only that the 1-hour, 6-hour and 24-hours depths be used as input. The storm return periods required for the application of methods in this Manual are the 2-, 5-, 10-, 25-, 50- and 100-year events. The 6-hour and 24-hour depths for these return periods can be read directly from Figures 6-6 through 6-17 at the end of this chapter. The1-hour depth for return periods can be calculated for all design return periods following this procedure:

Step 1: Calculate 2-year, 1-hour rainfall based on 2-year, 6-hour and 24-hour values.

$$Y_2 = 0.218 + 0.709 \cdot (X_1 \cdot X_1 / X_2) \qquad \text{(Eq. 6-1)}$$

Where:

Y_2 = 2-year, 1-hour rainfall (in)

X_1 = 2-year, 6-hour rainfall (in) from Figure 6-6

X_2 = 2-year, 24-hour rainfall (in) from Figure 6-12

Step 2: Calculate 100-year, 1-hour rainfall based on 2-year 6-hour and 24-hour values

$$Y_{100} = 1.897 + 0.439 \cdot (X_3 \cdot X_3 / X_4) - 0.008\ Z \qquad \text{(Eq. 6-2)}$$

Where

Y_{100} = 100-year, 1-hour rainfall (in)

X_3 = 100-year, 6-hour rainfall (in) from Figure 6-11

X_4 = 100-year, 24-hour rainfall (in) from Figure 6-17

Z = Elevation in hundreds of feet above sea level

Step 3: Plot the 2-year and 100-year, 1-hour values on the diagram provided in Figure 6-18 and connect the points with a straight line. The 1-hour point rainfall values for other recurrence intervals can be read directly from the straight line drawn on Figure 6-18.

Example: Determine the 10-year, 1-hour rainfall depth for downtown Colorado Springs.

Step 1*:* Calculate 2-year, 1-hour rainfall (Y_2) based on 2-year, 6-hour and 24-hour values. From Figure 6-6, the 2-year, 6-hour rainfall depth for downtown Colorado Springs is approximately 1.7 inches (X_1), and from Figure 6-12, the 2-year 24-hour depth is approximately 2.1 inches (X_2). The 2-year, 1-hour rainfall is calculated as follows:

$$Y_2 = 0.218 + 0.709 \cdot (1.7 \cdot 1.7 / 2.1) = 1.19 \text{ in} \qquad \text{(Eq. 6-3)}$$

Step 2*:* Calculate 100-year, 1-hour rainfall (Y_{100}) based on 100-year, 6-hour and 24-hour values. From Figure 6-11, the 100-year, 6-hour rainfall depth for downtown Colorado Springs is approximately 3.5 inches (X_3), and from Figure 6-17, the 100-year 24-hour depth is approximately 4.5 inches (X_4). Assume an elevation of 6,840 feet for Colorado Springs. The 100-year, 1-hour rainfall is calculated as follows:

$$Y_{100} = 1.897 + 0.439 \cdot (3.5 \cdot 3.5 / 4.6) - 0.008 \cdot (6{,}840 / 100) = 2.52 \text{ in} \qquad \text{(Eq. 6-4)}$$

Step 3*:* Plot 2-year and 100-year, 1-hour rainfall depths on Figure 6-18 and read 10-year value from straight line. This example is illustrated on Figure 6-18, with a 1-hour, 10-year rainfall depth of approximately 1.75 inches. Figure 6-18a provides the example, and Figure 6-18b provides a blank chart.

For Colorado Springs and much of the Fountain Creek watershed, the 1-hour depths are fairly uniform and are summarized in Table 6-2. Depending on the location of the project, rainfall depths may be calculated using the described method and the NOAA Atlas maps shown in Figures 6-6 through 6-17.

Table 6-2. Rainfall Depths for Colorado Springs

Return Period	1-Hour Depth	6-Hour Depth	24-Hour Depth
2	1.19	1.70	2.10
5	1.50	2.10	2.70
10	1.75	2.40	3.20
25	2.00	2.90	3.60
50	2.25	3.20	4.20
100	2.52	3.50	4.60

Where Z= 6,840 ft/100

These depths can be applied to the design storms or converted to intensities (inches/hour) for the Rational Method as described below. However, as the basin area increases, it is unlikely that the reported point rainfalls will occur uniformly over the entire basin. To account for this characteristic of rain storms an adjustment factor, the Depth Area Reduction Factor (DARF) is applied. This adjustment to rainfall depth and its effect on design storms is also described below. The UDFCD UD-Rain spreadsheet, available on UDFCD's website, also provides tools to calculate point rainfall depths and Intensity-Duration-Frequency curves[2] and should produce similar depth calculation results.

2.2 Design Storms

Design storms are used as input into rainfall/runoff models and provide a representation of the typical temporal distribution of rainfall events when the creation or routing of runoff hydrographs is required. It has long been observed that rainstorms in the Front Range of Colorado tend to occur as either short-duration, high-intensity, localized, convective thunderstorms (cloud bursts) or longer-duration, lower-intensity, broader, frontal (general) storms. The significance of these two types of events is primarily determined by the size of the drainage basin being studied. Thunderstorms can create high rates of runoff within a relatively small area, quickly, but their influence may not be significant very far downstream. Frontal storms may not create high rates of runoff within smaller drainage basins due to their lower intensity, but tend to produce larger flood flows that can be hazardous over a broader area and extend further downstream.

- **Thunderstorms**: Based on the extensive evaluation of rain storms completed in the Carlton study (Carlton 2011), it was determined that typical thunderstorms have a duration of about 2 hours. The study evaluated over 300,000 storm cells using gage-adjusted NEXRAD data, collected over a 14-year period (1994 to 2008). Storms lasting longer than 3 hours were rarely found. Therefore, the results of the Carlton study have been used to define the shorter duration design storms.

  To determine the temporal distribution of thunderstorms, 22 gage-adjusted NEXRAD storm cells were studied in detail. Through a process described in a technical memorandum prepared by the City of Colorado Springs (City of Colorado Springs 2012), the results of this analysis were interpreted and normalized to the 1-hour rainfall depth to create the distribution shown in Table 6-3 with a 5 minute time interval for drainage basins up to 1 square mile in size. This distribution represents the rainfall

depths over the duration of the storm as a fraction of the 1-hour depth and is also shown in Figure 6-19. By applying the 1-hour depths shown in Table 6-2 to the values shown in Table 6-3, a short-duration project design storm can be developed for any return period storm from a 2-year up to 100-year frequency. By applying the appropriate 1-hour depth for other project locations, a project design storm can be created for any location.

Table 6-3. 2-Hour Design Storm Distribution, ≤ 1 mi^2

Time (minutes)	Fraction of 1-Hour Rainfall Depth	Time (minutes)	Fraction of 1-Hour Rainfall Depth
5	0.014	65	1.004
10	0.046	70	1.018
15	0.079	75	1.030
20	0.120	80	1.041
25	0.179	85	1.052
30	0.258	90	1.063
35	0.421	95	1.072
40	0.712	100	1.082
45	0.824	105	1.091
50	0.892	110	1.100
55	0.935	115	1.109
60	0.972	120	1.119

- **Frontal Storms**: The characteristics of longer-duration "frontal storms" (general) is less well understood than the shorter duration thunderstorms and should be studied further. However, some events of this nature have been observed, such as the April 1999 storm which produced flooding on Fountain Creek, showing that these types of events do occur and tend to produce hazardous flood flows. In addition, modeling of the Jimmy Camp Creek drainage basin using the 24-hour, Type II distribution shows that it produces results reasonably comparably to recorded flow data. Therefore, the NRCS 24-hour Type II distribution has replaced the Type IIa distribution as the standard, long-duration design storm. This distribution can be applied to drainage basins up to 10 square miles without a DARF correction and is shown in Table 6-4. This distribution is included as a standard storm option in the HEC-HMS program.

Table 6-4. NRCS 24-Hour Type II Design Storm Distribution, <10 mi^2
(Fraction of 24-Hour Rainfall Depth)

Hour	Minutes			
	0	15	30	45
0	0.000	0.0020	0.0050	0.0080
1	0.0110	0.0140	0.0170	0.0200
2	0.0230	0.0260	0.0290	0.0320
3	0.0350	0.0380	0.0410	0.0440
4	0.0480	0.0520	0.0560	0.0600
5	0.0604	0.0680	0.0720	0.0760
6	0.0800	0.0850	0.0900	0.0950
7	0.1000	0.1050	0.1100	0.1150
8	0.1200	0.1260	0.1330	0.1400
9	0.1470	0.1550	0.1630	0.1720
10	0.1810	0.1910	0.2030	0.2180
11	0.2360	0.2570	0.2830	0.3870
12	0.6630	0.7070	0.7350	0.7580
13	0.7760	0.7910	0.8040	0.8150
14	0.8250	0.8340	0.8420	0.8490
15	0.8560	0.8630	0.8690	0.8750
16	0.8810	0.8870	0.8930	0.8980
17	0.9030	0.9080	0.9130	0.9180
18	0.9220	0.9260	0.9300	0.9340
19	0.9380	0.9420	0.9460	0.9500
20	0.9530	0.9560	0.9590	0.9620
21	0.9650	0.9680	0.9710	0.9740
22	0.9770	0.9800	0.9830	0.9860
23	0.9890	0.9920	0.9950	0.9980

2.2.1 Depth-Area Reduction Factors (DARFs)

Depth Area Reduction Factors (DARFs) are used to adjust point rainfall depths to average depths as the size of drainage basins increase. As a part of the 2011 rainfall study, Carlton analyzed radar data to develop DARF curves applicable to the Fountain Creek watershed, El Paso County and eastern Colorado. However, these relationships were determined for short-duration thunderstorms and are not applicable to longer-duration frontal storms. Therefore, the DARFs provided in the NOAA Atlas will continue to be applied for the frontal-type storms.

- **Thunderstorm DARFs**: The Carlton study provided DARF curves for various storm return periods for short-duration thunderstorm events; however, the difference between the sets of curves was determined to be insignificant. As described in the technical memorandum *Stormwater Management Assessment and Standards Development Project, Proposed Rainfall and Standard Design Storms* (City of Colorado Springs 2012), the 5-year set of DARF curves was selected for the development of thunderstorm type design storms. These DARF curves for short-duration events are shown in Figure 6-21 at the end of this chapter.

  As described in the memorandum documenting the development of design storms, the HEC-HMS program provides guidance on the application of DARFs to define adjusted design storms as the

drainage basin area increases. This is done be applying the appropriate DARF to the corresponding depth for the same duration throughout the storm distribution. The resulting adjusted design storms are shown in Table 6-5 and in Figure 6-19 at the end of this chapter. Because the DARFs decrease rather dramatically as drainage basin size increases, there is an upper limit for which these factors can be practically applied. The application of DARFs is based on the assumption that rainfall is uniform over the entire drainage basin being evaluated. When the DARF-adjusted average rainfall becomes too low it no longer is a reasonable representation of the more intense rainfall that occurs over only a portion of the drainage basin. By applying the appropriate 1-hour depth, a project design storm can be created for any location using Table 6-5.

Table 6-5. 2-Hour Design Storm Distributions by Drainage Basin Area
(DARF-adjusted fraction of 1-Hour Depth)

Time Min.	Drainage Basin Area (square miles)						
	0-1	>1-5	>5-10	>10-15	>15-20	>20-40	>40-60
0	0	0	0	0	0	0	0
5	0.014	0.014	0.014	0.014	0.015	0.015	0.017
10	0.046	0.044	0.041	0.041	0.042	0.042	0.040
15	0.079	0.076	0.074	0.074	0.073	0.070	0.068
20	0.120	0.116	0.109	0.109	0.106	0.102	0.095
25	0.179	0.176	0.169	0.168	0.163	0.157	0.147
30	0.258	0.249	0.239	0.236	0.227	0.216	0.198
35	0.421	0.396	0.354	0.327	0.307	0.276	0.242
40	0.712	0.655	0.559	0.495	0.448	0.381	0.315
45	0.824	0.756	0.637	0.560	0.506	0.422	0.345
50	0.892	0.824	0.700	0.619	0.566	0.479	0.396
55	0.935	0.866	0.740	0.658	0.601	0.512	0.428
60	0.972	0.901	0.774	0.690	0.634	0.543	0.456
65	1.004	0.934	0.806	0.717	0.661	0.570	0.482
70	1.018	0.948	0.821	0.732	0.678	0.589	0.501
75	1.030	0.962	0.835	0.746	0.692	0.603	0.515
80	1.041	0.973	0.849	0.760	0.706	0.617	0.529
85	1.052	0.984	0.863	0.774	0.720	0.631	0.543
90	1.063	0.995	0.875	0.788	0.734	0.645	0.557
95	1.072	1.006	0.886	0.802	0.748	0.659	0.571
100	1.082	1.017	0.896	0.813	0.762	0.673	0.585
105	1.091	1.026	0.907	0.824	0.773	0.687	0.599
110	1.100	1.036	0.918	0.835	0.783	0.698	0.611
115	1.109	1.045	0.929	0.846	0.794	0.709	0.622
120	1.119	1.054	0.938	0.857	0.805	0.720	0.633

Table Notes:
1. Distributions are similar to distribution created using HEC-HMS 3.5 Frequency Storm option, with peak intensity at 33% of storm duration and by averaging the distributions for the 1-, 2-, 5- and 100-year events.
2. Rainfall depth adjustment factors were based on data for Colorado Springs and adjusted using the 5-year DARFs developed by Carlton, January, 2011.

- **Frontal Storm DARFs**: Because the Carlton study did not include an evaluation of DARFs for longer-duration, frontal-type storms it was concluded that the DARFs based on the NOAA Atlas 2 guidance, with minor modifications by UDFCD (UDFCD 2001), should be used. Figure 6-22 provides DARF curves for the longer-duration, larger events. For drainage basins larger than 10

square miles, the curves provided in Figures 6-21 and 6-22 can be applied to account for aerial reductions in point precipitation for the durations shown.

Design storms for a 24-hour NRCS Type II distribution are integrated into the HEC-HMS software program and this program will create a DARF-adjusted design storm. The only data required are the unadjusted 24-hour rainfall depth for the return period being evaluated and the size of the drainage basin. The program makes the appropriate adjustments to the rainfall depth for the area of the basin being evaluated and distributes the rainfall over the storm duration accordingly.

2.2.2 Dominant Design Storm

For flood studies or when the highest probable design flow for sizing facilities is required, it may be necessary to evaluate both thunderstorms and frontal storms to determine the appropriate design flows. It is the responsibility of the designer to determine the dominant design storm for each project. Both peak flow rates and runoff volumes should be checked since the volume of runoff can be a critical design parameter for some types of facilities, especially those designed for detention storage.

Also, it must be recognized that each design storm applies to the total drainage area included in the study area and that the resulting flows only apply to the reach that receives the total area. To determine peak flows for smaller portions of the drainage basin, different design storms, based on different DARFs appropriate for the contributing area, may be needed to determine design flows. For example, within a 60-square-mile drainage basin, it may be necessary to apply a thunderstorm distribution to determine peak flows for sub-basins of 1 square mile and smaller and for other portions up to the area where the frontal storm dominates.

It is important that both of these types of design storms assume that the rainfall occurs uniformly over the entire drainage basin. For larger drainage basins, such as the area contributing to lower Fountain Creek, a spatial distribution of the storm is likely to be more representative of an appropriate design storm that will reproduce low frequency flood flow estimates. This type of storm distribution (based on "storm centering") was used in the 2006 *Fountain Creek Watershed Hydrology Report* and may need to be considered for certain projects, especially those involving large watersheds and/or large and complex stream systems. The application of this type of design storm is complicated and requires experience and judgment to determine the placement of the storm over the watershed so that the highest potential flows are created. Resources from the Colorado Department of Natural Resources, State Engineers Office, Dam Safety Office provide guidance on developing storm events for Extreme Storm Precipitation (ESP). Additionally, relevant work by Dr. James Guo at the University of Colorado-Denver includes a peer-reviewed paper "Storm Centering Approach for Flood Predictions from Large Watersheds" in the *Journal of Hydrologic Engineering* (Guo, publication pending as of July 2012). For extreme precipitation events, such as estimating the Probable Maximum Precipitation (PMP), see *Hydrometeorological Report No. 55A (HMR-55A), Probable Maximum Precipitation Estimates—United States Between the Continental Divide and the 103rd Meridian* and/or the Extreme Precipitation Analysis Tool (EPAT) developed by the State Engineer's Office.

2.3 Hydrologic Basis of Design for Water Quality—Water Quality Capture Volume

While guidance in the preceding sections focuses on the hydrologic events related to flood control and conveyance facilities, small frequently occurring events form the basis of design for water quality facilities. The water quality capture volume (WQCV), corresponding to roughly an 85th percentile event, defines storage volume requirements for stormwater best management practices (BMPs). The basis for establishing the 85th percentile event and guidance for implementing water quality facilities is described in the Volume 2 of this Manual.

The guidance provided in Volume 2 of this Manual was based on data from the Denver Metropolitan area. A detailed analysis of rainfall gage records in the Colorado Springs area was conducted to determine an appropriate value for the 85th percentile storm. The results of this analysis are reported in a technical memorandum prepared for the City titled "Water Quality Capture Volume Analysis for Colorado Springs" (Wright Water Engineers 2011). While there were some minor differences between the UDFCD data and the data from the Colorado Springs gages, on average, the curves were very similar. Based on the results of this report, the UDFCD results and methods for the WQCV are acceptable for determining the WQCV in Colorado Springs.

3.0 Rational Method

The Rational Method is used to determine runoff peak discharges for drainage basins up to and including 130 acres in size and when hydrologic routing is relatively simple. However, the drainage area should be divided into sub-basins that represent homogeneous land uses, soil types or land cover. The Rational Method is most typically applied for inlet and storm drain sizing.

The Rational Method is based on the direct relationship between rainfall and runoff, and is expressed by the following equation:

$$Q = C \cdot I \cdot A \qquad \text{(Eq. 6-5)}$$

In which:

Q = the maximum rate of runoff (cubic feet per second [cfs])

C = the runoff coefficient that is the ratio between the runoff volume from an area and the average rainfall depth over a given duration for that area

I = the average intensity of rainfall for a duration equal to the time of concentration (in/hr)

A = drainage basin area (acres)

The assumptions and limitations of the Rational Method are described in the UDFCD Manual, Volume 1, Runoff chapter. Standard Form 1 (SF-1) and Standard Form 2 (SF-2) are provided at the end of this chapter as Figure 6-23 and Figure 6-24, respectively to provide a standard format for Rational Method calculations. The SF-1 Form is used for calculating the time of concentration, and the SF-2 form is used to estimate accumulated peak discharges from multiple basins as storm runoff flows downstream in a channel or pipe. Results from the Rational Method calculations shall be included with the drainage report submittal. As an alternative to SF-1 and SF-2, the UD-Rational spreadsheet can be used to document basin parameters and calculations or other spreadsheets or programs can be used as long as the information and format is the similar to that shown in these standard forms.

3.1 Rational Method Runoff Coefficient (C)

The runoff coefficient represents the integrated effects of infiltration, detention storage, evaporation, retention, flow routing, and interception, all of which affect the time distribution and peak rate of runoff. Runoff coefficients are based on the imperviousness of a particular land use and the hydrologic soil type of the area and are to be selected in accordance with Table 6-6.

The procedure for determining the runoff coefficient includes these steps:

1. Categorize the site area into one or more similar land uses, each with a representative imperviousness, according to the information in Table 6-6.

2. Based on the dominant hydrologic soil type in the area, use Table 6-6 to estimate the runoff coefficient for the particular land use category for the design storms of interest.

3. Calculate an area-weighted average runoff coefficient for the site based on the runoff coefficients from individual land use areas of the site.

When analyzing an area for design purposes, urbanization of the full watershed, including both on-site and off-site areas, shall be assumed.

Gravel parking areas, storage areas, and access drives proposed on Site Improvement Plans shall be analyzed based on an imperviousness of 80%. This is due to the potential for gravel areas being paved over time by property owners and the resulting adverse impacts on the stormwater management facilities and adjacent properties.

There are some circumstances where the selection of impervious percentage values may require additional investigation due to unique land characteristics (e.g., recent burn areas). When these circumstances arise, it is the designer's responsibility to verify that the correct land use assumptions are made.

When multiple sub-basins are delineated, the composite C value calculation is:

$$C_c = (C_1A_1 + C_2A_2 + C_3A_3 + \ldots C_iA_i) / A_t \qquad \text{(Eq. 6-6)}$$

Where:

C_c = composite runoff coefficient for total area

C_i = runoff coefficient for subarea corresponding to surface type or land use

A_i = area of surface type corresponding to C_i (units must be the same as those used for total area)

A_t = total area of all subareas for which composite runoff coefficient applies

i = number of surface types in the drainage area

Table 6-6. Runoff Coefficients for Rational Method
(Source: UDFCD 2001)

Land Use or Surface Characteristics	Percent Impervious	Runoff Coefficients											
		2-year		5-year		10-year		25-year		50-year		100-year	
		HSG A&B	HSG C&D	HSG A&B	HSG C&D	HSG A&B	HSG C&D	HSG A&B	HSG C&D	HSG A&B	HSG C&D	HSG A&B	HSG C&D
Business													
Commercial Areas	95	0.79	0.80	0.81	0.82	0.83	0.84	0.85	0.87	0.87	0.88	0.88	0.89
Neighborhood Areas	70	0.45	0.49	0.49	0.53	0.53	0.57	0.58	0.62	0.60	0.65	0.62	0.68
Residential													
1/8 Acre or less	65	0.41	0.45	0.45	0.49	0.49	0.54	0.54	0.59	0.57	0.62	0.59	0.65
1/4 Acre	40	0.23	0.28	0.30	0.35	0.36	0.42	0.42	0.50	0.46	0.54	0.50	0.58
1/3 Acre	30	0.18	0.22	0.25	0.30	0.32	0.38	0.39	0.47	0.43	0.52	0.47	0.57
1/2 Acre	25	0.15	0.20	0.22	0.28	0.30	0.36	0.37	0.46	0.41	0.51	0.46	0.56
1 Acre	20	0.12	0.17	0.20	0.26	0.27	0.34	0.35	0.44	0.40	0.50	0.44	0.55
Industrial													
Light Areas	80	0.57	0.60	0.59	0.63	0.63	0.66	0.66	0.70	0.68	0.72	0.70	0.74
Heavy Areas	90	0.71	0.73	0.73	0.75	0.75	0.77	0.78	0.80	0.80	0.82	0.81	0.83
Parks and Cemeteries	7	0.05	0.09	0.12	0.19	0.20	0.29	0.30	0.40	0.34	0.46	0.39	0.52
Playgrounds	13	0.07	0.13	0.16	0.23	0.24	0.31	0.32	0.42	0.37	0.48	0.41	0.54
Railroad Yard Areas	40	0.23	0.28	0.30	0.35	0.36	0.42	0.42	0.50	0.46	0.54	0.50	0.58
Undeveloped Areas													
Historic Flow Analysis-- Greenbelts, Agriculture	2	0.03	0.05	0.09	0.16	0.17	0.26	0.26	0.38	0.31	0.45	0.36	0.51
Pasture/Meadow	0	0.02	0.04	0.08	0.15	0.15	0.25	0.25	0.37	0.30	0.44	0.35	0.50
Forest	0	0.02	0.04	0.08	0.15	0.15	0.25	0.25	0.37	0.30	0.44	0.35	0.50
Exposed Rock	100	0.89	0.89	0.90	0.90	0.92	0.92	0.94	0.94	0.95	0.95	0.96	0.96
Offsite Flow Analysis (when landuse is undefined)	45	0.26	0.31	0.32	0.37	0.38	0.44	0.44	0.51	0.48	0.55	0.51	0.59
Streets													
Paved	100	0.89	0.89	0.90	0.90	0.92	0.92	0.94	0.94	0.95	0.95	0.96	0.96
Gravel	80	0.57	0.60	0.59	0.63	0.63	0.66	0.66	0.70	0.68	0.72	0.70	0.74
Drive and Walks	100	0.89	0.89	0.90	0.90	0.92	0.92	0.94	0.94	0.95	0.95	0.96	0.96
Roofs	90	0.71	0.73	0.73	0.75	0.75	0.77	0.78	0.80	0.80	0.82	0.81	0.83
Lawns	0	0.02	0.04	0.08	0.15	0.15	0.25	0.25	0.37	0.30	0.44	0.35	0.50

3.2 Time of Concentration

One of the basic assumptions underlying the Rational Method is that runoff is a function of the average rainfall rate during the time required for water to flow from the hydraulically most remote part of the drainage area under consideration to the design point. However, in practice, the time of concentration can be an empirical value that results in reasonable and acceptable peak flow calculations.

