San Bernardino County Hydrology Manual

2026 Edition

SAN BERNARDINO COUNTY HYDROLOGY MANUAL

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The *San Bernardino County Hydrology Manual* (Manual) was first published in 1983 and subsequently revised in 1986 to incorporate advances in hydrologic modeling techniques and rainfall–runoff data calibration. At that time, concurrent efforts included the revision of the Orange County Hydrology Manual (1986) and the Los Angeles County Drainage Area (LACDA) study undertaken by the U.S. Army Corps of Engineers, Los Angeles District Office, for the hydrologic planning of the Santa Ana River Basin. The 1986 revision introduced improvements such as an updated Rational Method, revised unit hydrograph procedures, expanded design storm criteria, link-node modeling guidance, and desert hydrology parameters.

Since that revision, the Manual has served as the foundation for hydrology analyses, drainage studies, master plans of drainage, and development review throughout the County. Except for a 2010 addendum addressing arid region methodologies, no comprehensive update was undertaken until this present effort.

This updated edition integrates substantial advancements in data, technology, and practice, including: adoption of NOAA precipitation data countywide; modernization of soil maps and Curve Numbers; updated Antecedent Moisture Condition mapping; incorporation of public tools for Rational Method and hydrologic calculations; implementation of the U.S. Army Corps of Engineers' HEC-HMS software for hydrograph development; and new guidance on post-fire hydrology, sediment and debris hazards, and detention basin design.

This revision represents the County's continued commitment to providing a comprehensive, scientifically based, and climate-resilient hydrology manual to guide public agencies, consultants, and developers in the planning, design, and management of stormwater facilities throughout San Bernardino County.

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ACRONYMS AND ABBREVIATIONS

AMC	Antecedent moisture condition
AR	Atmospheric river
A-T	Adjustment-Transposition
BAER	Burned Area Emergency Response
BARC4	Burned-area reflectance classification
cfs	Cubic feet per second
CN	Curve Number
CONUS	Contiguous United States
DARF	Depth-Area Reduction Factor
DDF	Depth-Duration-Frequency
DEM	Digital elevation model
DSOD	Division of Safety of Dams
FF	Fire Factor
fps	Feet per second
GIS	Geographic information system
gNATSGO	Gridded National Soil Survey Geographic Database
HEC-HMS	Hydrologic Engineering Center-Hydrologic Modeling System
HSG	Hydrologic soil group
HWL	High-water level
IDC	Intensity-Duration Curve
NAM	North American Monsoon

NOAA	National Oceanic and Atmospheric Administration
NRCS	Natural Resources Conservation Service
NWS	National Weather Service
PRUN	Peak rainfall unit number
RR	Relief ratio
SBS	Soil burn severity
SCS	Soil Conservation Service
SFRR	Storm frequency relationship ratios
sq. mi.	Square mile
SWE	Snow water equivalent
UH	Unit Hydrograph
USACE	U.S. Army Corps of Engineers
USDA	U.S. Department of Agriculture
UTPO	Unit time period ordinate
WERT	Watershed Emergency Response Team
WHI	Wildfire Hydrologic Impact

GLOSSARY

Critical Duration

In a design storm event for a hydraulic structure, essentially the "time of concentration," which is the time it takes for water deposited at the most remote part of a watershed to flow to the structure, outlet, or a specific location.

Debris Basin

A basin primarily designed to capture sediment, debris, and other transported materials originating from undeveloped upstream watersheds. Its purpose is to facilitate the design of downstream storm drains, channels, or detention basins based on clear-water flow, without incorporating a bulking factor. However, the application of a bulking factor downstream may still be warranted if the debris basin is not adequately sized to retain the total estimated debris generated by a 100-year storm event from the contributing watershed.

Distribution Graph

A unit hydrograph whose ordinates are expressed in terms of percent of ultimate discharge. Generally developed as a block graph with each block representing its associated percentage of unit runoff that occurs during the specified unit time period. The unit time period used in the distribution graph is identical to the unit time period specified for the unit hydrograph.