For urban areas, the time of concentration (t_c) consists of an initial time or overland flow time (t_i) plus the travel time (t_t) in the storm sewer, paved gutter, roadside drainage ditch, or drainage channel. For non-urban areas, the time of concentration consists of an overland flow time (t_i) plus the time of travel in a concentrated form, such as a swale or drainageway. The travel portion (t_t) of the time of concentration can be estimated from the hydraulic properties of the storm sewer, gutter, swale, ditch, or drainageway. Initial time, on the other hand, will vary with surface slope, depression storage, surface cover, antecedent rainfall, and infiltration capacity of the soil, as well as distance of surface flow. The time of concentration is represented by Equation 6-7 for both urban and non-urban areas.

$$t_c = t_i + t_t \qquad \text{(Eq. 6-7)}$$

Where:

t_c = time of concentration (min)

t_i = overland (initial) flow time (min)

t_t = travel time in the ditch, channel, gutter, storm sewer, etc. (min)

3.2.1 Overland (Initial) Flow Time

The overland flow time, t_i, may be calculated using Equation 6-8.

$$t_i = \frac{0.395(1.1 - C_5)\sqrt{L}}{S^{0.33}} \qquad \text{(Eq. 6-8)}$$

Where:

t_i = overland (initial) flow time (min)
C_5 = runoff coefficient for 5-year frequency (see Table 6-6)
L = length of overland flow (300 ft <u>maximum</u> for non-urban land uses, 100 ft <u>maximum</u> for urban land uses)
S = average basin slope (ft/ft)

Note that in some urban watersheds, the overland flow time may be very small because flows quickly concentrate and channelize.

3.2.2 Travel Time

For catchments with overland and channelized flow, the time of concentration needs to be considered in combination with the travel time, t_t, which is calculated using the hydraulic properties of the swale, ditch, or channel. For preliminary work, the overland travel time, t_t, can be estimated with the help of Figure 6-25 or Equation 6-9 (Guo 1999).

$$V = C_v S_w^{0.5} \qquad \text{(Eq. 6-9)}$$

Where:

V = velocity (ft/s)

C_v = conveyance coefficient (from Table 6-7)

S_w = watercourse slope (ft/ft)

Table 6-7. Conveyance Coefficient, C_v

Type of Land Surface	C_v
Heavy meadow	2.5
Tillage/field	5
Riprap (not buried)*	6.5
Short pasture and lawns	7
Nearly bare ground	10
Grassed waterway	15
Paved areas and shallow paved swales	20

* For buried riprap, select C_v value based on type of vegetative cover.

The travel time is calculated by dividing the flow distance (in feet) by the velocity calculated using Equation 6-9 and converting units to minutes.

The time of concentration (t_c) is then the sum of the overland flow time (t_i) and the travel time (t_t) per Equation 6-7.

3.2.3 First Design Point Time of Concentration in Urban Catchments

Using this procedure, the time of concentration at the first design point (typically the first inlet in the system) in an urbanized catchment should not exceed the time of concentration calculated using Equation 6-10. The first design point is defined as the point where runoff first enters the storm sewer system.

$$t_c = \frac{L}{180} + 10 \qquad \text{(Eq. 6-10)}$$

Where:

t_c = maximum time of concentration at the first design point in an urban watershed (min)

L = waterway length (ft)

Equation 6-10 was developed using the rainfall-runoff data collected in the Denver region and, in essence, represents regional "calibration" of the Rational Method. Normally, Equation 6-10 will result in a lesser time of concentration at the first design point and will govern in an urbanized watershed. For subsequent design points, the time of concentration is calculated by accumulating the travel times in downstream drainageway reaches.

3.2.4 Minimum Time of Concentration

If the calculations result in a t_c of less than 10 minutes for undeveloped conditions, it is recommended that a minimum value of 10 minutes be used. The minimum t_c for urbanized areas is 5 minutes.

3.2.5 Post-Development Time of Concentration

As Equation 6-8 indicates, the time of concentration is a function of the 5-year runoff coefficient for a drainage basin. Typically, higher levels of imperviousness (higher 5-year runoff coefficients) correspond to shorter times of concentration, and lower levels of imperviousness correspond to longer times of

concentration, all other factors being equal. Although it is possible to calculate a longer time of concentration for a post-development condition versus a pre-development condition by increasing the length of the flow path, this is often a result of selecting unrealistic flow path lengths. As a matter of practice and for the sake of conservative design, it is required that the post-development time of concentration be less than or equal to the pre-development time of concentration. As a general rule and when sufficiently detailed development plans are not available, the post-development time of concentration can be estimated to be about 75% of the pre-development value.

3.2.6 Common Error in Calculating Time of Concentration

A common error in estimating the time of concentration occurs when a designer does not check the peak runoff generated from smaller portions of the catchment that may have a significantly shorter time of concentration (and, therefore, a higher rainfall intensity) than the drainage basin as a whole. Sometimes calculations using the Rational Method for a lower, urbanized portion of a watershed will produce a higher peak runoff than the calculations for the drainage basin as a whole, especially if the drainage basin is long or the upper portion has little or no impervious cover.

3.3 Rainfall Intensity (I)

The average rainfall intensity (I), in inches per hour, by recurrence interval, can be found from the Intensity-Duration-Frequency curves provided in Figure 6-5. The value for I is based on the assumption that the peak runoff will occur when the duration of the rainfall is equal to the time of concentration. For example, Figure 6-5 indicates a rainfall intensity of approximately 5.00 inches/hour for the 100-year event for a catchment with a time of concentration of 20 minutes. These curves are based on the rainfall depths for an elevation of 6,840 feet in the Colorado Springs area. IDF curves for other elevations or locations can be created using the UD-Rain spreadsheet based on 6-hour and 24-hour rainfall depths for each recurrence interval needed. The Z-1 (Zone 1) tab should be used for Arkansas River basin locations.

3.4 Drainage Basin Area (A)

The size of a drainage basin contributing runoff to a design point, in acres, is used to calculate peak runoff in the Rational Method. Accurately delineating the area contributing to each design point is one of the most important tasks for hydrologic analyses since the estimated runoff is directly proportional to the basin area. The area may be determined through the use of planimetric-topographic maps, supplemented by field surveys where topographic data has changed or where the contour interval is too great to distinguish the direction of flow. The drainage basin lines are determined by the natural topography, pavement slopes, locations of downspouts and inlets, paved and unpaved yards, grading of lawns, and many other features found on the urban landscape. In areas where there are storm drains, the entire contributing drainage area can sometimes be greater than the drainage area determined by topographic analysis of the ground surface, due to storm drains collecting runoff from areas that lie outside of the surface topographic extent of the basin.

4.0 NRCS Curve Number Loss and Dimensionless Unit Hydrograph Method

The NRCS curve number loss and dimensionless unit hydrograph method has used been the most widely used method in the region. It can be applied for drainage basins as small as 10 acres and is the only method that should be applied for drainage basins larger than 640 acres. This method can be used to estimate peak flows or to produce a runoff hydrograph and also provides estimates of runoff volume.

Detailed descriptions of the curve number loss method and the dimensionless unit hydrograph can be found in these references:

- U.S. Department of Agriculture Natural Resources Conservation Service (NRCS) 1986. *Urban Hydrology for Small Watersheds*. Technical Release 55 (TR-55) (Second Edition). Prepared by Conservation Engineering Division.

- U.S. Army Corps of Engineers (USACE) 2010. *Hydrologic Modeling System HEC-HMS User's Manual*. Hydrologic Engineering Center, CPD-74A.

While it is possible to perform hydrograph analysis using the NRCS curve number loss method and dimensionless unit hydrograph using spreadsheet tools, it is cumbersome. More commonly, computer models such as the USACE HEC-HMS model are used. This section describes model input requirements for pre- and post-development modeling using HEC-HMS. Primary inputs include basin characteristics such as the drainage area, curve number and lag time. In addition, channel routing parameters are specified in HEC-HMS.

Other computer programs that use the NRCS loss method and dimensionless unit hydrograph may also be used, provided that the model results can be replicated using HEC-HMS. However, the curve number option for calculating rainfall losses in EPA SWMM is not acceptable because it is not an accurate implementation of the NRCS method and may produce results that vary significantly from HEC-HMS and TR-55.

4.1 NRCS Curve Numbers

NRCS curve numbers range from 0 to 100 (the recommended lower limit is 40) and can be used to calculate the volume of runoff from a storm event based on land use characteristics. A curve number of 0 would represent zero runoff (100% losses), and a curve number of 100 would represent zero losses (100% runoff).

The selection of a curve number value depends on the type of soil, identified by the NRCS hydrologic soil group (HSG), the land cover or treatment, and the antecedent runoff condition (ARC).

4.1.1 Hydrologic Soil Groups (HSG)

HSGs are determined by soil surveys published by the NRCS, which are generally done on a county-wide basis. The NRCS Soil Survey Geographic (SSURGO) Database is an online tool that may be used to characterize soils and HSGs.

The locations of each soil type for the drainage basin being studied must be identified by their HSG designation. The four hydrologic soil groups are defined by soil scientists, according to their runoff potential, as:

- **Group A**: Low runoff potential. Soils having low runoff potential and high infiltration rates even when thoroughly wetted. These consist chiefly of deep, well to excessively drained sand or gravel and have a high rate of water transmission (> 0.30 in/hr).

- **Group B**: Moderate runoff potential. Soils having moderate infiltration rates when thoroughly wetted and consist chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission (0.15-0.30 in/hr).

- **Group C**: Moderate to High runoff potential. Soils having low infiltration rates when thoroughly wetted and consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately fine to fine texture. These soils have a low rate of water transmission (0.05-0.15 in/hr).

- **Group D**: High runoff potential. Soils have high runoff potential. They have very low infiltration rates when thoroughly wetted and consist chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over a nearly impervious material. These soils have a very low rate of water transmission (0.00-0.05 in/hr).

Soils in the Pike National Forest

Large portions of the Fountain Creek watershed extend into the foothills of the Rocky Mountains and also include the northern and eastern faces of Pikes Peak. Soils in these areas were mapped as part of a soil survey completed for the Pike National Forest in 1992. The Soil Survey of the Pike National Forest (USDA 1992) is a third order survey while the Soil Survey of El Paso County, encompassing the balance of the County, is a second order survey (USDA 1981). The order of a soil survey indicates its level of detail and intended use. Third order surveys are "extensive" in nature and are typically conducted at twice the scale when compared with more "intensive" second order surveys. According to the Soil Survey Manual (USDA 1993), "[t]hird order surveys are made for land uses that do not require precise knowledge of small areas or detailed soils information." As a result, soil mapping for some portions of the foothills does not have adequate resolution to accurately characterize rock outcroppings, depth to bedrock and potential for infiltration and runoff.

Many of the soils in the Pike National Forest were assigned to Group D likely due to the inclusion of scattered rock outcroppings and a perceived depth to bedrock. However, these soils are derived from decomposed Pikes Peak granite parent material that is highly fractured and deeply weathered below the soil profile. These soils have very gravelly coarse sandy loam textures and exhibit high infiltration rates with no free water occurring within the soil profile. As such, they do not meet the definition of HSG D as defined by the Soil Survey Manual (Table 3-9 in USDA 1993).

For the purposes of establishing hydrology for City projects, the HSG for soil mapping units in the Pike National Forest should be assigned as shown in Table 6-8. These HSG assignments vary from the original published data and may not be appropriate for hydrology studies requiring the approval of other agencies or jurisdictions (e.g., floodplain study requiring FEMA approval). Soils mapped in the Sphinx, Catamount, and Legault mapping units were originally published as Group D soils, but should be treated as HSG B. Soils mapped in the Ivywild mapping unit were originally published as Group C soils, but should be treated as HSG B. Soils mapped in the Circue land mapping unit were originally published as Group A soils, but should be treated as HSG D. Soils mapped in the Rock outcrop, Tecolote, Aquolls, Condie, and Pendant mapping units shall retain their published HSG. Other minor soils map units shall retain their published HSG. Where runoff from rock outcroppings flows onto pervious areas, it may also be reasonable to represent the outcroppings as disconnected impervious area based on guidance provided in TR-55.

Table 6-8. HSG for Soils in the Pike National Forest

Map Symbol	Major Soil Component	Assigned HSG
42,43,44,45,46,47	Sphinx	B
5,6,7	Catamount	B
21	Ivywild	B
33,34,35,36	Rock outcrop	D
24,25,26	Legault	B
48,49	Tecolote	B
9	Cirque land	D
2	Aquolls	D
10	Condie	B
29,31	Pendant	D

Note: Minor soil map units not listed above shall retain the published HSG.

4.1.2 Land Cover

The type of cover on the surface of the ground within a drainage basin has a significant effect on the amount and rate of runoff. Land cover includes type of vegetation, density of vegetation and impervious surfaces including roads, buildings, parking lots, etc. The standard method for adjusting curve number for imperviousness assumes that the impervious areas are directly connected to the receiving system. Adjustments to imperviousness to determine "effective imperviousness" can be made as described in Volume 3 of the UDFCD Manual (with accompanying spreadsheet) when runoff reduction practices such as BMPs and LID practices are implemented.

4.1.3 Antecedent Runoff Condition (ARC)

The ARC represents the conditions in the drainage basin prior to the onset of the design storm event relative to runoff potential and can be influenced by the type of storm being evaluated. It is represented by three categories: ARC I, ARC II and ARC III. ARC I represents the lowest runoff potential, and ARC III represents the highest. Considerations for thunderstorm and frontal storm ARCs include:

- **Thunderstorm ARCs**: Previously, an ARC II category was used as the standard condition for all design purposes. However, as a part of the update of this Manual, Colorado Springs conducted a hydrologic modeling study of the Jimmy Creek Camp watershed to evaluate appropriate curve number values. This study included model simulations that were calibrated to USGS stream gage data just upstream of the confluence with Fountain Creek. The curve number values presented in this section were selected based on NRCS guidance and the results of the modeling study. One of the most notable conclusions of the modeling analysis, which was also supported by the Carleton (2011) study, is that basin conditions prior to short-duration storm events are better represented by ARC I curve numbers. The modeling analyses showed that using curve numbers based on ARC II significantly overestimate pre-development runoff based on the relatively short-duration storm events that were studied. However, when areas develop most pervious areas will be landscaped and irrigated. Therefore, the developed condition ARC is better represented by curve numbers based on an ARC II condition.

- **Frontal Storm ARCs**: A detailed analysis of conditions prior to longer-duration storms has not been conducted due to the lack of adequate data. However, by observation and by a detailed evaluation of the April 1999 storm event, it is apparent that longer-duration storms tend to be part of a broader storm system with rainfall occurring in the days leading up to a more intense period of rainfall.

Therefore, curve numbers for longer-duration frontal storms will continue to be based on ARC II conditions. Under some conditions, an ARC III category could be appropriate, but there is insufficient storm and basin data to establish these conditions, but they are expected to be rare.

4.2 Pre-development Thunderstorm Curve Numbers

Pre-development (undeveloped) curve numbers are determined based on land use and cover, the HSG and the ARC. For undeveloped land, ARC I (lower runoff potential) applies for short-duration thunderstorms. Table 6-9 provides curve numbers for undeveloped, non-irrigated land that should be used for assessment of pre-development hydrology for thunderstorms.

4.3 Frontal Storm and Post-Development Thunderstorm Curve Numbers

Post-development curve numbers are determined using the standard guidance provided by the NRCS for ARC II from Technical Release (TR) 55 guidance (NRCS 1986) when pervious areas are landscaped and irrigated for both short-duration thunderstorms and longer-duration frontal storms. Because it is anticipated that conditions prior to frontal storm events in undeveloped drainage basins will have increased runoff potential, ARC II curve numbers should also be used for these analyses. Table 6-10 provides curve numbers to be used for assessing these conditions.

Also, to recognize that soils within a development project are usually disturbed and covered with top soil, sod or landscaping and irrigated, Type A soils must be represented as Type B soils for post development curve number calculations. Type A soils are not required to be represented as Type B soils if these portions of a site are avoided and protected during development. However, if they are irrigated, they must be represented by ARC II curve numbers.

4.4 Composite Curve Numbers

Drainage basins are often composed of various soil types, land uses, land covers or other features that cannot be represented by a single value from the standard tables. To represent these conditions a composite curve number must be calculated using the following equation:

$$CN_c = (CN_1 A_1 + CN_1 A_2 + CN_3 A_3 + \ldots CN_i A_i) / A_t \qquad \text{(Eq. 6-11)}$$

Where:

CN_c = composite curve number for total area

CN_i = curve number for subarea

A_i = area of each subarea (units must be consistent with units used for total area)

A_t = total area of all drainage subareas for which composite curve number applies

While compositing curve numbers is a fairly common practice, it is important to remember that curve numbers are non-linear and compositing methods assume a linear relationship (for example, a curve number of 90 does not produce twice as much runoff as a curve number of 45 for the same rainfall amount). If there are large variations in the magnitude of curve numbers that are being composited, sub-basins should be redefined to represent more homogeneous land uses or runoff can be calculated from individual land uses, added together and a representative curve number for the overall basin can be back-calculated. This is especially important in urban areas, where compositing of highly impervious areas with less pervious land uses can result in under prediction of peak runoff peaks and volumes. To the extent practical, subareas should be defined to avoid compositing of curve numbers for land uses with

distinct differences in runoff characteristics. Note that the composite curve number values shown in Table 6-10 for various land uses types do not include the adjacent streets and sidewalks. These areas, and their corresponding curve numbers, should be incorporated into the calculation of the overall composite curve number with the areas and curve numbers for the other land use types within a subarea.

Some software programs, including HEC-HMS, provide an option to represent directly connected impervious areas by entering a percent imperviousness for a subarea. In this case, the runoff volume from the directly connected impervious area is calculated separately from the remaining portion of the subarea which is represented by a composite curve number. When applying this method only directly connected impervious areas such as streets, sidewalks, driveways, parking areas and roof sections that are hydraulically connected should be included in the percent impervious value. The composite curve number used incorporates the curve number values for the various pervious areas and any disconnected impervious areas not included in the percent impervious value. This method may provide a more accurate representation of the effect of urbanization and directly connected imperviousness, especially for the more frequent storm events (ie. 5-year or less).

4.5 Initial Abstraction

The initial abstraction (Ia) represents a volume of rainfall that must fall to satisfy losses in a drainage basin before runoff begins. The default value for Ia is 0.20 times the potential maximum retention (S). Through modeling of the Jimmy Camp Creek drainage basin using gage-adjusted, NEXRAD-generated rainfall input and comparing model results with recorded flow data, it was determined that a more appropriate value for Ia is 0.10·S. Therefore, this value shall replace the default value for any evaluations that apply the NRCS curve number method for rainfall losses. To apply this adjustment when using HEC-HMS it will be necessary to provide the initial abstraction as a depth in inches rather to a fraction of the potential maximum retention. The initial abstraction in inches is calculated using Equation 6-12.

$$Ia = 0.1\,[(1000/CN) - 10] \qquad \text{(Eq. 6-12)}$$

Table 6-9. NRCS Curve Numbers for Pre-Development Thunderstorms Conditions (ARC I)

Fully Developed Urban Areas (vegetation established)[1]	Treatment	Hydrologic Condition	% I	Pre-Development CN			
				HSG A	HSG B	HSG C	HSG D
Open space (lawns, parks, golf courses, cemeteries, etc.):							
Poor condition (grass cover < 50%)	-----	-----	---	47	61	72	77
Fair condition (grass cover 50% to 75%)	-----	-----	---	29	48	61	69
Good condition (grass cover > 75%)	-----	-----	---	21	40	54	63
Impervious areas:							
Paved parking lots, roofs, driveways, etc. (excluding right-of-way	-----	-----	---	95	95	95	95
Streets and roads:							
Paved; curbs and storm sewers (excluding right-of-way)	-----	-----	---	95	95	95	95
Paved; open ditches (including right-of-way)	-----	-----	---	67	77	83	85
Gravel (including right-of-way)	-----	-----	---	57	70	77	81
Dirt (including right-of-way)	-----	-----	---	52	66	74	77
Western desert urban areas:							
Natural desert landscaping (pervious areas only)	-----	-----	---	42	58	70	75
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)	-----	-----	---	91	91	91	91
Developing Urban Areas[1]	**Treatment[2]**	**Hydrologic Condition[3]**	**% I**	**HSG A**	**HSG B**	**HSG C**	**HSG D**
Newly graded areas (pervious areas only, no vegetation)	-----	-----	---	58	72	81	87
Cultivated Agricultural Lands[1]	**Treatment**	**Hydrologic Condition**	**% I**	**HSG A**	**HSG B**	**HSG C**	**HSG D**
Fallow	Bare soil	-----	---	58	72	81	87
	Crop residue cover (CR)	Poor	---	57	70	79	85
		Good	---	54	67	75	79
Row crops	Straight row (SR)	Poor	---	52	64	75	81
		Good	---	46	60	70	77
	SR + CR	Poor	---	51	63	74	79
		Good	---	43	56	66	70
	Contoured (C)	Poor	---	49	61	69	75
		Good	---	44	56	66	72
	C + CR	Poor	---	48	60	67	74
		Good	---	43	54	64	70
	Contoured & terraced (C&T)	Poor	---	45	54	63	66
		Good	---	41	51	60	64
	C&T+ CR	Poor	---	44	53	61	64
		Good	---	40	49	58	63
Small grain	SR	Poor	---	44	57	69	75
		Good	---	42	56	67	74
	SR + CR	Poor	---	43	56	67	72
		Good	---	39	52	63	69
	C	Poor	---	42	54	66	70
		Good	---	40	53	64	69
	C + CR Poor	Poor	---	41	53	64	69
		Good	---	39	52	63	67
	C&T	Poor	---	40	52	61	66
		Good	---	38	49	60	64
	C&T+ CR	Poor	---	39	51	60	64
		Good	---	37	48	58	63
Close-seeded or broadcast legumes or rotation meadow	SR	Poor	---	45	58	70	77
		Good	---	37	52	64	70
	C	Poor	---	43	56	67	70
		Good	---	34	48	60	67
	C&T	Poor	---	42	53	63	67
		Good	---	30	46	57	63

Table 6-9. (continued)

Other Agricultural Lands[1]	Treatment	Hydrologic Condition	% I	HSG A	HSG B	HSG C	HSG D
Pasture, grassland, or range—continuous forage for grazing[4]	-----	Poor	---	47	61	72	77
	-----	Fair	---	29	48	61	69
	-----	Good	---	21	40	54	63
Meadow—continuous grass, protected from grazing and generally mowed for hay	-----	-----	---	15	37	51	60
Brush—brush-weed-grass mixture with brush the major element[5]	-----	Poor	---	28	46	58	67
	-----	Fair	---	18	35	49	58
	-----	Good	---	15	28	44	53
Woods—grass combination (orchard or tree farm)[6]	-----	Poor	---	36	53	66	72
	-----	Fair	---	24	44	57	66
	-----	Good	---	17	37	52	61
Woods[7]	-----	Poor	---	26	45	58	67
	-----	Fair	---	19	39	53	61
	-----	Good	---	15	34	49	58
Farmsteads—buildings, lanes, driveways, and surrounding lots	-----	-----	---	38	54	66	72
Arid and Semi-arid Rangelands[1]	**Treatment**	**Hydrologic Condition[8]**	**% I**	**HSG A**	**HSG B**	**HSG C**	**HSG D**
Herbaceous—mixture of grass, weeds, and low-growing brush, with brush the minor element	-----	Poor	---	-----	63	74	85
	-----	Fair	---	-----	51	64	77
	-----	Good	---	-----	41	54	70
Oak-aspen—mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple, and other brush	-----	Poor	---	-----	45	54	61
	-----	Fair	---	-----	28	36	42
	-----	Good	---	-----	15	23	28
Pinyon-juniper—pinyon, juniper, or both; grass understory	-----	Poor	---	-----	56	70	77
	-----	Fair	---	-----	37	53	63
	-----	Good	---	-----	23	40	51
Sagebrush with grass understory	-----	Poor	---	-----	46	63	70
	-----	Fair	---	-----	30	42	49
	-----	Good	---	-----	18	27	34
Desert shrub—major plants include saltbush, greasewood, creosotebush, blackbrush, bursage, palo verde, mesquite, and cactus	-----	Poor	---	42	58	70	75
	-----	Fair	---	34	52	64	72
	-----	Good	---	29	47	61	69

1. Average runoff condition, and Ia = 0.1S.

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

3. Hydraulic condition is based on combination 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 ≥ 20%), and (e) degree of surface roughness. Poor: Factors impair infiltration and tend to increase runoff. Good: Factors encourage average and better than average infiltration and tend to decrease runoff.

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. CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.

7. 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.

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

Table 6-10. NRCS Curve Numbers for Frontal Storms & Thunderstorms for Developed Conditions (ARCII)

Fully Developed Urban Areas (vegetation established)[1]	Treatment	Hydrologic Condition	% I	Pre-Development CN			
				HSG A	HSG B	HSG C	HSG D
Open space (lawns, parks, golf courses, cemeteries, etc.):							
Poor condition (grass cover < 50%)	-----	-----	---	68	79	86	89
Fair condition (grass cover 50% to 75%)	-----	-----	---	49	69	79	84
Good condition (grass cover > 75%)	-----	-----	---	39	61	74	80
Impervious areas:							
Paved parking lots, roofs, driveways, etc. (excluding right-of-way	-----	-----	---	98	98	98	98
Streets and roads:							
Paved; curbs and storm sewers (excluding 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)	-----	-----	---	63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)	-----	-----	---	96	96	96	96
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[1]	**Treatment[2]**	**Hydrologic Condition[3]**	**% I**	**HSG A**	**HSG B**	**HSG C**	**HSG D**
Newly graded areas (pervious areas only, no vegetation)	-----	-----	---	77	86	91	94
Cultivated Agricultural Lands[1]	**Treatment**	**Hydrologic Condition**	**% I**	**HSG A**	**HSG B**	**HSG C**	**HSG 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 & 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
		Good	---	61	73	81	84
	C + CR Poor	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

Table 6-10. (continued)

Other Agricultural Lands[1]	Treatment	Hydrologic Condition	% I	HSG A	HSG B	HSG C	HSG D
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	-----	-----	---	30	58	71	78
Brush—brush-weed-grass mixture with brush the major element[5]	-----	Poor	---	48	67	77	83
	-----	Fair	---	35	56	70	77
	-----	Good	---	30	48	65	73
Woods—grass combination (orchard or tree farm)[6]	-----	Poor	---	57	73	82	86
	-----	Fair	---	43	65	76	82
	-----	Good	---	32	58	72	79
Woods[7]	-----	Poor	---	45	66	77	83
	-----	Fair	---	36	60	73	79
	-----	Good	---	30	55	70	77
Farmsteads—buildings, lanes, driveways, and surrounding lots	-----	-----	---	59	74	82	86
Arid and Semi-arid Rangelands[1]	**Treatment**	**Hydrologic Condition[8]**	**% I**	**HSG A**	**HSG B**	**HSG C**	**HSG 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
Sagebrush with grass understory	-----	Poor	---	-----	67	80	85
	-----	Fair	---	-----	51	63	70
	-----	Good	---	-----	35	47	55
Desert shrub—major plants include saltbush, greasewood, creosotebush, blackbrush, bursage, palo verde, mesquite, and cactus	-----	Poor	---	63	77	85	88
	-----	Fair	---	55	72	81	86
	-----	Good	---	49	68	79	84

: Ia = 0.1 S

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

3. Hydraulic condition is based on combination 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 ≥ 20%), and (e) degree of surface roughness. Poor: Factors impair infiltration and tend to increase runoff. Good: Factors encourage average and better than average infiltration and tend to decrease runoff.

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 occasional

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

6. CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods

7. 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.

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

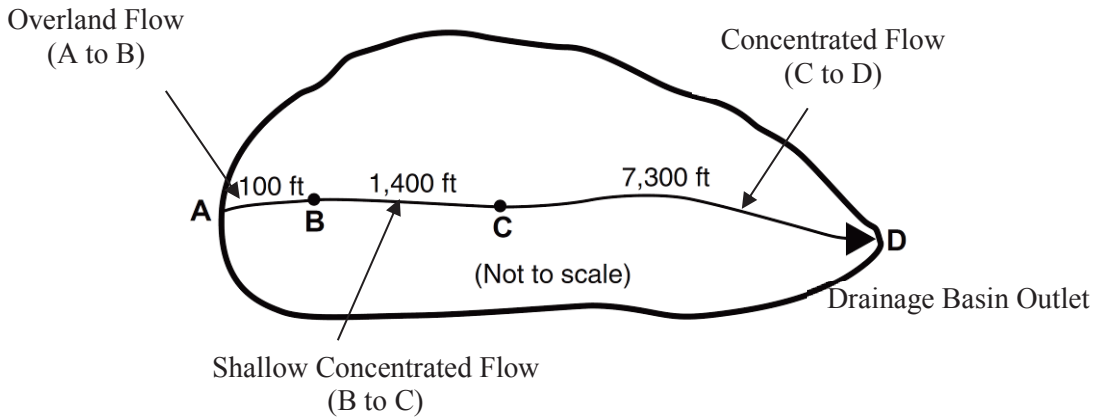
4.6 Lag Time

While the NRCS curve numbers are used to calculate the volume of runoff and magnitude of losses, to transform the volume of runoff into a hydrograph using the NRCS dimensionless unit hydrograph, the lag time must be specified. The lag time is defined as the time from the centroid of the rainfall distribution of a storm to the peak discharge produced by the watershed. For this Manual, the lag time is defined as a fraction of the time of concentration (t_c) as shown in Equation 6-13.

$$t_{lag} = 0.6 \cdot t_c \qquad \text{(Eq. 6-13)}$$

The time of concentration is calculated following the guidance provided in TR-55 (NRCS 2005) by dividing the flow path into multiple segments. These segments can generally be categorized as overland flow, shallow concentrated flow and concentrated or channelized flow. For each of the flow segments, the estimated 2-year flow or the "low flow" should be used to calculate velocity.

Figure 6-1. Flow Segments for Time of Concentration



The Time of Concentration is the sum of overland flow time and the t_t values for the various consecutive flow segments:

$$t_c = t_i + t_{t1} + t_{t2} + t_{t3} \ldots t_{tm} \qquad \text{(Eq. 6-14)}$$

Where:

t_c = time of concentration (hr)

t_i = overland (initial) flow time (hr)

t_{tm} = travel time for each flow segment (hr)

m = number of flow segments

4.6.1 Overland Flow Time for NRCS Method

The overland flow time represents the time for runoff to travel over the upper most portion of a drainage basin before there is enough flow to become concentrated into identifiable flow paths. This travel time can be estimated using the slope of the ground and the type of ground cover. Overland flow lengths should not exceed 100 feet for urban areas and 300 feet for undeveloped areas.

$$T_i = 0.007(n \cdot L)^{0.8} / (P_2)^{0.5} S^{0.4} \quad \text{(Eq. 6-15)}$$

Where:

T_i = overland flow time (hr)

n = Manning's roughness coefficient

L = flow length (ft)

P_2 = 2-year, 24-hour rainfall (in)

S = slope of hydraulic grade line (ft/ft)

Typical roughness coefficients for the overland flow portion of the drainage basin are provided in Table 6-11. Be aware that Manning's roughness coefficients for overland flow are different from Manning's n values for open channels and conduits. Manning's n values for channels and conduits should not be used for overland flow.

Table 6-11. Roughness Coefficients (Manning's n) for NRCS Overland Flow

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

4. [1]The values are a composite of information compiled by Engman (1986).

5. [2]Includes species such as weeping lovegrass, bluegrass, buffalograss, blue gramma grass, native grass mixtures.

6. [3]When selecting n, consider cover to a height of about 0.1 feet. This is the only part of the plant cover that will obstruct sheet flow.

4.6.2 Shallow Concentrated Flow

Flow that travels in defined flow paths, small shallow channels in undeveloped basins or in swales or gutters in developed basins normally has higher velocities than overland flow. Its travel time can be estimated by dividing its flow length by its average velocity. Average velocities for shallow concentrated flow can be estimated from Figure 6-25.

4.6.3 Concentrated Flow

Once flow enters a storm sewer or open channel, it becomes concentrated and its travel time can also be estimated by dividing its travel length into segments. Travel time is the ratio of flow length to flow velocity.

$$T_t = L / (3600 \cdot V) \quad \text{(Eq. 6-16)}$$

Where:

T_t = travel time (hr)

L = flow length (ft)

V = velocity (ft/s)

3,600 = conversion factor from seconds to hours

The average velocity in concentrated flow segments can be estimated by Manning's equation:

$$V = 1.49\ R_h^{2/3}\ S^{1/2} / n \quad \text{(Eq. 6-17)}$$

Where:

V = average velocity (ft/s)

A_w = Area of cross section conveying flow (ft^2)

R_h = hydraulic radius (ft) equal to A_w/P_w

P_w = wetted perimeter (ft)

S = friction slope/slope of energy grade line (typically assumed to be equivalent to channel bottom slope for uniform flow) (ft/ft)

n = Manning's roughness coefficient for open channel flow

As a general rule, and when sufficiently detailed development plans are not available, the post-development time of concentration can be estimated to be 75% of the pre-development value within the areas of the basin that are to be urbanized.

4.7 Peak Flow Estimation

For preliminary design purposes or for estimating allowable release rates, peak flows may be estimated using the NRCS method by calculating the parameters for curve number and t_c as described above. The following equations provide an estimate of peak flows for a given return period:

$$q = q_p \cdot A \cdot Q \quad \text{(Eq. 6-18)}$$

$$q_p = 484 \cdot A \cdot Q / t_p \quad \text{(Eq. 6-19)}$$

$$Q = (P - 0.1 \cdot S)^2 / (P + (1\text{-}0.9 \cdot S)) \quad \text{(Eq. 6-20)}$$

$$S = 1{,}000/CN - 10 \text{ for } I_a = 0.1 \cdot S \quad \text{(Eq. 6-21)}$$

$$t_p = D/2 + 0.06\ t_c = 0.67\ t_c\ \text{, where } (D = 0.133\ t_c) \quad \text{(Eq. 6-22)}$$

Where:

q = peak discharge (cfs)

q_p = unit peak discharge in (cfs/ mi^2)

A = drainage basin area (mi^2)

Q = direct runoff (in)

P = rainfall depth for storm return period and duration (in)

S = potential maximum retention after runoff begins (in)

CN = composite curve number for the ARC applied

I_a = initial abstraction as a fraction of S (in)

t_p = time to peak discharge (hr)

t_c = time of concentration (hr)

Limitations of the peak flow estimation method are:

- The drainage basin must be hydrologically homogeneous (i.e., describable by one curve number). Land use, soils and cover must be distributed uniformly throughout the drainage basin.
- The drainage basin must have only one main stream or, if more than one, the branches must have similar t_c values.
- There are no effects due to reservoir routing.
- The weighted curve number must be greater than 40.

5.0 EPA Stormwater Management Model (EPA SWMM)

EPA's SWMM 5 is a computer model that is used to generate surface runoff hydrographs from sub-basins and then route and combine these hydrographs. The purpose of the discussion of SWMM in this chapter is to provide general background on the use of the model to perform more complex stormwater runoff calculations using SWMM. Complete details about the use of the model, specifics of data format and program execution is provided in the Users' Manual for SWMM 5.0. Software, Users' Manual and other information about EPA's SWMM 5.0 may be downloaded from the EPA website. The following section includes excerpts from the SWMM 5.0 User's Manual (EPA 2008) that describes capabilities and primary inputs for the model.

5.1 Model Overview

The EPA Stormwater Management Model User's Manual, Version 5.0 (EPA 2008) provides the following overview of SWMM and its hydrologic and hydraulic modeling capabilities:

[SWMM] *is a dynamic rainfall-runoff simulation model used for single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas. The runoff component of SWMM operates on a collection of subcatchment areas that receive precipitation and generate runoff and pollutant loads. The routing portion of SWMM transports this runoff through a system of pipes, channels, storage/treatment devices, pumps, and regulators. SWMM tracks the quantity and quality of runoff generated within each subcatchment, and the flow rate,*

flow depth, and quality of water in each pipe and channel during a simulation period comprised of multiple time steps.

SWMM accounts for various hydrologic processes that produce runoff from urban areas. These include:[2]

- *Time-varying rainfall,*
- *Evaporation of standing surface water,*
- *Snow accumulation and melting,*
- *Rainfall interception from depression storage,*
- *Infiltration of rainfall into unsaturated soil layers,*
- *Percolation of infiltrated water into groundwater layers,*
- *Interflow between groundwater and the drainage system,*
- *Nonlinear reservoir routing of overland flow.*

Spatial variability in all of these processes is achieved by dividing a study area into a collection of smaller, homogeneous subcatchment areas, each containing its own fraction of pervious and impervious sub-areas. Overland flow can be routed between sub-areas, between subcatchments, or between entry points of a drainage system.

SWMM also contains a flexible set of hydraulic modeling capabilities used to route runoff and external inflows through the drainage system network of pipes, channels, storage/treatment units and diversion structures. These include the ability to:

- *Handle networks of unlimited size*
- *Use a wide variety of standard closed and open conduit shapes as well as natural channels*
- *Model special elements such as storage/treatment units, flow dividers, pumps, weirs, and orifices…*
- *Model various flow regimes, such as backwater, surcharging, reverse flow, and surface [in dynamic flow mode].*

Typical model elements for an urban drainage SWMM model include the following:

Rain Gages

Rain Gages supply precipitation data for one or more subcatchment areas in a study region. The rainfall data can be either a user-defined time series or come from an external file. Several different popular rainfall file formats currently in use are supported, as well as a standard user-defined format.

The principal input properties of rain gages include:

- *Rainfall data type (e.g., intensity, volume, or cumulative volume),*
- *Recording time interval (e.g., hourly, 15-minute, etc.),*
- *Source of rainfall data (input time series or external file),*
- *Name of rainfall data source.*

[2] For most urban drainage applications in Colorado Springs, the hydrologic processes that will generally modeled are those related to rainfall-runoff, hydraulic conveyance elements (channels and pipes) and detention routing. Other modeling capabilities including snowmelt hydrology, surface water/groundwater interactions, and water quality algorithms are usually applied only in special cases by experienced users.

Subcatchments (i.e., sub-basins):

Subcatchments are hydrologic units of land whose topography and drainage system elements direct surface runoff to a single discharge point. The user is responsible for dividing a study area into an appropriate number of subcatchments, and for identifying the outlet point of each subcatchment. Discharge outlet points can be either nodes of the drainage system or other subcatchments.

Subcatchments can be divided into pervious and impervious subareas. Surface runoff can infiltrate into the upper soil zone of the pervious subarea, but not through the impervious subarea. Impervious areas are themselves divided into two subareas - one that contains depression storage and another that does not. Runoff flow from one subarea in a subcatchment can be routed to the other subarea, or both subareas can drain to the subcatchment outlet.

Principal input parameters for subcatchments include:

- [Infiltration method and associated parameters (Horton or Green Ampt—curve number algorithm is inconsistent with TR-55 and should not be used in Colorado Springs)],
- *Assigned rain gage,*
- *Outlet node or subcatchment,*
- *Assigned land uses,*
- *Tributary surface area,*
- *Imperviousness,*
- *Slope,*
- *Characteristic width of overland flow* [additional information provided below],
- *Manning's n for overland flow on both pervious and impervious areas* [see Table 6-11],
- *Depression storage in both pervious and impervious areas, and*
- *Percent of impervious area with no depression storage.*

Junction Nodes

Junctions are drainage system nodes where links join together. Physically, they can represent the confluence of natural surface channels, manholes in a sewer system, or pipe connection fittings. External inflows can enter the system at junctions. Excess water at a junction can become partially pressurized while connecting conduits are surcharged and can either be lost from the system or be allowed to pond atop the junction and subsequently drain back into the junction.

The principal input parameters for a junction are:

- *Invert elevation,*
- *Height to ground surface,*
- *Ponded surface area when flooded (optional),*
- *External inflow data (optional).*

Outfall Nodes

Outfalls are terminal nodes of the drainage system used to define final downstream boundaries under Dynamic Wave flow routing. For other types of flow routing they behave as a junction. Only a single link can be connected to an outfall node.

The boundary conditions at an outfall can be described by any one of the following stage relationships:

- *The critical or normal flow depth in the connecting conduit,*
- *A fixed stage elevation…*
- *A user-defined time series of stage versus time.*

The principal input parameters for outfalls include:

- *Invert elevation,*
- *Boundary condition type and stage description,*
- *Presence of a flap gate to prevent backflow through the outfall.*

Storage Units

Storage units are drainage system nodes that provide storage volume. Physically they could represent storage facilities as small as a catch basin or as large as a lake. The volumetric properties of a storage unit are described by a function or table of surface area versus height.

The principal input parameters for storage units include:

- *Invert elevation,*
- *Maximum depth,*
- *Depth-surface area data,*
- *Evaporation potential,*
- *Ponded surface area when flooded (optional),*
- *External inflow data (optional).*

Conduits

Conduits are pipes or channels that move water from one node to another in the conveyance system. Their cross-sectional shapes can be selected from a variety of standard open and closed geometries. Most open channels can be represented with a rectangular, trapezoidal, or user-defined irregular cross-section shape.

Outlets

Most commonly in SWMM, outflow from a storage unit (detention pond) can be defined by orifice and/or weir flow that can be determined from the geometry of the outlet structure. When special head-discharge relationships exist that cannot be easily modeled with weirs and/or orifices, SWMM provides an option for the user to define an outlet rating curve. The following describes the outlet option in SWMM from the User's Manual:

Outlets are flow control devices that are typically used to control outflows from storage units. They are used to model special head-discharge relationships that cannot be characterized by pumps, orifices, or weirs. Outlets are internally represented in SWMM as a link connecting two nodes. An outlet can also have a flap gate that restricts flow to only one direction. A user-defined rating curve determines an outlet's discharge flow as a function of the head difference across it. Control Rules can be used to dynamically adjust this flow when certain conditions exist.

SWMM also has options for flow dividers, pumps, flap gates with control rules and other features that typically are not used in most urban drainage applications.

5.1.1 Surface Flows and Routing Features

The SWMM model is different from other hydrologic methods, which generally treat a sub-basin as a single unit with associated losses (infiltration). SWMM on the other hand conceptualizes a sub-basin as a rectangle consisting of two planes, one pervious and the other impervious and uses a kinematic wave conceptualization of overland flow to generate flow from these two planes, as shown in Figures 6-2 and 6-3 below.

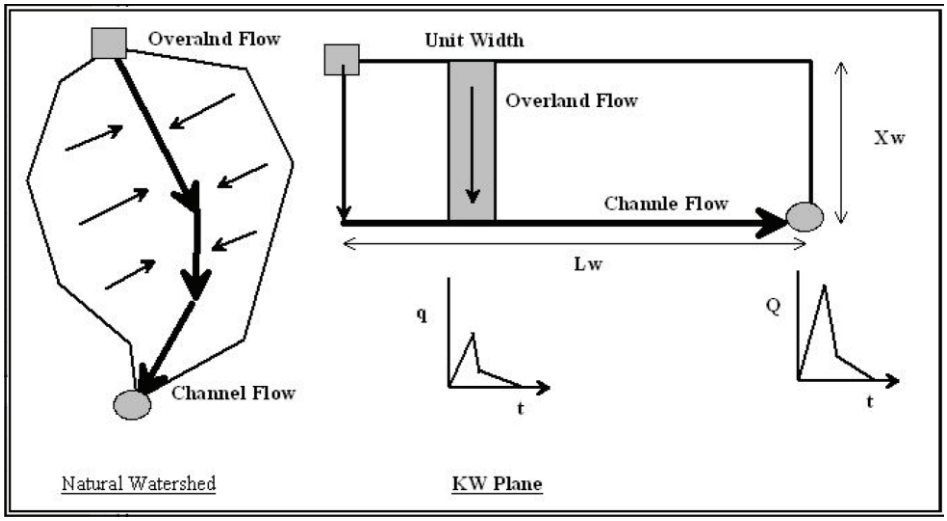
SWMM represents a watershed by an aggregate of idealized runoff planes, channels, gutters, pipes and specialized units such as storage nodes, outlets, pumps, etc. The program can accept rainfall hyetographs and make a step-by-step accounting of rainfall infiltration losses in pervious areas, surface retention, overland flow, and gutter flow leading to the calculation of hydrographs. After SWMM calculates hydrographs from a number of sub-basins, the resulting hydrographs from these sub-basins can be combined and routed through a series of links (i.e., channels, gutters, pipes, dummy links, etc.) and nodes (i.e., junctures, storage, diversion, etc.) to compute the resultant hydrographs at any number of design points within the watershed.