Duration

The specified length of storm time under study. May be expressed in any time unit, such as seconds, minutes, hours, days, or seasons.

Effective Rainfall

Total rainfall less infiltration, evaporation, transpiration, interception, and storage.

Exceedance (Cumulative) Probability

The probability that a precipitation event of a specified depth and duration will be exceeded in one year.

Flow-By Detention Basin

A detention basin located adjacent to a main channel or storm drain where runoff enters the basin only when the water level in the main channel or storm drain rises above a certain elevation. Under normal flow conditions, runoff bypasses the basin, and only higher flows are diverted into the basin for detention.

Flow-Through Detention Basin — A detention basin designed so that runoff from the upstream watershed flows directly through the basin, typically passing through the basin's storage area before continuing downstream. The basin temporarily stores runoff to reduce peak flow rates, but it also allows continuous flow through it.

Frequency — How often events with the specified precipitation depth and duration occur. Expressed in terms of either the return period or exceedance probability, both of which are defined below.

Intensity-Duration — By dividing precipitation depth by duration, an average intensity for a specified duration is obtained.

Joint Use Detention — A regional or local detention basin used for a secondary purpose, such as a park, football field, parking lot, golf course, lake, etc.

Lag — The time from the beginning of a continuous series of effective unit period rainfall over a watershed and the point at which the runoff rate at the concentration point reaches 50% of its peak value.

Local Detention Basin — A basin that is owned by an individual, organization, or municipality other than the San Bernardino County Flood Control District and is not part of the District’s existing or planned drainage system. This type of basin reduces the downstream peak flow rate but will not be considered in downsizing future downstream storm drains, channels, or basins.

Precipitation Depth — The amount of precipitation occurring during a specified duration of storm time. Precipitation depth is usually expressed in inches.

Regional Detention Basin — A basin that is owned and operated by the San Bernardino County Flood Control District and can be part of its existing or proposed drainage system. The basin may be used jointly for other purposes. Its main role is to reduce the downstream peak flow rate, and the necessary downstream storm drain or channel size.

Return Period (Recurrence Interval) — The long-term average time between occurrences of an event of a given depth and duration, either equaled or exceeded.

S-Graph	A summation hydrograph developed by plotting watershed discharge (percent of ultimate discharge) as a function of time (percent of lag).
Summation Hydrograph	A curve illustrating the time distribution of runoff rates resulting from a continuous series of unit period effective rainfalls over the tributary watershed upstream of the point of concentration. The ordinates of the summation hydrograph are expressed in percent of the ultimate discharge.
Temporary Detention Basin	A local detention basin used to reduce downstream peak flow rates until ultimate storm drain facilities can be constructed as part of a phased development. The life of the basin shall generally not exceed 10 years.
Ultimate Discharge	The maximum rate of watershed runoff resulting from a specified effective rainfall intensity. In this Manual, for an effective rainfall rate of 1 inch occurring over a unit period of 1 hour, the ultimate discharge is 645 cfs for every sq. mi. of watershed area. For other unit periods, the ultimate discharge is calculated by dividing 645 by the unit period in hours and multiplying by the watershed area in square miles.
Unit Hydrograph	A time distribution of runoff flow rates resulting from 1 inch of effective rainfall occurring uniformly over the tributary watershed upstream of a point of concentration, during a specified unit time period. The unit effective rainfall is generally assumed to occur as an equivalent constant rainfall intensity during a specified unit period of time.

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CHAPTER 1 INTRODUCTION

1.1. Purpose

The *San Bernardino County Hydrology Manual* (Manual) updates the August 1986 *San Bernardino County Hydrology Manual*, the April 6, 2010, *San Bernardino County Hydrology Manual Addendum for Arid Regions*, and the *Detention Basin Design Criteria for San Bernardino County*. The Manual provides criteria, methodologies, computational procedures, and examples to aid in determining watershed peak flow rates and volumes under various conditions for hydrologic studies under the County's purview.

1.2. Hydrologic Protection Standards

The County aims to plan flood protection for habitable structures and other non-floodproofed improvements based on the 100-year return-frequency event. Consequently, all drainage plans should reflect design measures consistent with the 100-year flood protection criterion.