Stormwater runoff hydrographs generated by SWMM using either the Horton or Green-Ampt rainfall/runoff methods can be routed through a system of stormwater conveyances, diversions, storage facilities, and other elements of a complex urban watershed. It is up to the model user to demonstrate compatibility between SWMM model results and model results that would be achieved using the NRCS curve number procedures, both in terms of rates and volumes. Under no circumstances shall the curve number method in SWMM be used because it is not an accurate representation of the NRCS curve number loss method as published in TR-55 and implemented in HEC-HMS.

Figure 6-2 illustrates how a single kinematic flow plane can be used to represent a portion of a watershed with overland flow occurring over a specified overland flow length (X_w) and being collected and conveyed to the sub-basin outlet by channel or gutter flow with a width of L_w. The choice of L_w, which also defines X_w because the plane is conceptualized as a rectangle, is one of the most important (and sensitive) parameters in a SWMM model. Another key parameter for modeling overland flow in SWMM is the Manning's n value for overland flow. These values are provided in Table 6-11 and should not be confused with Manning's n values for open channel flow, which are typically considerably lower than Manning's n values for overland flow.

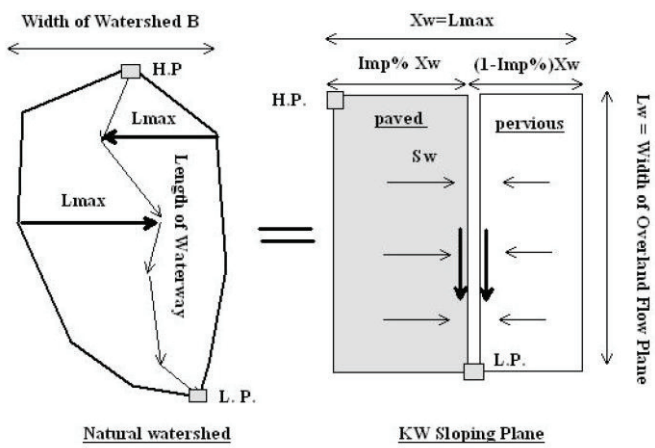
Figure 6-3 illustrates how this would be applied to a sub-basin with both pervious and impervious kinematic wave planes.

Figure 6-2. Conceptual SWMM Watershed Schematic



Source: Urban Watersheds Research Institute (UWRI), Stormwater Planning and Design Using EPA SWMM Computer Model, Nov. 2011.

Figure 6-3. SWMM Kinematic Overland Flow Conceptualization of Watershed with Pervious and Impervious Areas



Empirical formulas:
(1) Lw= 2.2 L (2) Lw=1.67 L (3) Lw=2.0 L (4) Lw=A/Lmax

Source: Urban Watersheds Research Institute (UWRI), Stormwater Planning and Design Using EPA SWMM Computer Model, Nov. 2011.

To aid in selection of appropriate values for L_W, a number of relationships have been developed between sub-basin geometric characteristics and the ratio of L_W/L (conceptualized overland flow length [L_w] divided by the actual length of the watershed [L]). These empirical relationships are presented in Table 6-12. Shape factors can be calculated based on general watershed shapes and measured watershed characteristics (i.e. area, waterway length), and the value of L_w to enter into the SWMM model can be calculated by multiplying the shape factor by the actual waterway length (L). The shape factors in Table 6-12 are applicable only up to shape factors of approximately 4.0.

Table 6-12. Kinematic Wave Shape Factors ($X=A/L^2 \leq 4$)

Shapes	Lw/L
Asymmetric	$(1.5-Z)\,[2.286(A/L^2) - 0.286(A/L^2)^2]$
Central Channel (Z=0.5)	$2.286(A/L^2) - 0.286(A/L^2)^2$
Rectangle ($A/L^2=B/L$)	$2.286(B/L) - 0.286(B/L)^2$
Square ($A/L^2 = 1$)	2.0
Side Channel (Z=1.0)	$1.143(A/L^2) - 0.143(A/L^2)^2$
Rectangle ($A/L^2=B/L$)	$1.143(B/L) - 0.143(B/L)^2$
Square ($A/L^2=1$)	1.0
Asymptotic Conditions	
A close to zero ($A/L^2 \sim 0$)	~ 0
A very large ($A/L^2 > 4$)	~ 4.57

In setting up the SWMM model, it is critical that overflow links for storm sewers and diversion junctions be provided in the model. The combination of storm sewers and overflow paths allows the user to model flows when pipes and/or smaller channels do not have the capacity to convey higher flows. Under these conditions, the excess flows are diverted to the overflow channels (links), avoiding unrealistic "choking" of the flow that can lead to errors in the calculated peak flow values downstream are prevented.

There are several types of conveyance elements that one can select from a menu in SWMM. One element that is now available, that was not available in older versions, is a user-defined irregular channel cross-section, similar to the way cross-sections 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 Manual (EPA 2005).

5.1.2 Flow Routing Method of Choice

The kinematic wave routing method is the recommended routing option in SWMM for planning purposes. Dynamic wave routing for most projects is not necessary, does not improve the accuracy of the runoff estimates and can be much more difficult to implement because it requires much information to describe the entire flow routing system in minute detail. In addition, it has tendencies to become unstable when modeling some of the more complex elements and/or junctions. When planning for growth, much of the required detail may not even be available (e.g., location of all drop structures and their crest and toe elevations for which a node has to be defined in the model). With dynamic routing, setting up overflow links and related nodes is much more complicated and exacting.

The use of dynamic wave routing is appropriate when evaluating complex existing elements of a larger system. It is an option that can also offer some advantages in final design and its evaluation because it provides hydraulic grade lines and accounts for backwater effects.

5.2 Application of SWMM

SWMM is an acceptable model for application provided that the following conditions are satisfied:

- The curve number option in the EPA SWMM model must not be used.

- If SWMM is used, it is recommended that the user follow the guidance in the Runoff chapter (and Volume 3) of the UDFCD Manual for selection of proper infiltration parameters.
- Regardless of the infiltration method used, it is incumbent on the design engineer to demonstrate reasonable equivalency between SWMM results and those that would be obtained from the standard NRCS procedure in terms of runoff rates and volumes. Justification must be provided for why the SWMM model is being used.
- The SWMM model should not be applied by inexperienced users.
- Proprietary versions of SWMM for which there is no valid software license to conduct a detailed review and run of the model are not permitted.

For additional guidance, refer to the Runoff chapter and Volume 3 of the UDFCD Manual and/or the EPA SWMM manual available on EPA's website.

6.0 Sub-basin Delineation and Hydrograph Routing

Rainfall/runoff models such as the NRCS dimensionless unit hydrograph method within the HEC-HMS program and EPA SWMM require that a systematic approach be used to delineate and combine sub-basins within the larger drainage basin being evaluated. Sub-basins should be about 130 acres in size and be delineated to represent areas of the basin that are relatively homogeneous. Besides topography, features that might be used to identify sub-basins are land uses (existing and future), soil types and land cover. Identifying locations or design points where flow information is important may also determine sub-basin delineation.

Hydrographs from each sub-area must be routed and combined to determine the hydrograph for the entire drainage area contributing to design points. Sub-basins are joined by routing elements that may have a wide variety of characteristics, but are typically open channels. Hydrograph routing must account for the effects of flow traveling in channels, through storage areas and other features, such as diversion channels that change the hydrograph. The designer should identify sub-basins and routing elements prior to coding a model so that element numbers and descriptions are systematic and help in the interpretation of model results.

6.1 Channel Routing

The Kinematic Wave Channel Routing Method or the Muskingum-Cunge Method are the preferred methods, although other methods may be acceptable upon approval on a case-by-case basis. Where appreciable hydrograph attenuation is anticipated due to storage effects along a reach, a method that explicitly accounts for channel storage effects, such as the Modified-Puls method, may also be applied.

6.1.1 Kinematic Wave Channel Routing

The Kinematic Wave Channel Routing Method is used to route an upstream inflow hydrograph through a reach with known geometric characteristics. Theoretically, a flood wave routed by the Kinematic Wave Channel Routing Method is translated, but not attenuated, through a reach (although a degree of attenuation is introduced by the finite difference solution to the governing equations). The lack of significant peak attenuation during hydrograph translation is a fairly common characteristic of urban conveyances. Table 6-13 summarizes input parameters required for the Kinematic Wave Channel Routing Method. Manning's roughness values should be selected in accordance with the Open Channels chapter.

Table 6-13. Kinematic Wave Channel Routing Method Inputs

Input Parameter	Note
Length (ft)	Determined as the actual length of the flow path along thalweg.
Slope (ft/ft)	Calculate as change in elevation divided by channel length.
Manning's n	Determine according to Open Channels Chapter of Manual or specific guidance from the model's User Manual. The Manning's n values used for channel flow are different from Manning's n values which are used for overland flow (i.e., Table 6-10, which are typically an order of magnitude or so higher than Manning's n values for channelized flow.)
Shape	Trapezoid, deep or circular. Trapezoidal can also be used for rectangular and triangular cross-sections by specifying appropriate side slopes and bottom width. Use deep channel when flow depth ≈ channel width. Some programs have an "irregular channel" option.
Width or Diameter (ft)	Representative bottom width and diameter for circular conveyances.
Side Slope (H:V) (ft/ft)	For trapezoidal or triangular channels only.
Minimum Number of Routing Increments	The minimum number of steps is related to the finite difference solution of the governing equations. The minimum number of routing increments is automatically determined by the program but optionally can be entered by the user (not recommended by City). In HEC-HMS, this input parameter is the number of "subreaches."

6.1.2 Muskingum-Cunge Channel Routing

When more natural channel characteristics are present and storage in the channel is available to attenuate flows, the Muskingum-Cunge method may be more appropriate, with input parameters shown in Table 6-14.

Table 6-14. Muskingum-Cunge Channel Routing Method Inputs

Input Parameter	Note
Length (ft)	Determined as the actual length of the flow path along thalweg.
Slope (ft/ft)	Calculate as change in elevation divided by channel length.
Manning's n	Determine according to Open Channels Chapter of Manual or specific guidance from the model's User Manual.
Shape	Trapezoid, deep or circular. Trapezoidal can also be used for rectangular and triangular cross-sections by specifying appropriate side slopes and bottom width. Use deep channel when flow depth ≈ channel width. Typical inputs for trapezoidal channels include side slopes and bottom width.
Channel Cross Section	Define channel cross section using eight-point method or standard shape.
Side Slope (H:V) (ft/ft)	For trapezoidal and triangular channels only.
Minimum Number of Routing Increments	The minimum number of steps is related to the finite difference solution of the governing equations. The minimum number of routing increments is automatically determined by the program but optionally can be entered by the user (not recommended).

6.2 Reservoir Routing

For watersheds with significant detention structures, the effects of routing hydrographs through facilities can have important implications on the peak flow rates and their timing from sub-watersheds. Hydrologic modeling and analysis must account for the effects of detention by performing reservoir routing calculations. The criteria and methods for reservoir routing are presented in the Storage Chapter of this Manual and are also documented in the user's manuals and technical reference manuals for many of the software packages. Options for routing using common methods are included in HEC-HMS and SWMM and many other commercially available hydrology software packages.

7.0 Runoff Reduction Methods

Conventional methods for evaluating increased runoff volume and peak flows associated with urbanization make certain assumptions about the relationship between impervious surfaces and their effect on runoff. A primary assumption of many conventional methods is that the impervious surfaces are directly connected to the drainage features receiving the runoff. In reality, this connection is not always so direct, and adjusting land use planning and design practices to "disconnect" impervious areas (i.e. route flows from impervious areas to pervious areas rather than the gutter and street inlets), can reduce the rate and volume of runoff downstream. Many of the same practices that have been developed for improving water quality are also beneficial for reducing runoff volumes and peak flows. These practices can generally be referred to as Best Management Practices (BMPs), Low Impact Development (LID) and Green Infrastructure (GI) approaches. The effects of urbanization, the selection of BMPs, the implementation of LID approaches and their potential for reducing runoff are discussed in detail in Volume 2 in this Manual. Key concepts associated with these practices are briefly summarized below with regard to their implications for estimating runoff.

7.1 Four Step Process

UDFCD has long recommended a "Four Step Process" for receiving water protection that focuses on reducing runoff volumes, treating the water quality capture volume (WQCV), stabilizing drainageways, and implementing long-term source controls. The Four Step Process pertains to management of smaller, frequently occurring events, as opposed to larger storms for which drainage and flood control infrastructure are sized. The Four Step Process is summarized as follows:

1. Step 1: Reduce runoff by disconnecting impervious area, eliminating "unnecessary" impervious area and encouraging infiltration into soils that are suitable.
2. Step 2: Treat and slowly release the WQCV.
3. Step 3: Stabilize stream channels.
4. Step 4: Implement source controls.

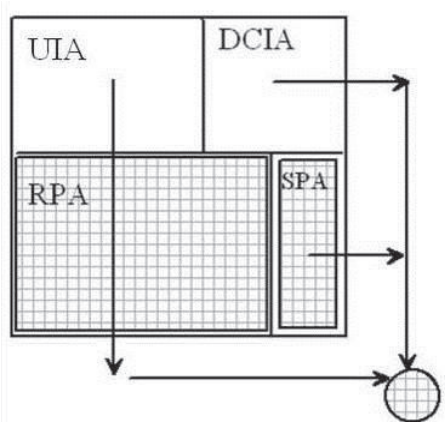
Implementation of these four steps helps to achieve stormwater permit requirements. Added benefits of implementing the complete process can include improved site aesthetics through functional landscaping features that also provide water quality benefits. Additionally, runoff reduction can decrease required storage volumes, increasing developable land and reduce the size of downstream facilities. A detailed description of the Four Step Process is provided in Volume 2 of this Manual, providing BMP selection tools and quantitative procedures for completing these steps.

There are two primary approaches to reducing runoff volume and peak flows provided in this Manual. The first is to represent runoff reduction practices in the standard methods by converting the effects of these practices into a reduced value for imperviousness on a basin or sub-basin level. The second is to more directly represent the physical impacts of the BMPs and LID practices through modeling each of the elements at a sub-basin level. There is a significant difference in the level of detail and expertise required in the application of these two approaches. Most situations can be reasonably addressed through the application of an adjusted value for imperviousness, or "effective imperviousness".

7.2 Effective Imperviousness

Runoff calculations typically assume that imperviousness in a drainage basin is directly connected to the receiving system or that combines impervious runoff factors with pervious factors, creating a composite value. To adequately evaluate methods for runoff reduction practices such as Minimizing Directly Connected Impervious Area (MDCIA), BMPs, and LID, it is necessary to be able to segregate these sources of runoff. Conceptually, the relationship between impervious and pervious areas is shown in Figure 6-4.

Figure 6-4. Land Use Components



Where:

DCIA = directly connected impervious area

UIA = unconnected impervious area

RPA = receiving pervious area

SPA = separate pervious area

Efforts to reduce runoff and plan development projects can be assisted by considering how runoff from each portion of the project site travels to the receiving system.

Master Planning Level

When runoff reduction practices are anticipated for a development project that is in the early stages of planning, runoff reduction benefits can be estimated as described in Volume 2. The Effective imperviousness from Volume 2 can be used to adjust the impervious values applied to the development of runoff coefficients and Curve Numbers.

Site-level

When a more detailed site plan is available that provides sufficient detail for the development plan so that impervious surfaces can be identified, a more precise evaluation of runoff reduction can be estimated in greater detail. Two methods are available for evaluation: 1) SWMM modeling using the cascading plane approach and 2) the UDFCD Imperviousness Reduction Factor (IRF) charts and spreadsheets. Both methods provide guidance on how to account for conveyance-based or storage-based features.

SWMM modeling requires a higher level of expertise and experience and a very detailed representation of each of the BMP or LID features. A detailed description of how to implement this approach is provided in Volume 2.

The IRF approach allows the designer to calculate revised values for imperviousness for each BMP or LID feature and combine them with runoff coefficients for other methods with some flexibility in the level of detail required. A spreadsheet tool (UD-BMP) to provide the accumulated Effective Imperviousness is available to the designer. This tool requires that individual features of each sub-basin, the 1-hour water quality rainfall depth, the minor storm depth and the major storm depth and is described in Volume 2.

7.2.1 Application of Effective Imperviousness

When the details of how a development project may be constructed are not known, the benefits of BMP and LID practices can be approximated so that hydrologic estimates of runoff and infrastructure sizing can be adjusted in anticipation of future implementation.

Once determined, the adjusted values of imperviousness can be applied to any of the methods described in the chapter to calculate revised values for runoff volume and peak flows.

- The NRCS method presented inTR-55 includes procedures for accounting for disconnected impervious area. TR-55 guidance should be used for adjusting HEC-HMS model parameters to account for disconnected impervious area.

- The UDFCD Imperviousness Reduction Factor (IRF) charts and spreadsheets provide another method to account for runoff reduction due to BMPs that provide the WQCV and conveyance-based BMPs (e.g., swales) that promote infiltration. When detailed site characteristics and routing are known, the UDFCD method can be used to calculate an "effective imperviousness" that can then be used to look up revised Rational Method runoff coefficients or curve numbers corresponding to the reduced imperviousness. The IRF method is described in detail in Volume 3 of the UDFCD Manual and in a peer-reviewed paper by Guo et al. (2010).

7.2.2 Effective Imperviousness Spreadsheet

Because most sites will consist of multiple sub-basins, some using the conveyance-based approach and others using the storage-based approach, a spreadsheet capable of applying both approaches to multiple sub-basins to determine overall site effective imperviousness and volume reduction benefits is a useful tool. The UD-BMP workbook has this capability. A full description of the spreadsheet capabilities are provided in Volume 2.

8.0 Estimating Baseflows

8.1 Baseflow Estimates for Gaged Streams

When reliable low-flow stream measurements are available, as they are for many larger drainageways such as Fountain and Monument Creeks, the best method for developing baseflow estimates is to analyze the long-term gage record, using baseflow separation techniques and knowledge of timing of major diversions and other factors related to the administration of water rights to develop a baseflow hydrograph (i.e., a hydrograph of flow versus time, excluding the effects of storm events). This baseflow hydrograph can then be analyzed statistically to determine probabilities of different baseflow levels, as well as seasonal trends. It is typically acceptable to adjust the measured baseflows from the gage nearest the site by multiplying the measured flow by the ratio of watershed area contributing at the point of interest on the stream to the area contributing to the stream gage. The *Fountain Creek Watershed Hydrology Report* (USACE 2006) provides some baseflow data.

When determining baseflow at gaged sites, baseflow separation techniques can be used. Baseflow separation is the process of dividing a hydrograph into direct runoff and baseflow. Several different techniques can be used for baseflow separation, the simplest of which is to draw a straight line on the hydrograph extending from the point of lowest discharge before surface runoff begins across to the point on the receding limb of the hydrograph where it is evident that flows have approached pre-storm baseflow

levels. Other techniques for baseflow separation can be found in many hydrology references and include exponential extension of hydrograph recession to baseflow conditions.

8.2 Baseflow Estimates for Ungaged Streams (Developing Relationships for Baseflow as a Function of Area)

Methods for developing estimates of baseflows for ungaged streams are not widely available and are difficult to develop due to the lack of baseflow data on many smaller drainageways. This section summarizes a method that was recently developed in the Denver metropolitan area that could serve as a method for rough estimates of baseflows in Colorado Springs.

In the case of the UDFCD analysis, regional data were analyzed to develop relationships for baseflow as a function of tributary area using statistical software. These relationships may be used to estimate baseflow in gaged or ungaged areas. UDFCD developed baseflow equations for watersheds in the Denver area using flow data that have been collected at 29 gage sites around the Denver area for over 30 years. This data set was used to characterize baseflow as a function of area using the following steps:

1. The baseflow data set was scrutinized for outlier values. All zero and apparent rainfall affected flows (i.e., higher flow values) were removed.

2. The baseflow data for each gage site was ranked from low to high and Weibul probability distributions were computed.

3. The data were plotted to identify the 95th percentile and lower values of baseflows. Anything above 95th percentile value was set to the 95th percentile value.

4. The "cleaned-up" monthly baseflow data was run through the U.S. Army Corps of Engineers HEC-SSP software. A Bulletin 17B protocol flow frequency analysis was run for each gage. The 1.01- and 2-year monthly baseflows were determined for each gage.

5. All of the computed 1.01-year and 2-year baseflow values for each month were plotted against the gross tributary area of each gage site and a regression equation was determined.

A linear regression equation was developed to represent baseflows for each month and had a high coefficient of regression (R^2 value indicating a good statistical correlation) for the 2-year flows. The 1.01-year regressions did not have consistently acceptable regression coefficients, and for many of the months, regression coefficients were quite low; therefore, these data were not used in baseflow estimation for UDFCD's purposes. The baseflow regression equations for each month take the form of Equation 6-22 below. The coefficients (*K* values) are summarized in Table 6-15.

$$Q = K \cdot A \qquad \text{(Eq. 6-23)}$$

Where:

Q = 2-year baseflow (cfs)—the 2-year baseflow is the peak baseflow that could be expected in any given month on average once every two years, or in other terms, the flow that would have a 50% chance of being exceeded in any given month

K = the linear regression coefficient—unless monthly analysis of baseflows is needed, a coefficient, K, of 0.3 should be applied

A = tributary drainage basin area (mi^2)

Table 6-15. Coefficients (*K*) and Regression Coefficients (R^2) for the 2-Year Baseflows

Month	Coefficient K	R^2
Mar	0.205	0.81
Apr	0.235	0.80
May	0.301	0.94
Jun	0.281	0.90
Jul	0.289	0.91
Aug	0.286	0.85
Sep	0.242	0.88
Oct	0.260	0.87

For general purposes of analysis, the designer should assume a coefficient (*K*) of 0.30 for calculating baseflows, unless there is a need to analyze a specific month (or months). Lower coefficients for winter months are indicative of seasonal precipitation and runoff trends.

It is important to recognize data limits and extrapolations for watersheds significantly larger than the areas in the data set analyzed may not be defensible. For the data set analyzed here, the upper limit of watershed size is approximately 25 square miles. For much larger areas, use of long-term USGS water resources flow gage data would be more appropriate for estimating baseflows.

A similar method could be developed, specific to Colorado Springs, if adequate baseflow data are available; however, the relationships derived based on the Denver data set will at least provide approximations. In all cases, the design engineer should visit the stream during dry weather conditions to evaluate the reasonableness of baseflow estimates.

9.0 Design Flows for Low-Flow Channels

The "low-flow" portion of the channel is most active and most affected by changes in hydrology due to development. Even with effective detention storage facilities upstream of "natural" channel reaches, it is anticipated that increases in flow volumes and frequency will cause channels to become unstable. By stabilizing the low-flow portion of channels, it is anticipated that more costly channel stabilization projects can be avoided. Also, by including a low-flow channel in the design section of constructed natural channels some natural channel functions can be preserved.

9.1 Stabilized Natural Channels

Investigations into flow records and modeling efforts on Jimmy Camp Creek, a 67 square mile tributary to Fountain Creek, have shown that, due to uncertainty in input parameters for rainfall/runoff models and the complex conditions associated with "bankfull" flow conditions, it appears more appropriate to use measured field data rather than rainfall-runoff modeling to estimate natural channel low-flow channel design flows. Typical return periods for bankfull flows in natural streams fall between the 1-year and 2-year event. However, analyses comparing flows calculated from measured bankfull channel dimensions with modeled flows from design storms indicate that the return period of bankfull flows may not be consistent throughout a large drainage basin. No one set of model configurations consistently produced flows approximating the bankfull flows. By applying regression methods to the bankfull data, a relationship between drainage area and bankfull flow has been developed as an alternative to rainfall/runoff modeling. The results of the bankfull data collection and modeling analyses are described

in a technical memorandum titled; ”Low Flow Estimation for Natural Channel Design”, (Matrix Design Group, March 22, 2013).

The measured bankfull values were adjusted to account for variability in the data and to provide a low-flow design value that should more reliably transport sediment loads. The low-flow regression equations for stabilized natural channels are provided as Equation 6-2 below. This approach assumes that runoff from development will be attenuated by passing through detention storage facilities so that flow in the design reach are similar to flows that occurred in the undeveloped basin.

$$Q_{low\text{-}flow} = 103\ DA^{0.4} \qquad \text{(Eq. 6-2)}$$

Where:

$Q_{low\text{-}flow}$ = design low-flow discharge (cfs)

DA = tributary drainage basin area (mi^2)

9.2 Constructed Natural Channels and Constructed Channels

For constructed natural channels or constructed channels where the design is based on fully developed or partially developed condition flows without full attenuation due to detention storage, the 2-year storm event based on developed basin conditions shall be used for design of the low-flow channel.

9.3 Fountain Creek and Monument Creek

To determine the low flow for designs on Fountain Creek and Monument Creek, the long-term gage data should be analyzed for the project reach using standard methods for statistical analysis (e.g., Log Pearson III analysis with a sufficient period of record of good quality data). Based on the frequency analysis of gage data, the 1.3-year flow should be used to size low-flow channel improvements.

If sufficient baseflow data are available, a similar procedure should be used to estimate baseflows through the project reach, including considerations for seasonal variability.

10.0 Design Hydrology Based on Future Development Conditions

10.1 On-site Flow Analysis

Full site development shall be considered when the design engineer selects runoff coefficients or impervious percentage values and performs the hydrologic analyses for on-site areas. Changes in flow patterns and sub-basin boundaries due to site grading and proposed street and roadway locations must be considered. Time of concentration calculations must reflect increased surface flow velocities and velocities associated with proposed runoff conveyance facilities.

10.2 Off-site Flow Analysis

Fully developed conditions shall be considered when the design engineer selects runoff coefficients or impervious percentage values and performs the hydrologic analyses for off-site areas. Where the off-site area is undeveloped, fully developed conditions shall be projected using the best available land use information, current zoning, or approved land use applications. The City shall be consulted to verify all assumptions regarding future development in off-site areas. If information is not available, runoff calculations shall be based on the impervious percentage value presented in Table 6-6.

Where the off-site area is fully or partially developed, the hydrologic analysis shall be based on existing platted land uses, constructed conveyance facilities, and developed topographic characteristics. Consideration of potential benefits related to detention provided in off-site areas depends on the type of detention provided and whether or not the off-site tributary area is part of a major drainageway basin, as discussed previously in this chapter.

11.0 Consideration of Detention Benefits in Off-Site Flow Analysis

11.1 Major Drainageway Basin Distinction

When determining whether on-site detention benefits may be recognized in off-site flow analysis, a distinction is made between systems that are part of the major drainageway basin system (defined as generally greater than 130 acres of tributary area) and for those that are higher upstream in the watershed (generally less than 130 acres of tributary area), and are not considered a part of the major drainageway basin system.

11.2 Analysis When System is Part of a Major Drainageway Basin

When determining minor storm event peak flow rates from off-site areas, no benefit shall be recognized for detention in the off-site areas.

For determination of peak flow rates from the major storm event and other less frequent events, no benefit shall be recognized for on-site detention in the off-site areas. While the smaller on-site detention ponds provide some benefit immediately downstream, it has been shown that the benefit diminishes as the number of relatively small ponds increases with the accumulation of more tributary area. It has been suggested that there may be very little benefit along the major drainageway when numerous on-site detention ponds are provided in the upstream watershed (Urbonas and Glidden 1983).

For determination of peak flow rates from the major storm event and other less frequent events, the benefits provided by constructed, publicly operated and maintained, regional detention facilities in the off-site areas may be recognized, if approved by the City. On-site and regional detention facilities are discussed in more detail in the Storage Chapter.

11.3 Analysis When System is Not Part of a Major Drainageway Basin

When determining minor storm event peak flow rates from off-site areas, no benefit shall be recognized for detention in the off-site areas.

For determination of peak flow rates from the major storm event and other less frequent events, runoff may be calculated assuming historic runoff rates if the off-site area is undeveloped. Benefits of constructed and City-accepted on-site detention facilities in the off-site area can be recognized if the off-site area is partially or fully developed.

12.0 Additional Considerations Regarding Conveyance of Runoff from Major Drainageway Basins

Although the benefits provided by constructed, publicly operated and maintained regional detention facilities may be recognized if approved by Colorado Springs Engineering, a fully developed “emergency conditions” scenario must be analyzed that does not consider the benefits of upstream regional detention facilities. Conveyance facilities and channel improvements may be designed considering the benefits of

upstream regional detention when approved by Colorado Springs Engineering. In addition, it must be shown that the "emergency conditions" runoff can be safely conveyed, using additional capacity provided by freeboard or buffer areas, without impacting proposed structures or homes. Consideration of this additional scenario is warranted because of the potential threat to public health, safety, and welfare associated with flooding along major drainageways.

13.0 References

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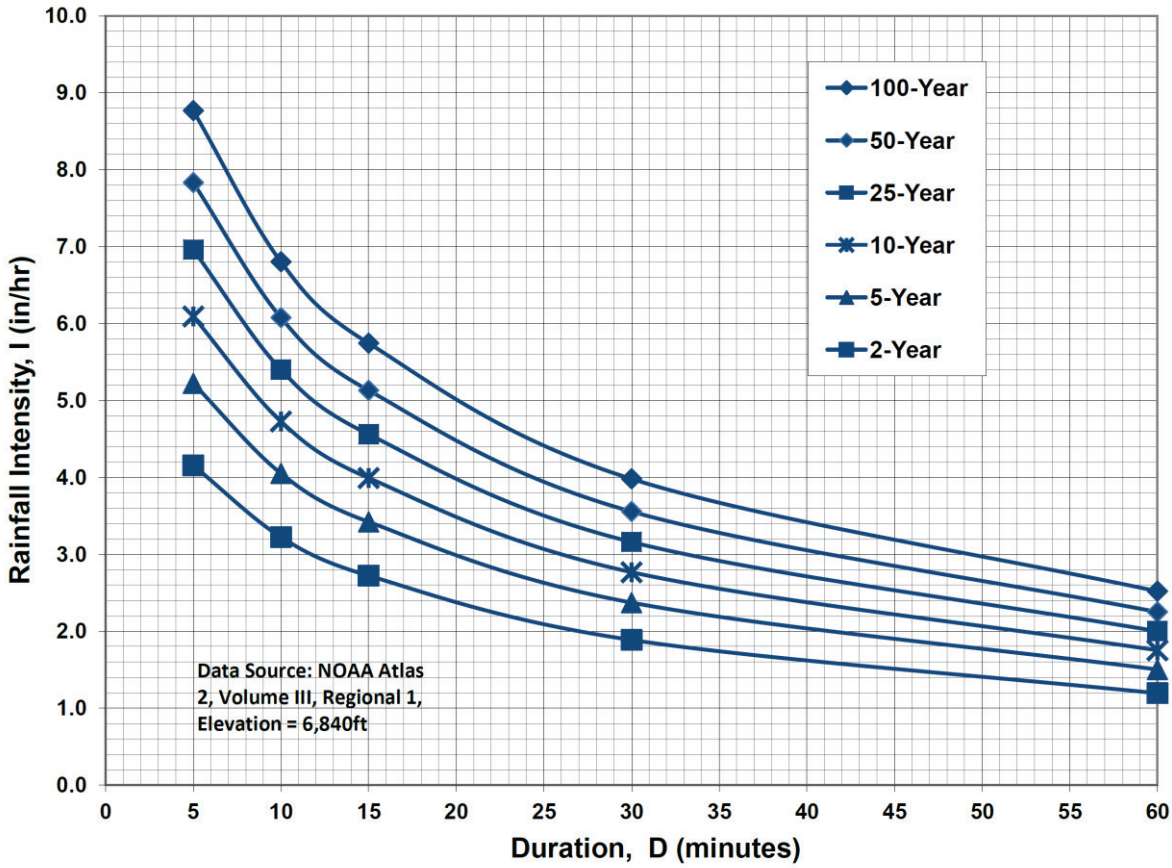
Figure 6-5. Colorado Springs Rainfall Intensity Duration Frequency



IDF Equations

$I_{100} = -2.52 \ln(D) + 12.735$

$I_{50} = -2.25 \ln(D) + 11.375$

$I_{25} = -2.00 \ln(D) + 10.111$

$I_{10} = -1.75 \ln(D) + 8.847$

$I_{5} = -1.50 \ln(D) + 7.583$

$I_{2} = -1.19 \ln(D) + 6.035$

Note: Values calculated by equations may not precisely duplicate values read from figure.

Figure 6-6. 2-Year, 6-Hour Precipitation
Tenths of an Inch (NOAA Atlas 2)

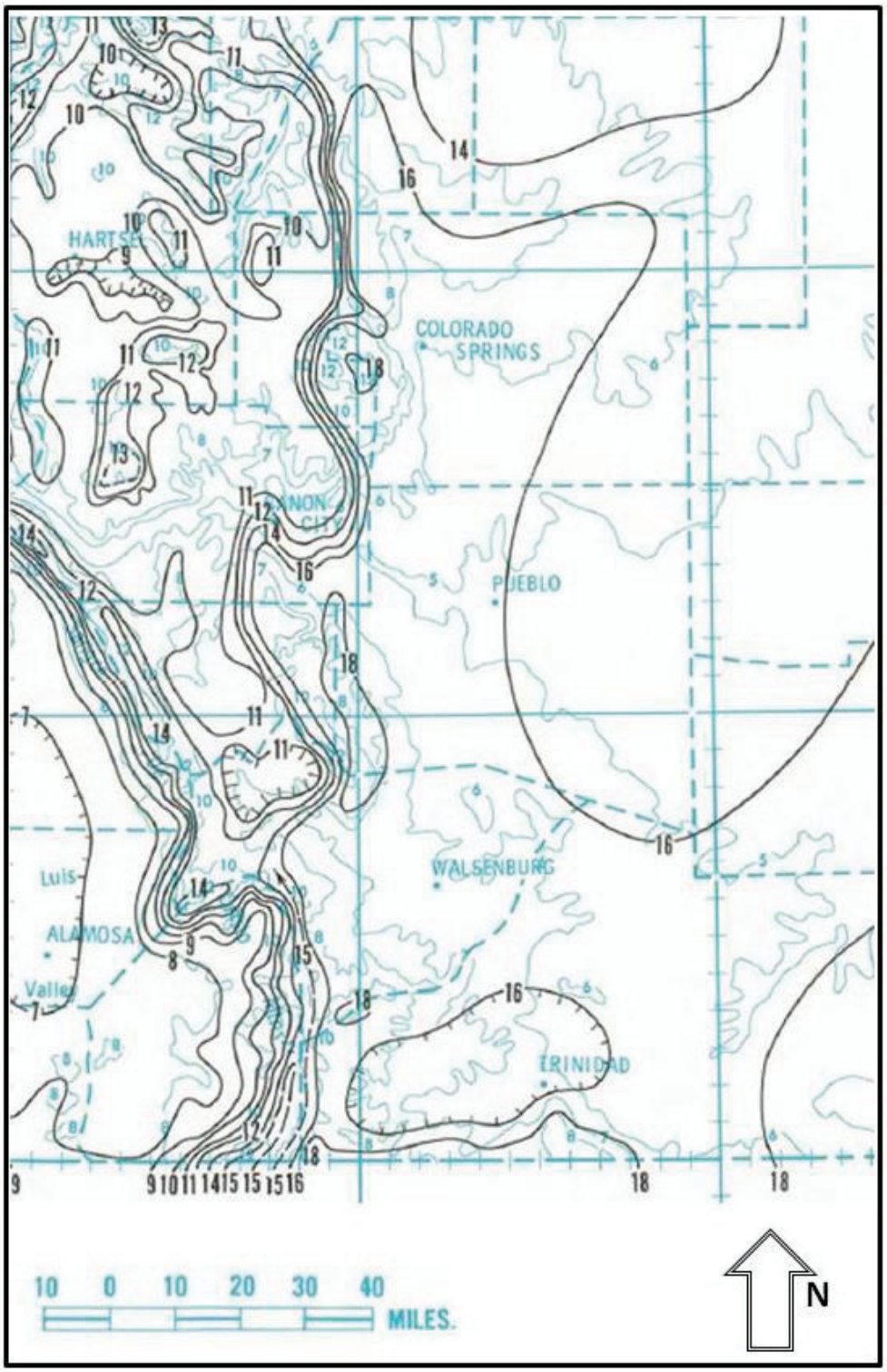


Figure 6-7. 5-Year, 6-Hour Precipitation
Tenths of an Inch (NOAA Atlas 2)

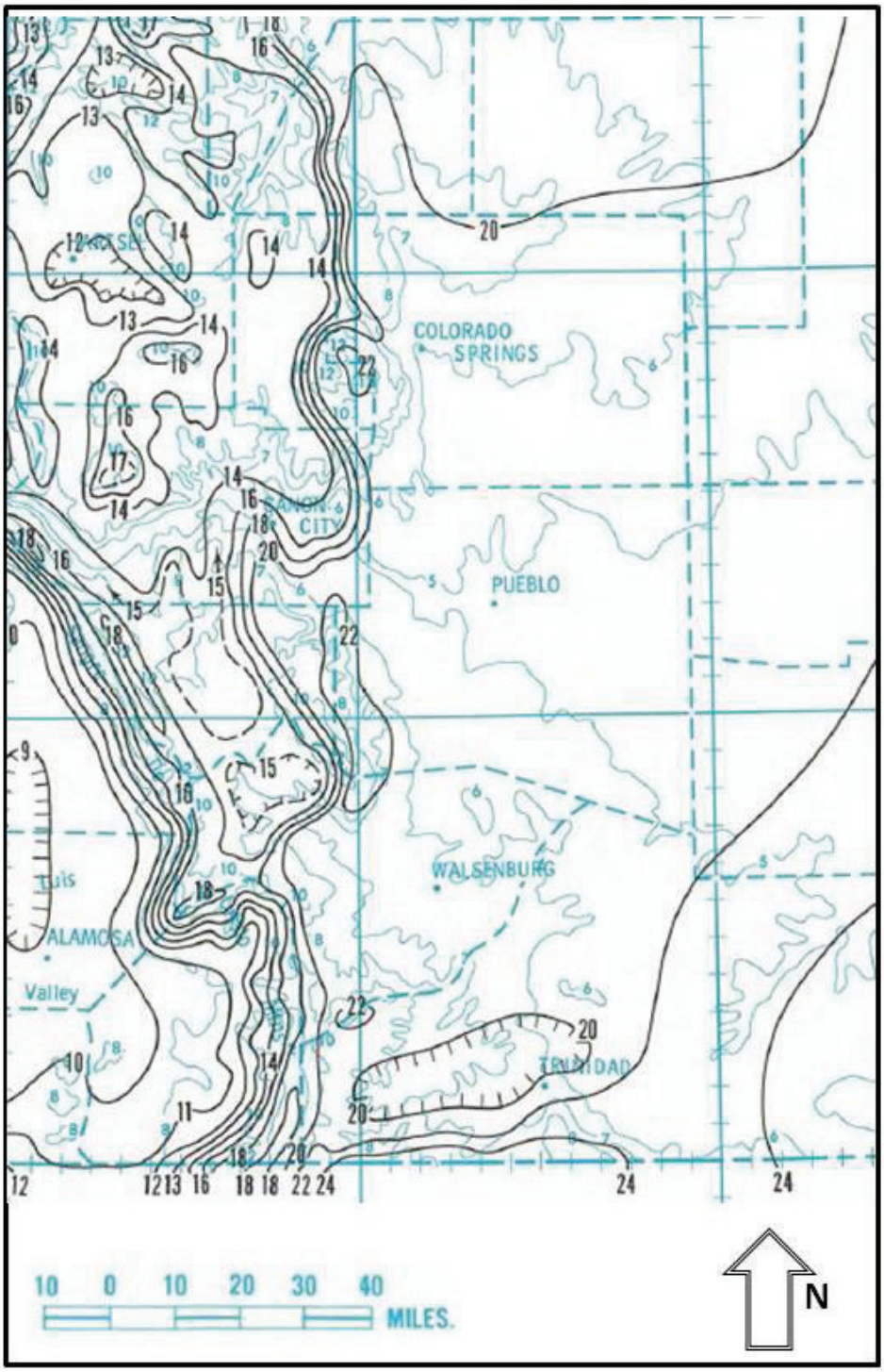


Figure 6-8. 10-Year, 6-Hour Precipitation
Tenths of an Inch (NOAA Atlas 2)

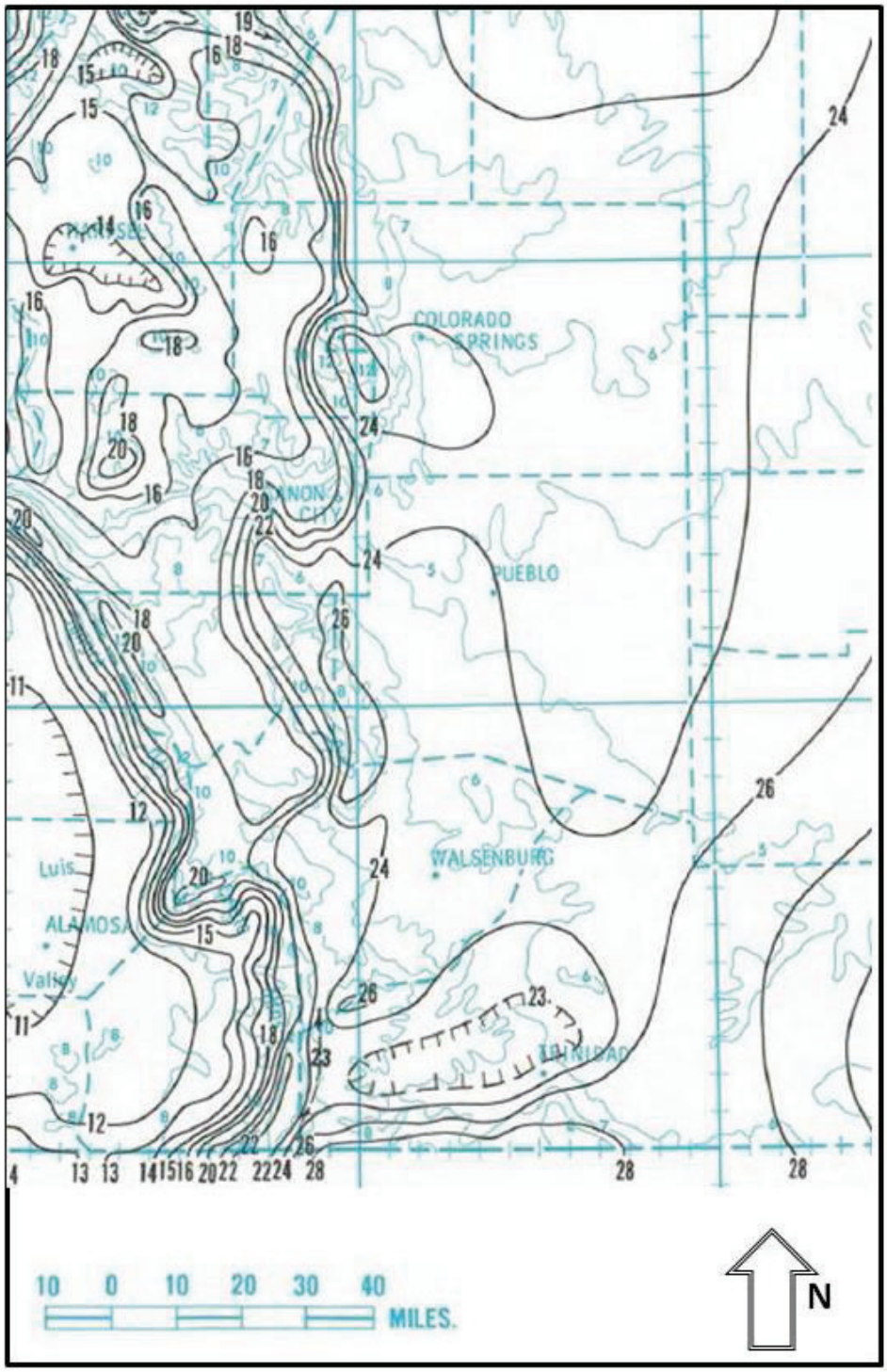


Figure 6-9. 25-Year, 6-Hour Precipitation
Tenths of an Inch (NOAA Atlas 2)