The County also identifies design considerations for flood events with more frequent recurrence intervals. Protection levels associated with the 10-year and 25-year events may be applied to major roadways, catch basin sump design, and similar features where appropriate.

1.3. Contents

The Manual discusses the hydrologic tenets of precipitation and watershed losses in detail, including post-fire hydrologic considerations. The Manual also includes chapters on the criteria and approaches to converting rainfall to runoff using the Rational Method and Unit Hydrograph Method. Dedicated chapters cover making post-fire hydrologic parameter adjustments using sediment bulking methodology and procedures. The Manual also includes the County detention basin criteria, which was a stand-alone document.

Users are encouraged to review the Table of Contents, List of Abbreviations, Glossary, specific chapter sections, and bibliography as needed. As an overview, the Manual chapters are organized as follows:

- **Chapter 2: Precipitation**
- **Chapter 3: Losses**
- **Chapter 4: Rational Method**
- **Chapter 5: Unit Hydrograph Method**
- **Chapter 6: Post-Fire Hydrologic Parameter Adjustments**
- **Chapter 7: Sediment Bulking**
- **Chapter 8: Detention Basin Design**

1.4. Revisions

Because of ongoing technical and administrative changes in the field of stormwater management, revisions to this manual will be required from time to time. Such revisions will take place on an ongoing, as-needed basis and will be posted on the County's Flood Control webpage. The County's Director of the Department of Public Works or Chief Flood Control Engineer is hereby authorized to approve future revisions to the Manual.

CHAPTER 2 PRECIPITATION

2.1. Classification of Precipitation

This Manual refers to several types of storm precipitation mechanisms that may occur in San Bernardino County watersheds. Storm precipitation is generally classified by the mechanism that elevates the moisture-rich air mass, causing it to cool. For example, frontal precipitation occurs when warm, moisture-rich air is lifted along a frontal surface over a region of cooler air. Likewise, nonfrontal precipitation refers to storm precipitation that is not associated with fronts. Convective precipitation is caused by the upward movement of warm air through dense, cold air, such as thunderstorms over the desert. Rainfall intensities related to this process range from light showers to catastrophic cloudbursts. Orographic precipitation occurs when moisture-rich air is forced to rise and cool as it moves over mountain barriers.

2.2. Mechanism for Cooling

Each of the elevation mechanisms discussed above involves a cooling process associated with the resulting precipitation. Cyclonic cooling, for example, is generally classified based on its association with either frontal or nonfrontal effects.

The nonfrontal grouping is linked to the convergence and subsequent elevation of air accompanying a low-pressure area. Nonfrontal cyclonic precipitation of extratropical origin is typically associated with moderate-intensity rainfall (or snowfall) of long duration. Such storms may last for several days and deliver anywhere from 1 inch to over 6 inches of precipitation. In contrast, nonfrontal cyclonic precipitation of tropical origin can produce more than twice the rainfall in half the time.

Orographic cooling occurs when moisture-laden air is lifted along a mountain barrier, causing it to expand and cool at higher elevations where the pressure is lower. Due to the mountain barrier, a “rain shadow” may form on the leeward side. Generally, the windward sides of mountain barriers are cloudier, receive more precipitation, and experience smaller temperature fluctuations than the leeward sides.

When vertical instability of moisture-laden air is induced by surface heating, a convection current develops. Convective precipitation is typically associated with short-duration, high-intensity events and can lead to catastrophic flooding.

Atmospheric rivers (ARs) are an important source of moisture for San Bernardino County. ARs are long, narrow, and transient corridors of strong horizontal water vapor transport typically associated with a low-level jet stream ahead of the extratropical cyclone cold front (American Meteorological Society 2023). Landfalling ARs produce a wide variety of impacts worldwide. In California, the impacts range from beneficial precipitation (e.g., water supply, power generation) to hazardous precipitation that causes floods and landslides (Dettinger 2013, Ralph et al. 2006).