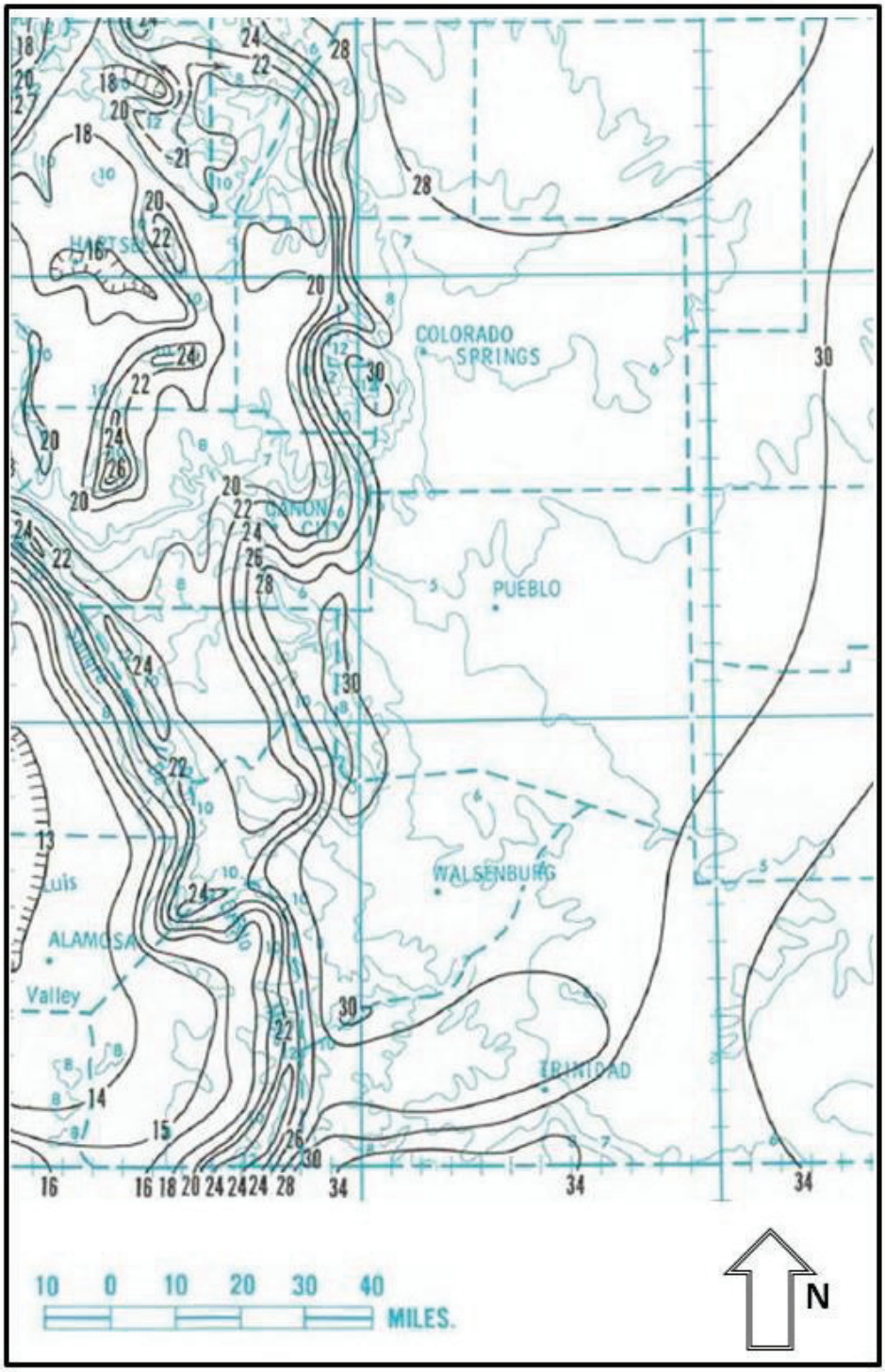


Figure 6-10. 50-Year, 6-Hour Precipitation
Tenths of an Inch (NOAA Atlas 2)

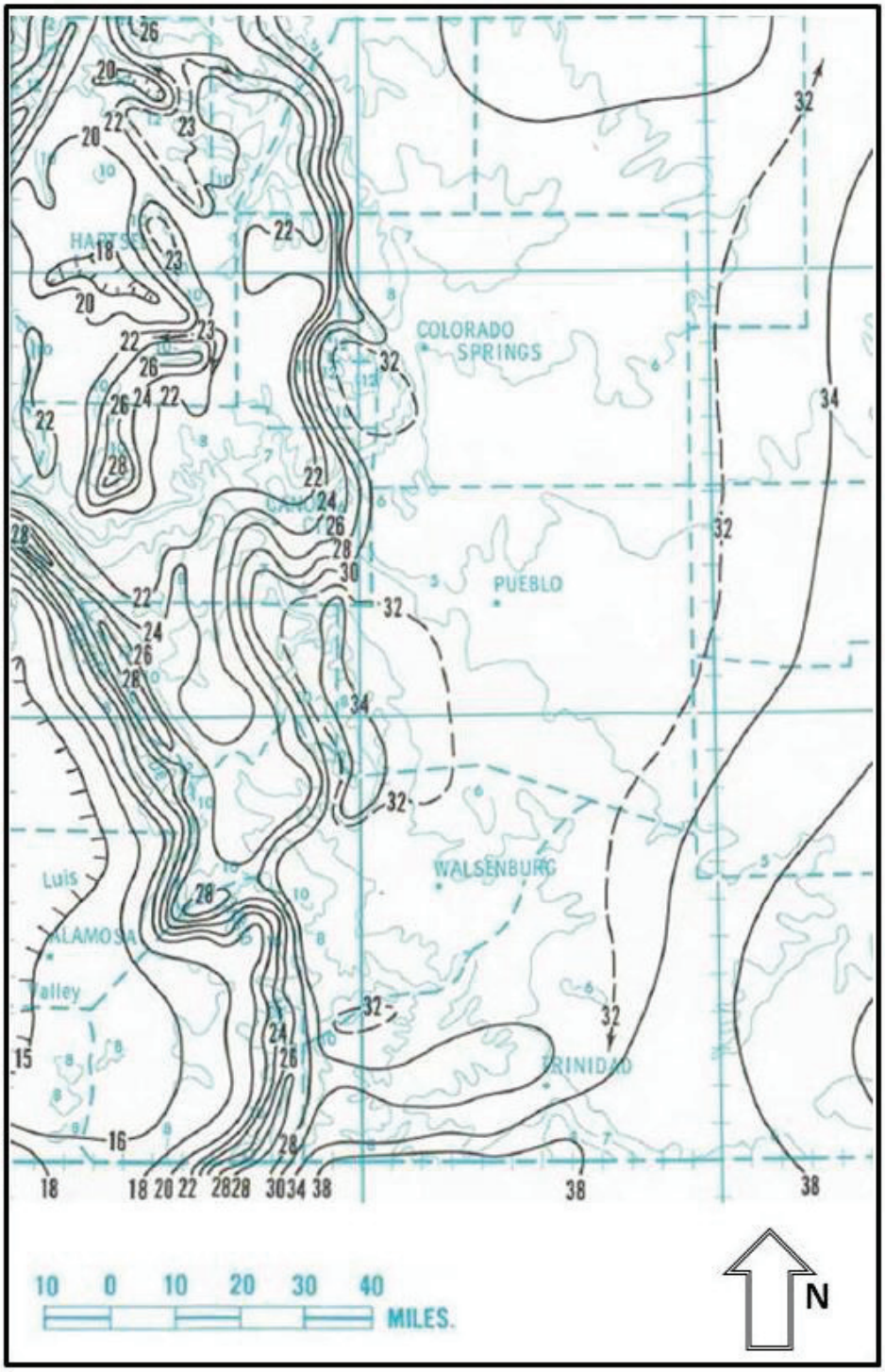


Figure 6-11. 100-Year, 6-Hour Precipitation
Tenths of an Inch (NOAA Atlas 2)

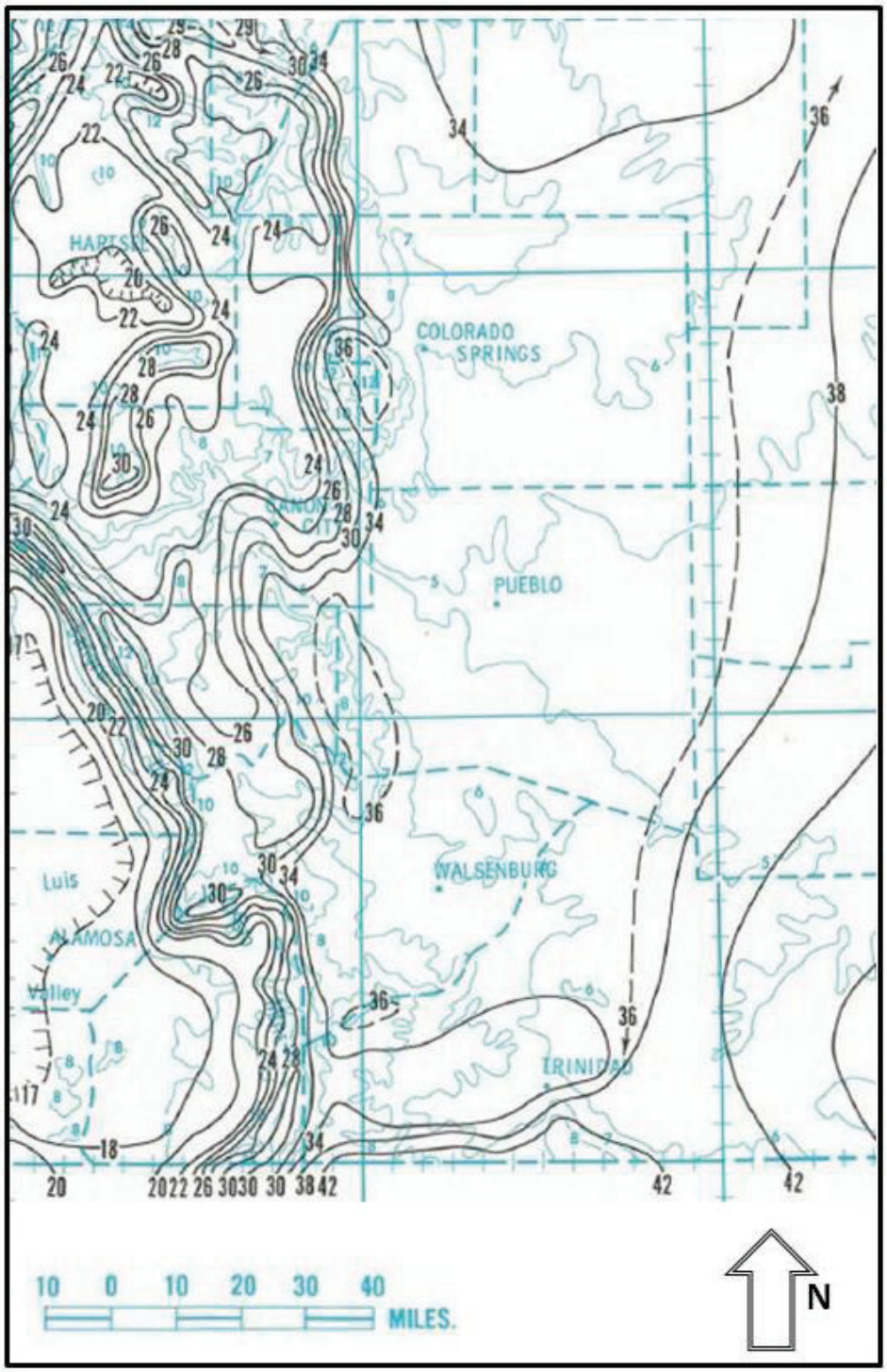


Figure 6-12. 2-Year, 24-Hour Precipitation
Tenths of an Inch (NOAA Atlas 2)

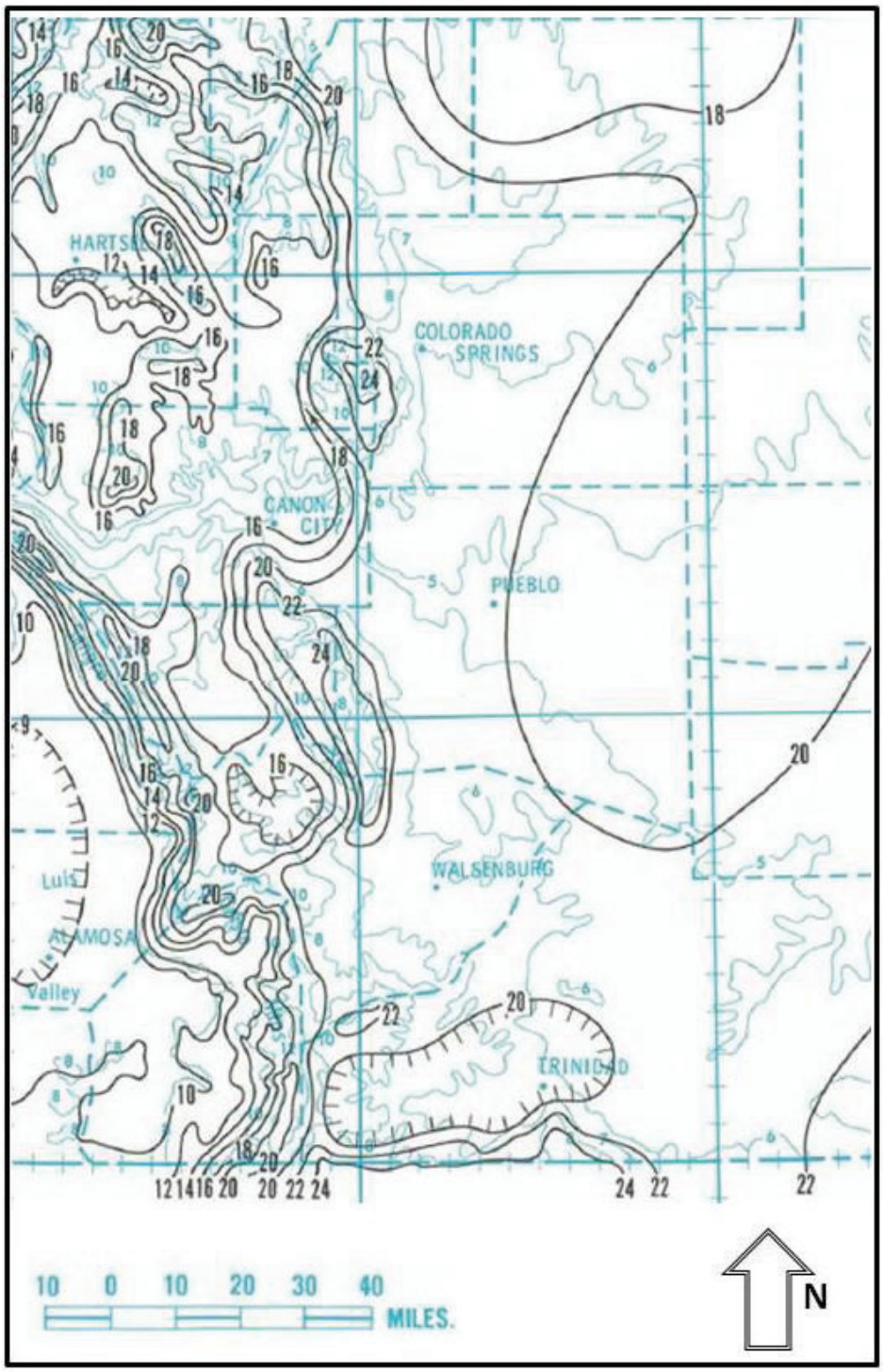


Figure 6-13. 5-Year, 24-Hour Precipitation
Tenths of an Inch (NOAA Atlas 2)

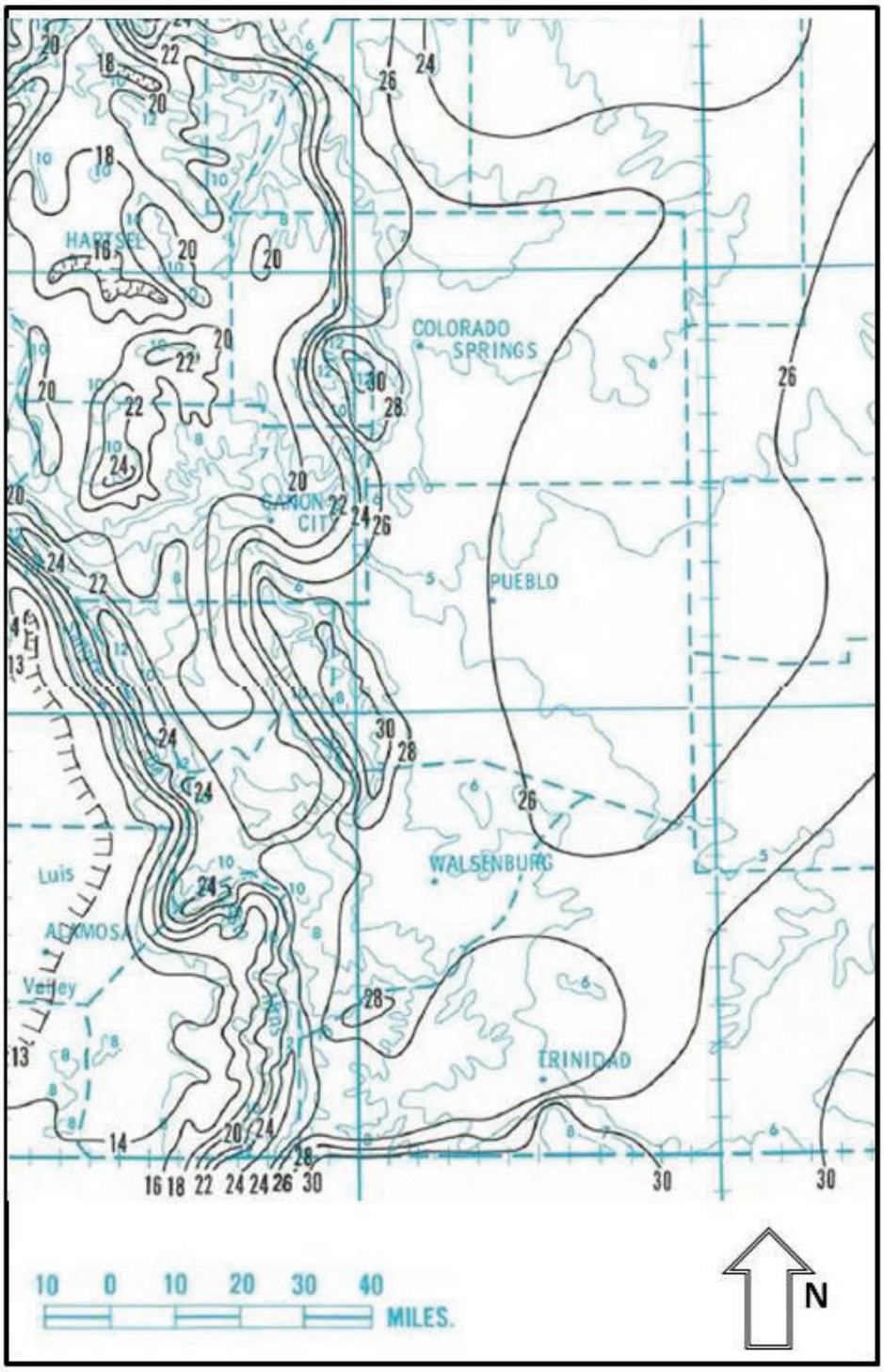


Figure 6-14. 10-Year, 24-Hour Precipitation
Tenths of an Inch (NOAA Atlas 2)

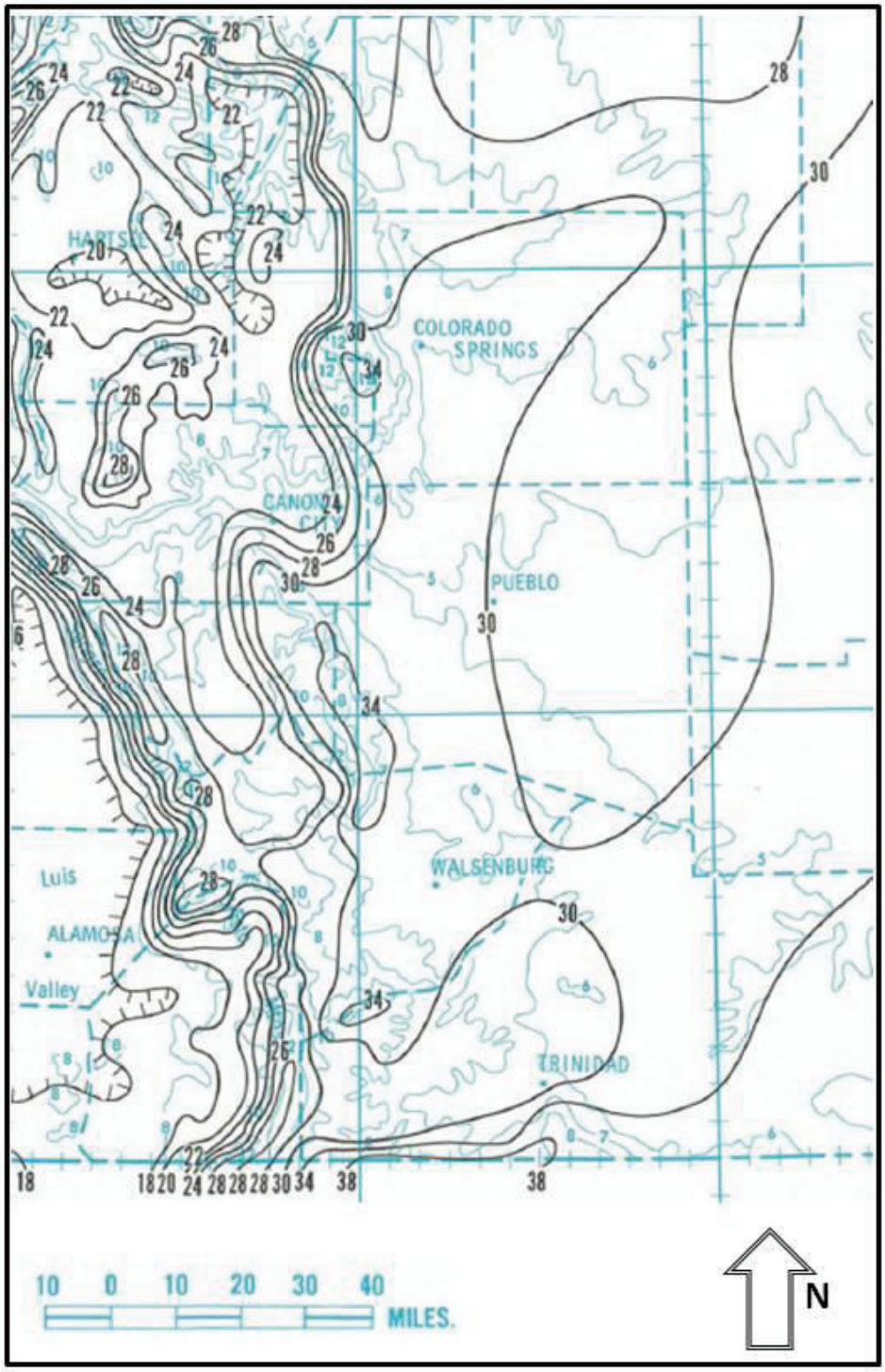


Figure 6-15. 25-Year, 24-Hour Precipitation
Tenths of an Inch (NOAA Atlas 2)