While ARs are mostly known for their long-duration, median-intensity precipitation, some landfalling ARs might present periods of short-duration, high-intensity precipitation caused by any of the mechanisms described above. Usually, along the U.S. west coast, these periods of short-duration and high-intensity precipitation are accompanied by a frontal passage with narrow cold frontal rainbands. High-intensity precipitation events are then embedded within the more typical long-duration, median-intensity precipitation.

San Bernardino County also receives moisture from the North American Monsoon (NAM), which becomes active in the U.S. Southwest in the summer. The NAM brings rain and thunderstorms to the desert. The eastern part of the county is on the northwestern edge of the NAM influence and typically does not receive as much rain from the NAM.

Due to the mixing characteristics of the storm events in the county, this Manual does not provide standards for each type of precipitation. Instead, the information in this Manual accounts for the fact that multiple storm mechanisms might occur at the same time and be embedded within each other.

2.3. Precipitation Depth, Duration, and Frequency Definitions

Due to the apparent randomness of precipitation patterns and intensities, a strictly deterministic analysis of precipitation quantities is not possible. Consequently, a statistical evaluation is generally used. In statistical analysis, the following definitions of precipitation depth, duration, and frequency are applied:

Precipitation Depth: The amount of precipitation occurring during a specified duration of storm time. Precipitation depth is usually expressed in inches.

Duration: The specified length of storm time under study. Duration may be expressed in any time unit, such as seconds, minutes, hours, days, or seasons.

Frequency: The rate at which events with a specified precipitation depth and duration occur. Frequency is expressed in terms of either the return period or the exceedance probability, both of which are defined below.

Intensity-Duration: The average precipitation intensity for a specified duration, obtained by dividing the precipitation depth by the duration.

Critical Duration: In a design storm event for a hydraulic structure, it is essentially the “time of concentration,” which is the time it takes for water deposited at the most remote part of a watershed to flow to the structure, outlet, or a specific location.

In hydrologic analysis, rainfall intensity is usually the most important parameter. This value relates both precipitation volume to storm duration and storm runoff to storm precipitation, with intensity serving as an upper bound for the watershed runoff rate. However, to provide a reasonable level of flood protection, the statistical concept of return frequency is used, which aids in assigning a probabilistic meaning to a precipitation event.

The following definitions are used in this Manual:

Exceedance (Cumulative) Probability: The probability that a precipitation event of a specified depth and duration will be exceeded in 1 year.

Return Period (Recurrence Interval): The long-term average number of years between occurrences of an event of a given depth and duration, either equaled or exceeded.

The exceedance probability (p) and return period (T) are related as shown in **Equation 2-1**:

Equation 2-1 Exceedance Probability and Return Period Relationship

$$p = \left(\frac{1}{T}\right)$$

where:

p = Exceedance probability
T = Return period

From the above definitions, it can be argued that a 100-year precipitation event, for example, will not necessarily occur once every 100 years, but rather has a finite probability of occurring in several consecutive years.

2.4. Measurement and Synthesis of Precipitation Data

In hydrologic studies, it is essential to analyze the maximum precipitation intensities occurring across a watershed. By examining historical records of maximum rainfall intensities over various durations, a statistical interpretation can be applied to estimate maximum rainfall depths and intensities as a function of storm duration and return frequency.

San Bernardino County operates and maintains both automatic recording and standard (manual) rain gauges throughout the region, compiling the data in its annual Hydrologic Data Report. Additional sources of precipitation data include the U.S. Weather Bureau, the U.S. Army Corps of Engineers (USACE), the U.S. Geological Survey, and other governmental and private cooperative weather observers.

For each automatic recording rain gauge, precipitation records are analyzed to determine the annual maximum rainfall depth for several key durations (e.g., 5 minutes, 10 minutes, 15 minutes, etc.). These data points are then arranged in ascending order for each storm duration, spanning the historical record of the gauge, and plotted on normal probability paper. Using this accumulated rainfall depth-duration data, statistical models can be applied to assign return frequencies (or return periods) to observed values and estimate maximum rainfall depths for higher, unmeasured return frequencies (e.g., the 100-year return frequency). The resulting values for each rain gauge are commonly referred to as "point precipitation" to distinguish them from average precipitation values over larger areas.

However, several factors contribute to the potential underestimation of true maximum point rainfall intensities. These include the misalignment of peak storm intensities with rain gauge locations, the sparse distribution of the gauge network (which may miss small, highly intense rainfall events), potential mechanical failures of gauging devices, and environmental variables such as wind effects. As a result, the rainfall data can generally be assumed to underestimate the actual maximum point rainfall intensities.

2.5. NOAA Data and Precipitation Frequency Estimates

The National Oceanic and Atmospheric Administration (NOAA) publishes the NOAA Atlas 14 (Perica et al. 2011) series, which provides precipitation frequency estimates. NOAA Atlas 14 offers point precipitation estimates for various durations, ranging from 5 minutes to 60 days, and for different storm frequencies, from 1-year to 1,000-year events. These estimates represent the depth of precipitation expected to occur, on average, over a specific area and duration for a given storm frequency.

The most recent NOAA Atlas 14 for California was published in 2011 and revised in 2014, incorporating precipitation data through October 2010. The primary data sources used in its development include:

Rain gauge measurements: Point precipitation measurements spanning extensive historical records. These data points vary in temporal resolution, ranging from 15-minute intervals to daily totals.

Gridded maps of mean annual precipitation (Daly et al. 2021): These maps are used to convert discrete point measurements into a continuous spatial dataset, as illustrated in **Figure 2-4**.

NOAA Atlas 14 provides precipitation estimates without distinguishing between rainfall and snowfall. In certain cases, this distinction may be important, as rainfall generates immediate runoff, while snowfall typically accumulates and melts later in the season. A more detailed discussion of snow-related considerations is provided in **Section 2.14**.

The County maintains an extensive rain gauge network with high temporal resolution. This network currently comprises 211 sites, with records dating back to October 2, 1883. The County contributed precipitation data from this network to the development of NOAA Atlas 14 for California, ensuring that estimates for the region were based on the highest-quality data available through October 2010.

At the time of this Manual’s publication, NOAA Atlas 14 is the recommended standard for precipitation frequency estimates, based on the performed analysis. However, NOAA Atlas 14 should be considered an approximation of point precipitation values and should be verified against all available rainfall data sources. Any deviation in the use of the provided rainfall data must be approved by the County.

The County reserves the right to update this standard without requiring a full revision of the Manual. Any future NOAA precipitation data releases or updates are considered automatically acceptable for use upon publication, unless otherwise specified by the County, and do not necessitate issuance of a Manual update. Furthermore, the County retains the authority to request supplemental analysis on a project-specific basis to verify precipitation estimates using gauge data.

2.5.1. Precipitation Data and Estimation Methods

Precipitation data can be obtained either as point or gridded data using NOAA precipitation data. Point precipitation data can be used for small watersheds (≤1 square mile (sq. mi.)), though gridded data converted to areal average precipitation are acceptable and preferred. Larger watersheds (>1 sq. mi.) should use gridded data converted to areal average precipitation within the watershed. The following section outlines the differences between these methods and serves as a guide for obtaining and adjusting precipitation estimates for various frequencies and durations.

The following steps provide a structured approach for acquiring and processing precipitation data:

1. Obtain Precipitation Data

   Retrieve precipitation estimates from NOAA data sources (refer to **Sections 2.5.1.1** and **2.5.1.2**).
2. Apply the Depth-Area Reduction Factor (DARF)

   This factor applies only to watersheds with an area exceeding 1 sq. mi. For further details, refer to **Chapter 5**.

This methodology ensures that precipitation estimates are as accurate and representative as possible for hydrologic analysis.

2.5.1.1. Point Precipitation

The NOAA precipitation data website (PF Map: Contiguous US) is the recommended standard for point precipitation values. To ensure accessibility in cases where the NOAA precipitation data website is unavailable, the County has developed isohyetal maps based on NOAA Atlas 14 data in various formats, available upon request from the Flood Control District.