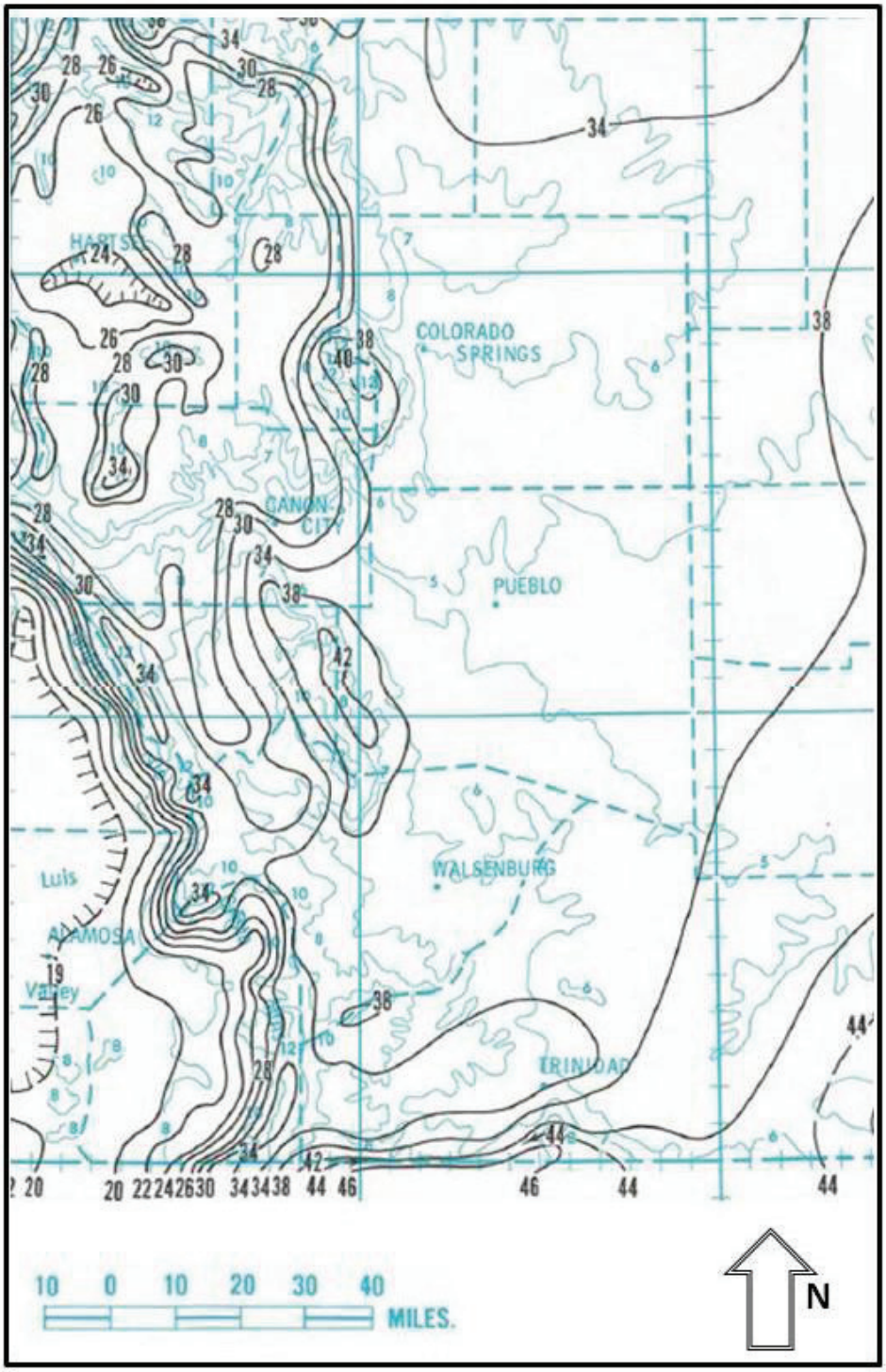


Figure 6-16. 50-Year, 24-Hour Precipitation
Tenths of an Inch (NOAA Atlas 2)

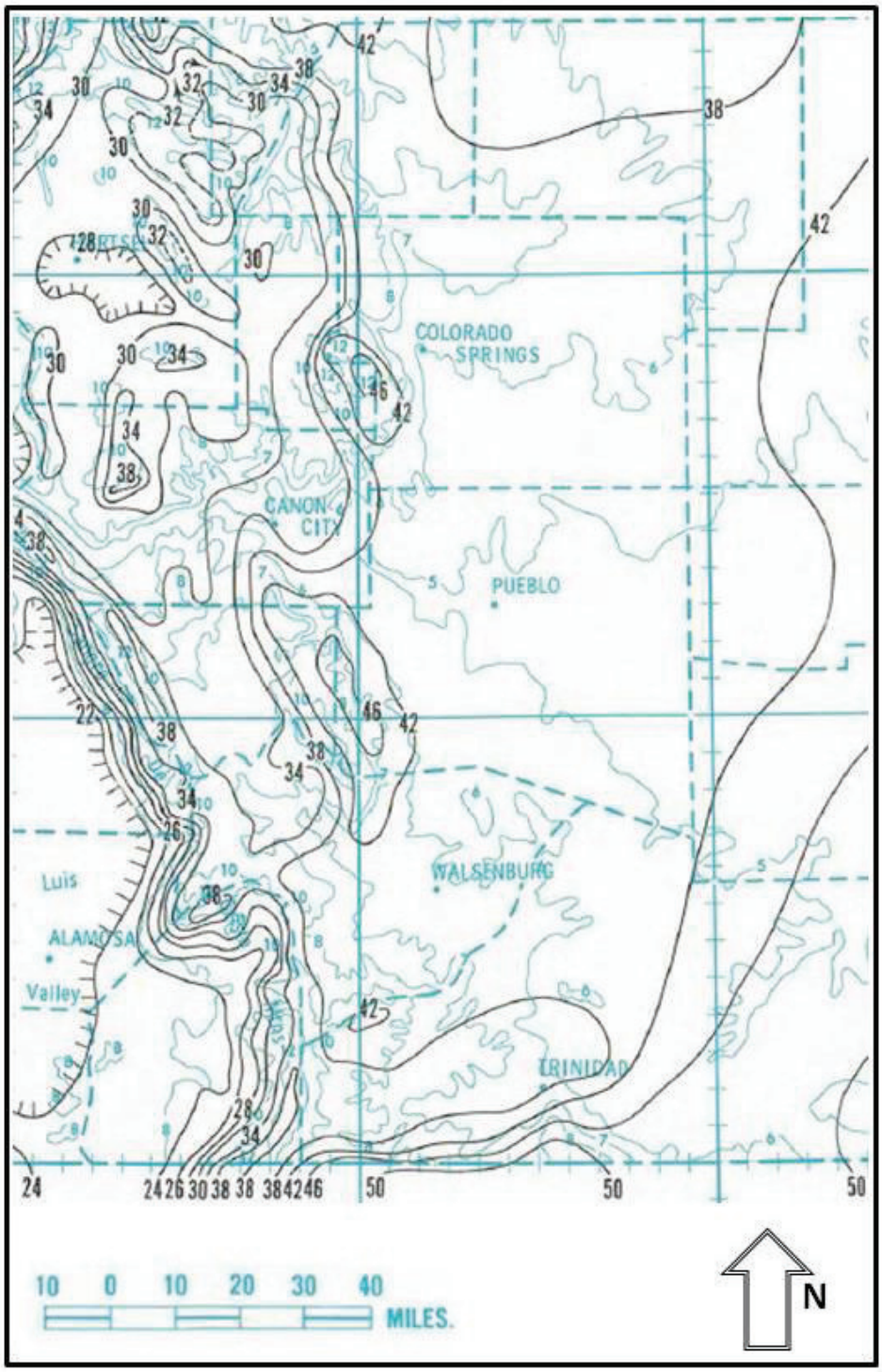


Figure 6-17. 100-Year, 24-Hour Precipitation
Tenths of an Inch (NOAA Atlas 2)

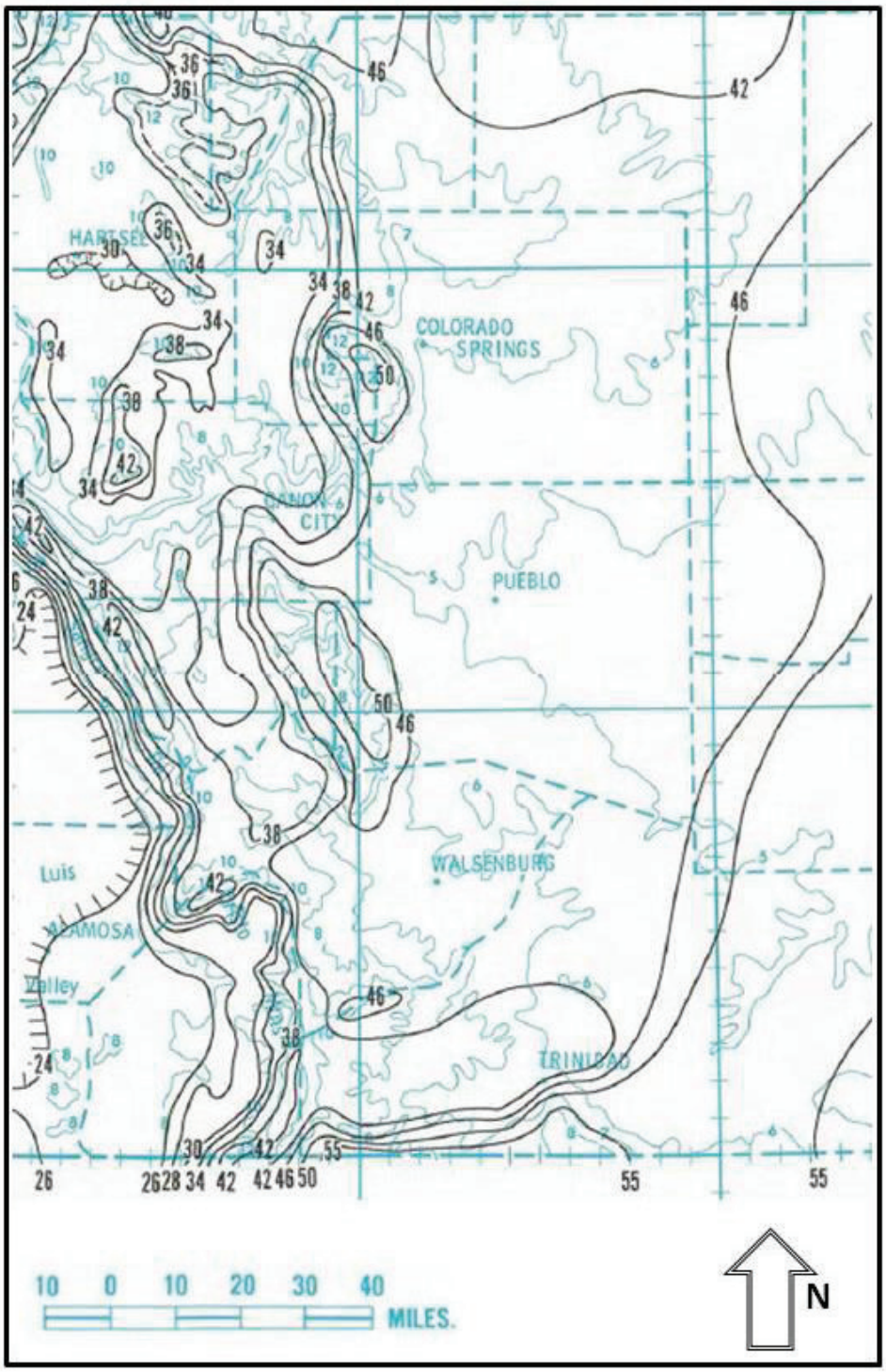


Figure 6-18a. Example Nomograph for Determination of 1-Hour Rainfall Depth for Range of Recurrence Intervals
based on 2- and 100-year 1-Hour Values (NOAA Atlas 2)

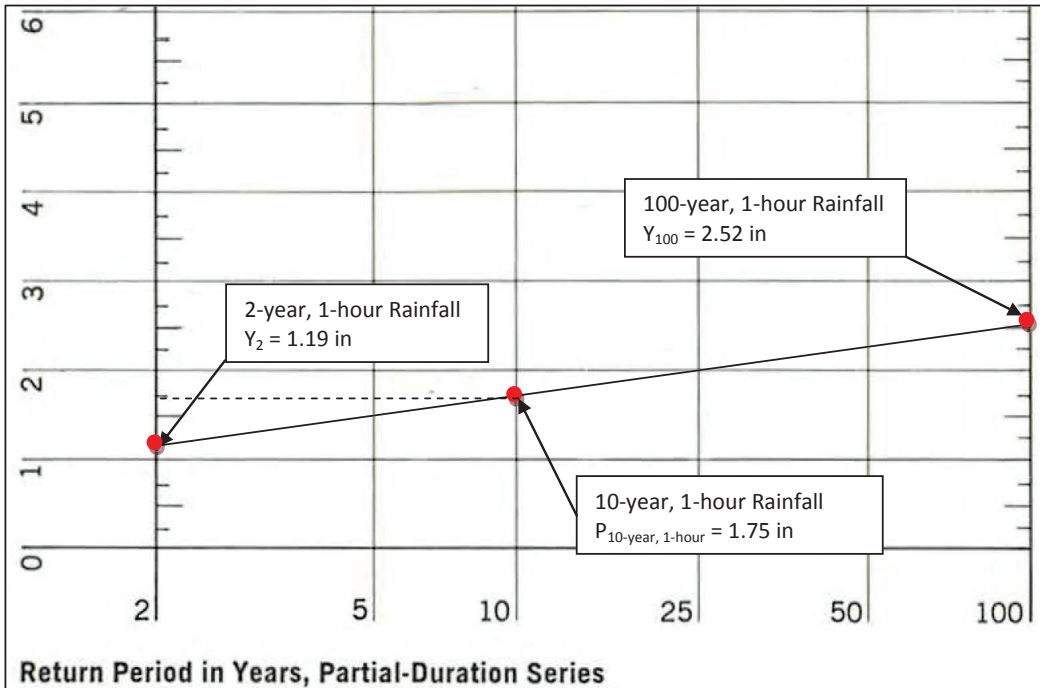


Figure 6-18b. Blank Nomograph for Determination of 1-Hour Rainfall Depth for Range of Recurrance Intervals
based on 2- and 100-year 1-Hour Values (NOAA Atlas 2)

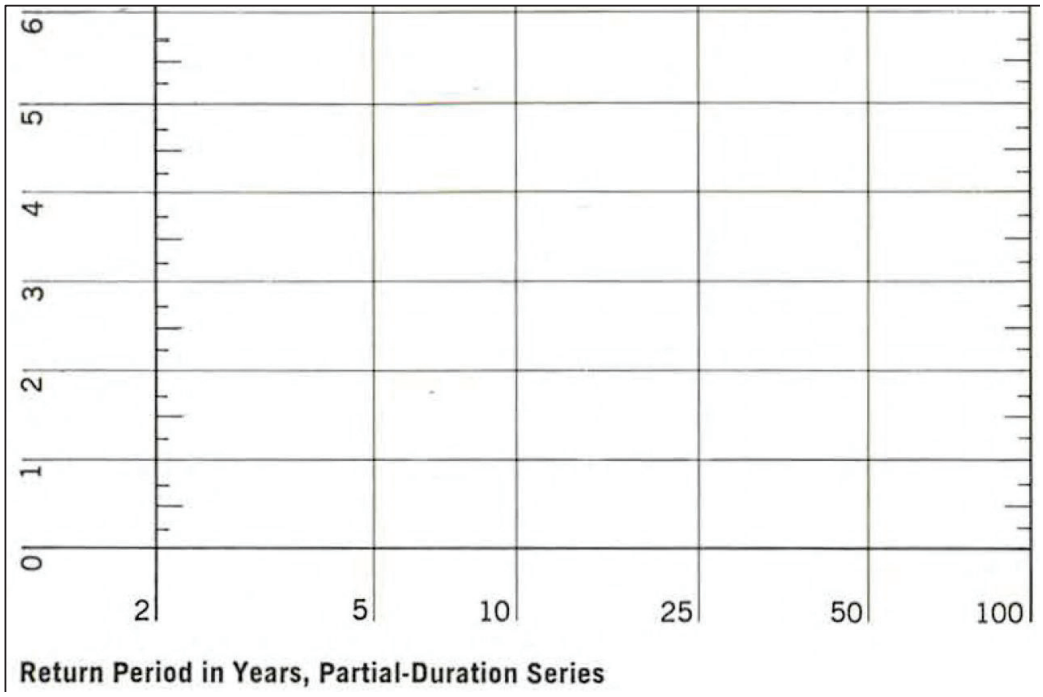


Figure 6-19. 2-Hour Design Storm Distributions By Drainage Basin Area

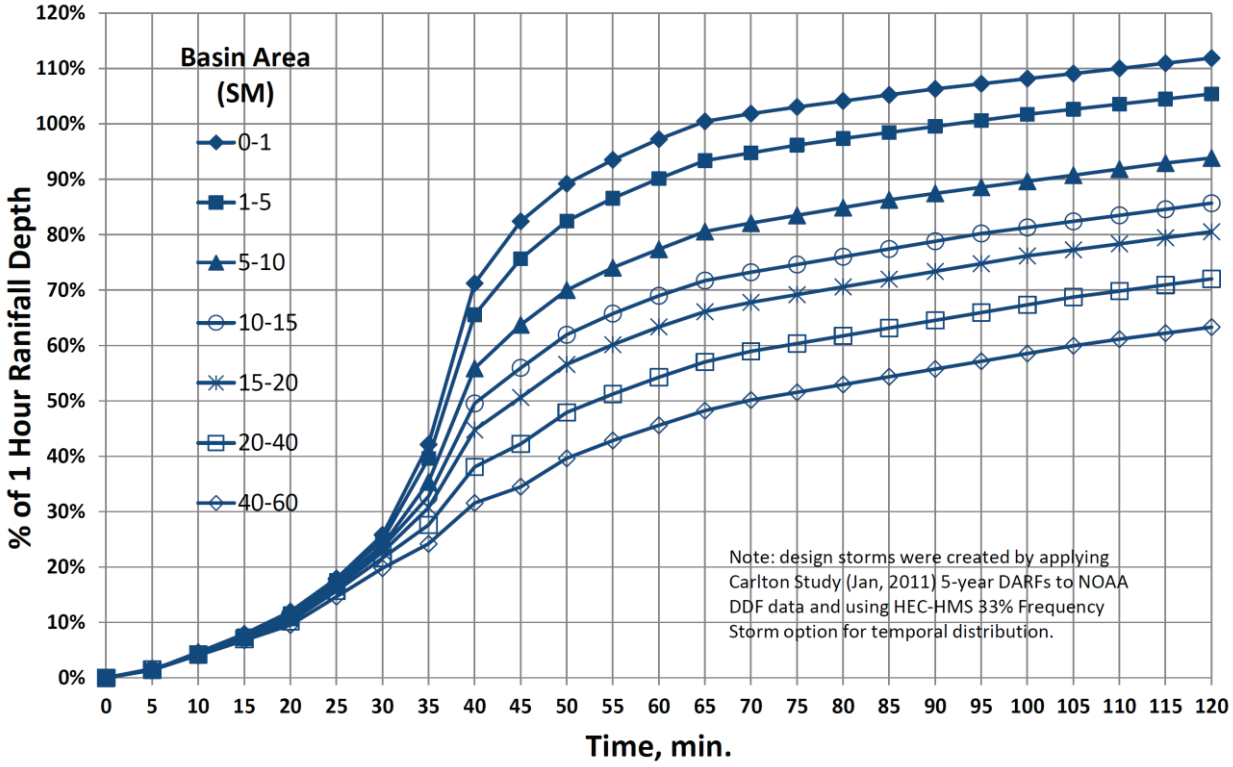


Figure 6-20. NRCS Type II 24-Hour Storm Distribution (≤ 10 mi^2)

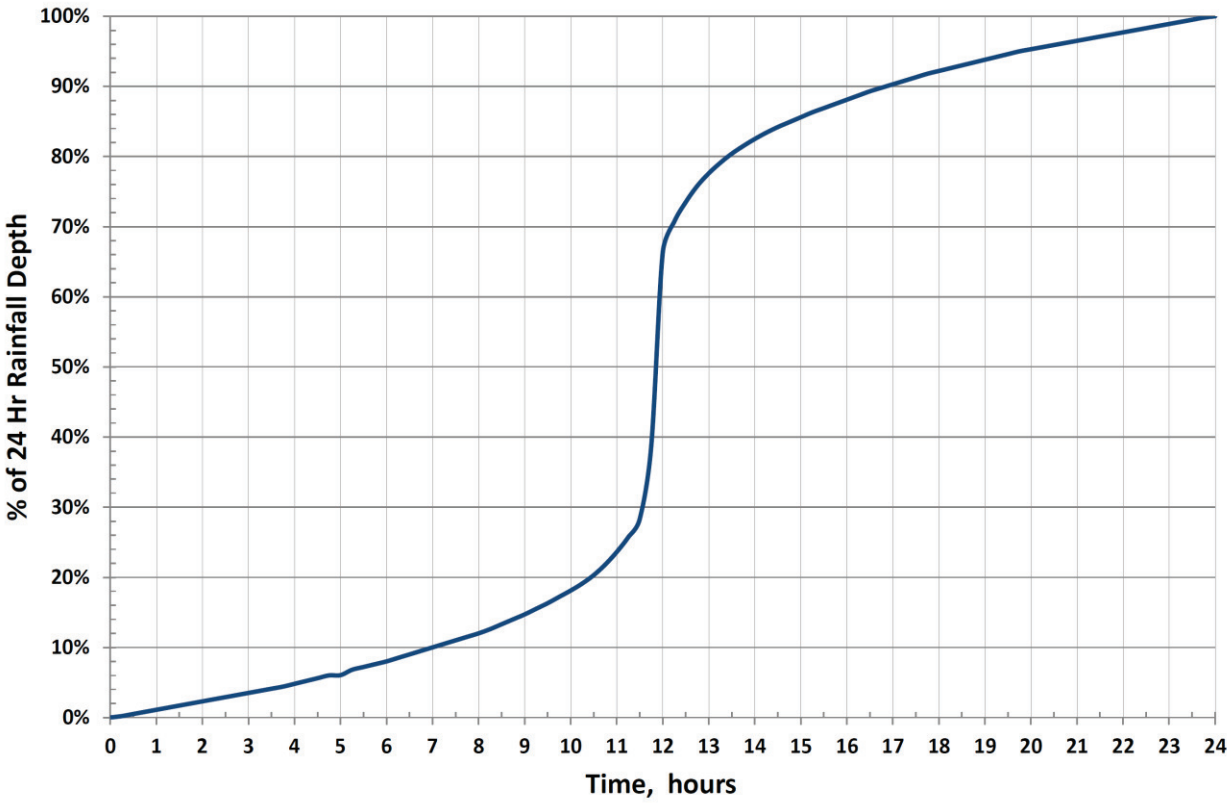


Figure 6-21. Depth-Area-Duration Adjustment Factors for 2-Hour Thunderstorms
(Carlton 2011)

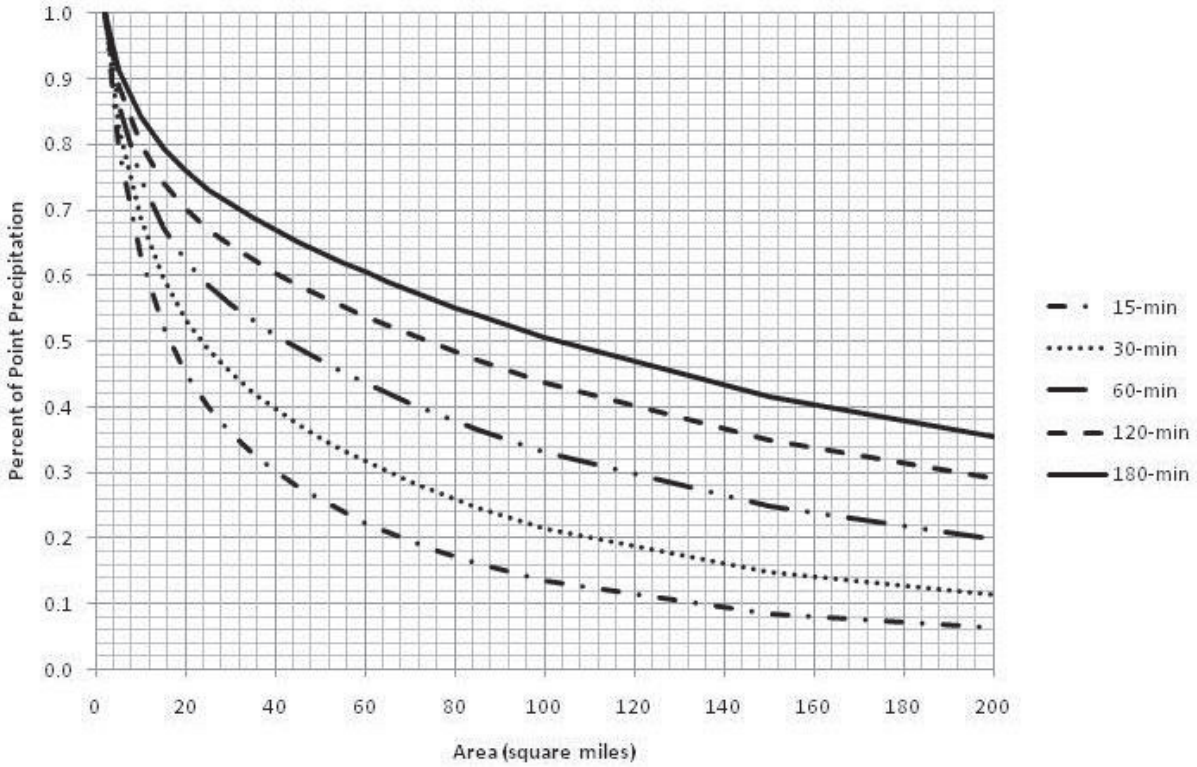


Figure 6-22. Depth-Area-Duration Adjustment Factors for 24-Hour Frontal Storms

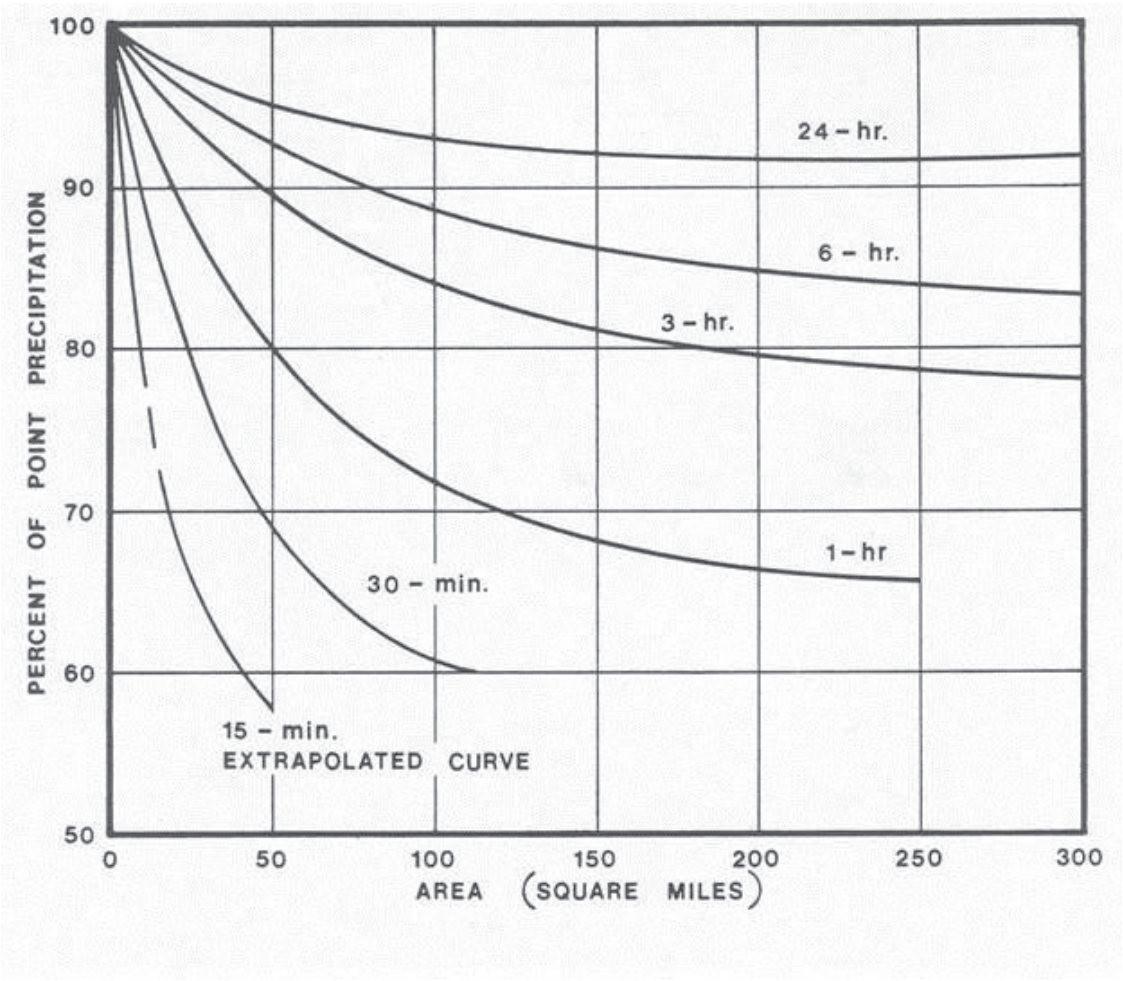


Figure 6-23. Standard Form SF-1, Time of Concentration

Standard Form SF-1. Time of Concentration

PROJECT _____
PREPARED BY _____ DATE _____

Sub-Basin Data			Initial/Overland Time (t_i)			Travel Time (t_t)				t_c Check (Urbanized Basins)		Final t_c	Remarks
DESIGN	C_5	AREA (A) acres	LENGTH (L) feet	SLOPE (S) %	t_i minutes	LENGTH (L) feet	SLOPE (S) %	Velocity (V) ft/sec	t_t minutes	TOTAL LENGTH (L) feet	t_c=(L/180)+10 minutes	$t_c = t_i + t_t$ minutes	

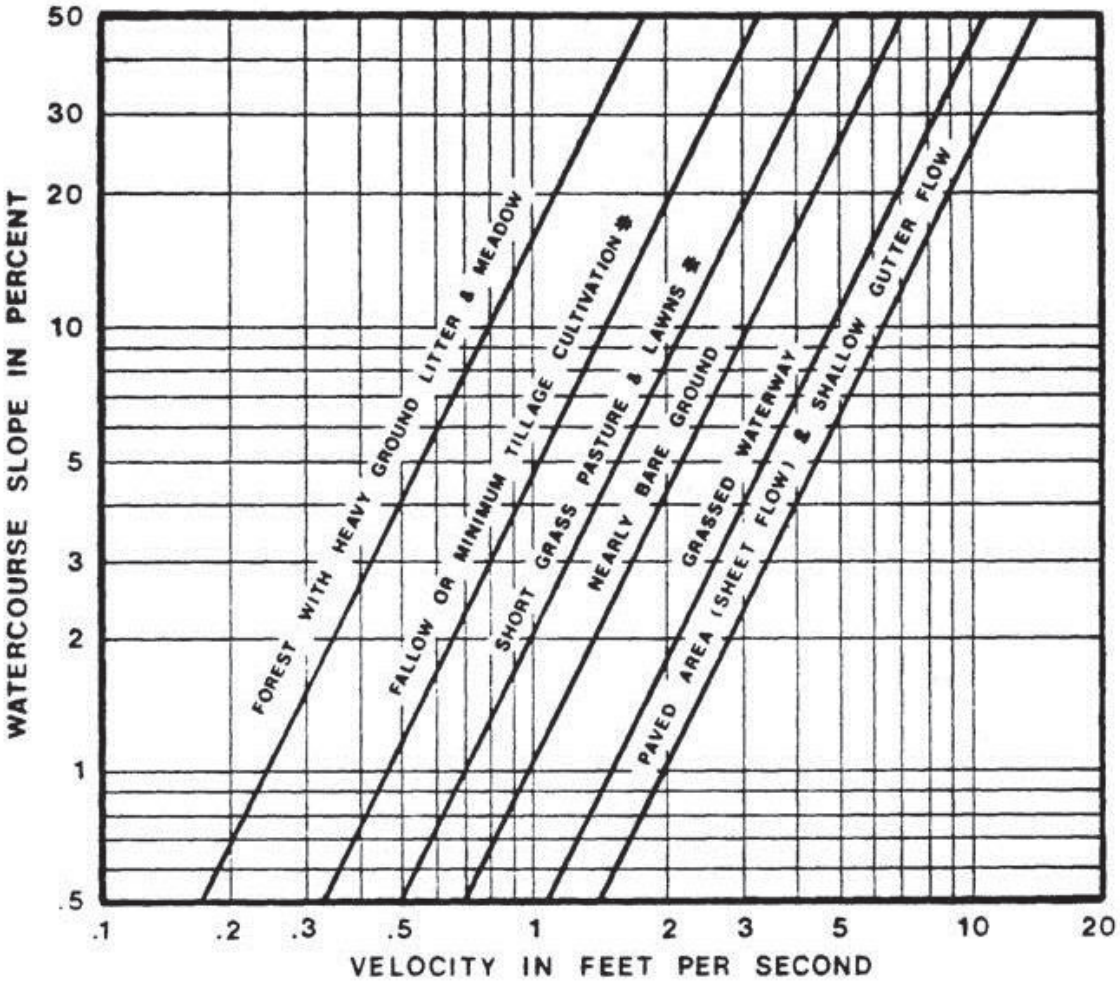
Figure 6-24. Standard Form SF-2, Rational Method Calculations

Standard Form SF-2. Storm Drainage System Design (Rational Method Procedure)

CALCULATED BY:	JOB NO.:
DATE:	PROJECT:
CHECKED BY:	DESIGN STORM:

STREET		DIRECT RUNOFF							TOTAL RUNOFF				STREET		PIPE			TRAVEL TIME			REMARKS
	DESIGN POINT	AREA DESIGN	AREA (A)	RUNOFF COEFF. (C)	t_c	CA	I	Q	t_c	Σ(CA)	I	Q	SLOPE	STREET FLOW	DESIGN FLOW	SLOPE	PIPE SIZE	LENGTH	VELOCITY	t_t	
			ac		min		in/hr	cfs	min		in/hr	cfs	%	cfs	cfs	%	in	ft	ft/sec	min	

Figure 6-25. Estimate of Average Concentrated Shallow Flow



3.2.1 Full Spectrum Detention

Full Spectrum Detention (FSD) is a design concept introduced by UDFCD (Urbonas and Wulliman 2005) that provides better control of the full range of runoff rates that pass through detention facilities than the conventional multi-stage concept. This concept also provides some mitigation of increased runoff volumes by releasing a portion of the increased runoff volume at a low rate over an extended period of time (up to 72 hours). This concept can be applied for any size drainage basin up to 640 acres and can be integrated into on-site, sub-regional or regional detention designs.

By providing an Excess Urban Runoff Volume (EURV) in the lower portion of the facility storage volume with an outlet control device similar to a Water Quality Capture Volume (WQCV), frequent and infrequent inflows are released at rates approximating undeveloped conditions. The EURV is based on the incremental difference between the developed and undeveloped runoff volume for the range of storms that produce runoff from impervious land surfaces. It was determined that the incremental increase in runoff volume from basins was relatively constant per acre of additional impervious area. The runoff relationships used to develop the EURV approach are illustrated in Figures 13-4a and 13-4b. Figure 13-4b shows that the increased volume of runoff per acre of impervious area remains relatively constant over a range of storm events.

Designing a detention basin to capture the EURV and release it slowly (at a rate similar to WQCV release rates) means that the frequent storms, smaller than approximately the 2-year event, will be reduced to very low flows near or below the sediment carrying threshold value for downstream drainageways. Additionally, by incorporating an outlet structure that limits 100-year runoff to the allowable release rate or to the undeveloped condition rate, the discharge hydrograph for storms between the 2-year and 100-year storm event will approximate the hydrograph for undeveloped conditions. This reduces the likelihood that runoff hydrographs from multiple detention facilities will combine to increase downstream discharges above undeveloped conditions and helps to more effectively mitigate the effects of urbanization.

4.1.2 EURV

UDFCD has developed empirical equations for estimating the EURV portion of the storage volume that can be applied to on-site, sub-regional or regional FSD ponds.

The empirical equations are as follows:

For NRCS Soil Group A:

$$EURV_A = 1.1\ (2.0491(I/100) - 0.1113) \qquad \text{Equation 13-5}$$

For NRCS Soil Group B:

$$EURV_B = 1.1\ (1.2846(I/100) - 0.0461) \qquad \text{Equation 13-6}$$

For NRCS Soil Group C/D:

$$EURV_{CD} = 1.1\ (1.1381(I/100) - 0.0339) \qquad \text{Equation 13-7}$$

Where:

$EURV_K$ = Excess Urban Runoff Volume in watershed inches, K=A, B or C/D soil group

I = drainage basin imperviousness, %

These equations apply to all FSDs and the EURV need not be added to the flood control volume or to the WQCV. When more than one soil type or land use is present in the drainage basin, the EURV must be weighted by the proportionate areas of each soil type and/or land use. If hydrologic routing is used to size the flood control volume, the EURV remains the same as calculated by these equations and is included in the pond's stage/storage configuration for modeling.

4.1.3 Initial Surcharge Volume

The initial surcharge volume is at least 0.3 percent of the WQCV and should be 4- to 12-inches deep. The initial surcharge volume is included in the WQCV and does not increase the required total storage volume.

4.1.4 Design Worksheets

The Full Spectrum Worksheet in the UD-Detention Spreadsheet performs all of these calculations for the standard designs. For multi-level ponds, the flood control volumes are calculated for the two design storm frequencies: the major storm and the minor storm.

4.2 Allowable Release Rates

Allowable release rates from detention facilities vary with the type of facility and with the storage volume type, as follows:

- **Flood Storage Volume**: The flood storage release rates are determined by the allowable release rates that are intended to approximate storm event runoff rates from the undeveloped upstream drainage basin.
- **EURV**: The EURV release rate is determined based on a72-hour drain time. The purpose of this slow release rate is to mitigate the impacts of increased runoff volumes due to development by reducing the potential for downstream erosion.
- **WQCV**: The WQCV release rate is determined based on a 40-hour drain time for extended detention basins. The purpose of this slow release rate is to provide time for pollutants to settle, The WQCV is incorporated into the EURV and works with it to release less erosive flows. The method for determining this design rate is described in Chapter 3 of Volume 2 of this Manual.

4.2.1 Flood Storage Release Rates

Allowable releases rates from the flood storage element of detention may be based on generalized average unit runoff rates or estimates of pre-development runoff rates. Allowable unit release rates (cfs/ac) may be used for any type of detention, however, when a hydrograph routing method is applied (for regional or

sub-regional ponds), estimated undeveloped condition release rates may be used instead.

Allowable release rates depend on pre-development basin conditions, such as soil type and land cover and the design storm. NRCS Curve Numbers (CN) represent soil and land cover conditions and the antecedent runoff condition (ARC). As described in Chapter 6, Hydrology, watershed conditions prior to short duration, 2-hour storms normally have a low runoff potential and should be represented by ARC I CNs.

Allowable unit release rates for the 2-hour design storm with ARC I CNs are provided in Table 13-2. These values represent average runoff rates from typical undeveloped basins assuming that the entire basin is covered with a single NRCS hydrologic soil group (HSG). When more than one HSG is present in the drainage basin, the allowable unit release rates must be weighted by the proportionate areas of each soil type to determine a composite allowable unit release rate.

Table 13-2. Allowable Unit Release Rates (cfs/ac)
(For 2-hour Design Storm w/ARC I CNs)

Design Return Period (years)	NRCS Hydrologic Soil Group		
	A	B	C&D
2	0.00	0.01	0.04
5	0.00	0.04	0.30
100	0.10	0.30	0.50

When pre-development runoff rates are estimated instead of using the allowable unit release rates, an undeveloped runoff rate shall be calculated for each of the design return periods shown in Table13-2 and compared to the calculated corresponding release rates from the proposed pond. The release rates from the proposed pond must be equal to or less than the estimated pre-development runoff rates. Pre-development runoff estimates must be based on the appropriate basin parameters, methods and storm characteristics as described in Chapter 6, Hydrology.

4.2.2 EURV Release Rate

The EURV is intended to fully drain within a 72-hour period after the end of the storm. This is accomplished by a control plate placed in the outlet structure with the appropriate orifice (hole) sizes and spacing similar to those used for the release of the WQCV, see Volume 2 of this Manual. UDFCD has estimated the area of the holes in the control plate based on Equation 13-8.

$$A_o = 88V^{(0.95/H^{0.085})}/T_D(S^{0.09})H^{(2.6\,S^{0.3})} \qquad \text{Equation 13-8}$$

Where:

A_o = area per row of orifices spaced on 4-inch centers (in^2)
V = design volume (WQCV or EURV, acre-ft)
T_D = time to drain the prescribed volume (hrs)
(i.e., 40 hours for WQCV or 72 hours for EURV)
H = depth of volume (ft)
S = slope (ft/ft)

The Full Spectrum Worksheet in the UD-Detention Spreadsheet performs these calculations for the

standard designs. However, depending on the upstream basin conditions and the pond and outlet configurations the designer may need to revise the control plate hole configuration to meet drain time criteria. To confirm that a pond design operates as intended an inflow hydrograph must be routed through a pond model.

Figure 13-4a. Excess Urban Runoff Volume (EURV) per Runoff Return Period [Type C/D Soils]
(Source: Urbonas and Wulliman 2005)

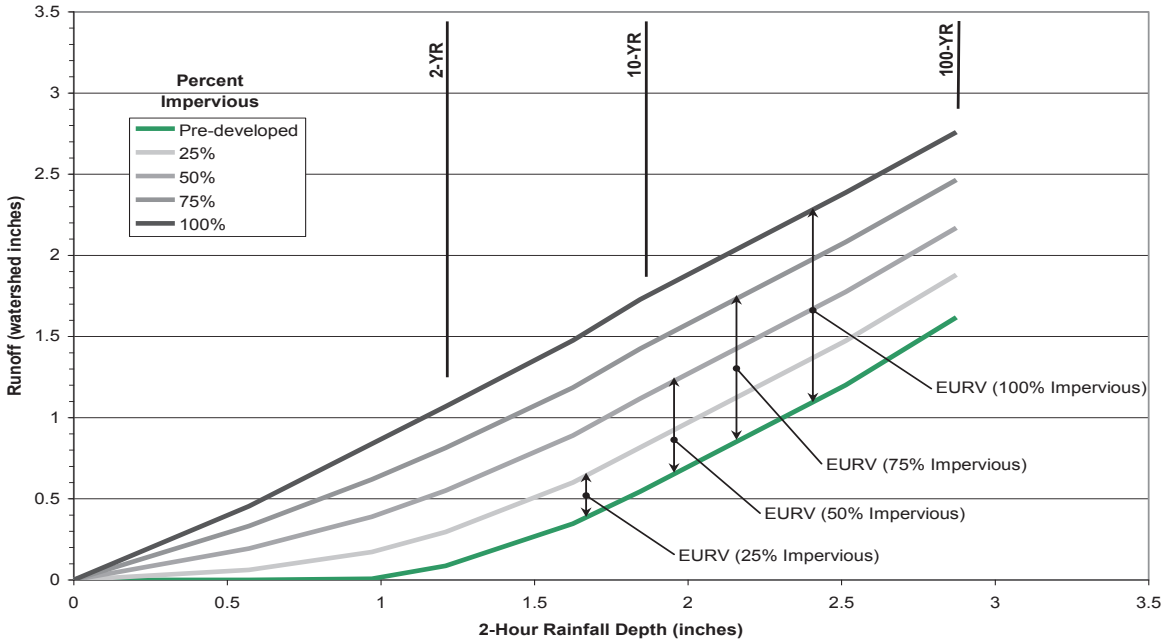


Figure 13-4b. Excess Urban Runoff Volume (EURV) per Impervious Acre [Type C/D Soils]

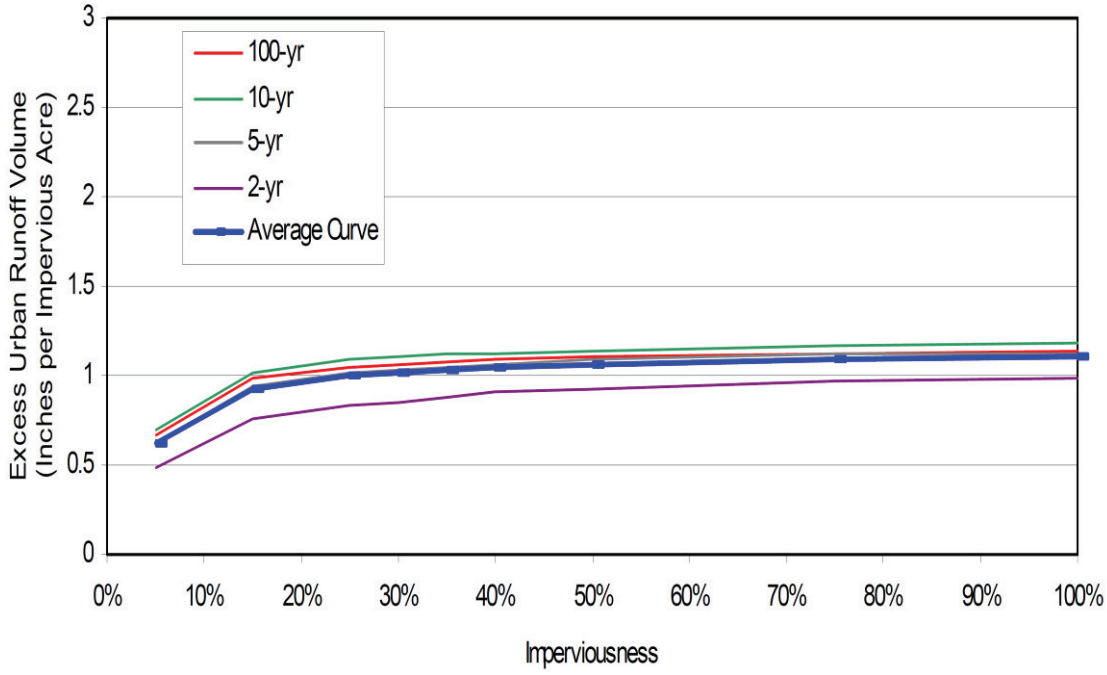


Figure 13-5. Options for Detention. WQCV and EURV Configurations