Figure 2-2 provides a method for estimating precipitation values for intermediate return periods between the 2-year and 100-year storms based on corresponding precipitation depths. To use the figure, plot the precipitation values for the 2-year and 100-year storms. A straight line is then drawn to connect these points, allowing interpolation of precipitation estimates for other return periods along the line. This figure should only be used if the required data cannot be obtained from the NOAA website.

Point precipitation data should not be used for watersheds exceeding 1 sq. mi., as it does not adequately represent spatial variability in precipitation.

2.5.1.2. Areal Average Precipitation

For large watersheds, precipitation variability can significantly impact hydrologic analyses, making point precipitation estimates insufficient for accurate assessments. The use of geographic information system (GIS) tools in conjunction with gridded precipitation data from the NOAA website provides a more reliable methodology by enabling spatially distributed analysis and calculating the average precipitation across the watershed. This approach is recommended for watersheds exceeding 1 sq. mi., ensuring more precise and representative precipitation estimates for hydrologic analysis.

NOAA grid data can be accessed through the Hydrometeorological Design Studies Center website (https://hdsc.nws.noaa.gov/pfds/pfds_gis.html). The website provides compressed ASCII grid files with spatially interpolated precipitation frequency estimates based on both the partial duration series and the annual maximum series.

Metadata files in Federal Geographic Data Committee compliant XML format, along with projection files (*.prj), are automatically downloaded with the corresponding grids. These files can be obtained using the pull-down menu (as shown in **Figure 2-1**).

The following parameters should be specified:

- **Volume**: 6 – California
- **Type:** Precipitation frequency estimates
- **Series**: Partial duration series
- **Average recurrence interval:** Includes 1-, 2-, 5-, 10-, 25-, 50-, 100-, 200-, 500-, or 1,000-year
- **Duration:** Available durations include 5-minute, 10-minute, 15-minute, 30-minute, 60-minute, 2-hour, 3-hour, 6-hour, 12-hour, 24-hour, 2-day, 3-day, 4-day, 7-day, and up to 60-day

  Note: Gridded data must be divided by 1,000 before use.

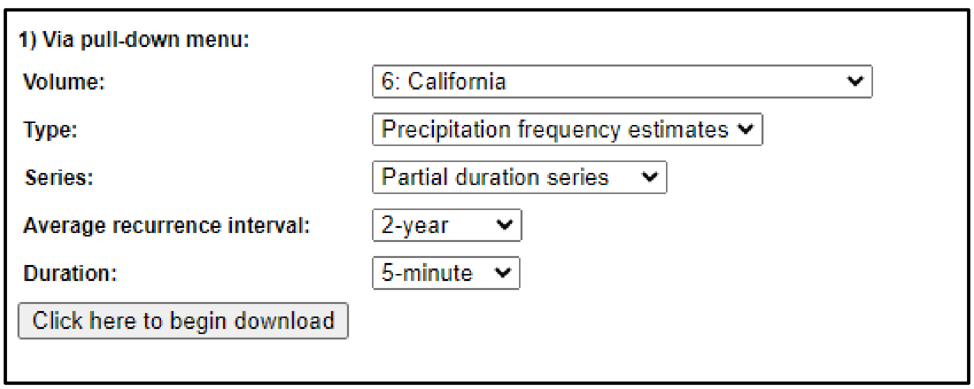


Figure 2-1. Pull-Down Menu to Download Gridded Maps

There are three common methods for using gridded precipitation data in hydrologic modeling: (1) point precipitation, (2) area-averaged precipitation, and (3) rain-on-grid. The use of point precipitation, which extracts a precipitation depth at a specific point considered representative of the watershed, should be reserved for planning purposes or for small watersheds that do not exceed 1 sq. mi. For hydrologic studies, it is typically recommended to use either spatially averaged precipitation for the modeled subbasin(s) or rain-on-grid (i.e., distributed modeling). Each type of precipitation data (point, area-averaged, and rain-on-grid) should account for the DARF, if applicable.

This Manual does not provide detailed recommendations for performing distributed hydrologic evaluations. Therefore, if distributed hydrologic evaluations are used for a specific project, the methods being implemented must first be approved by the County.

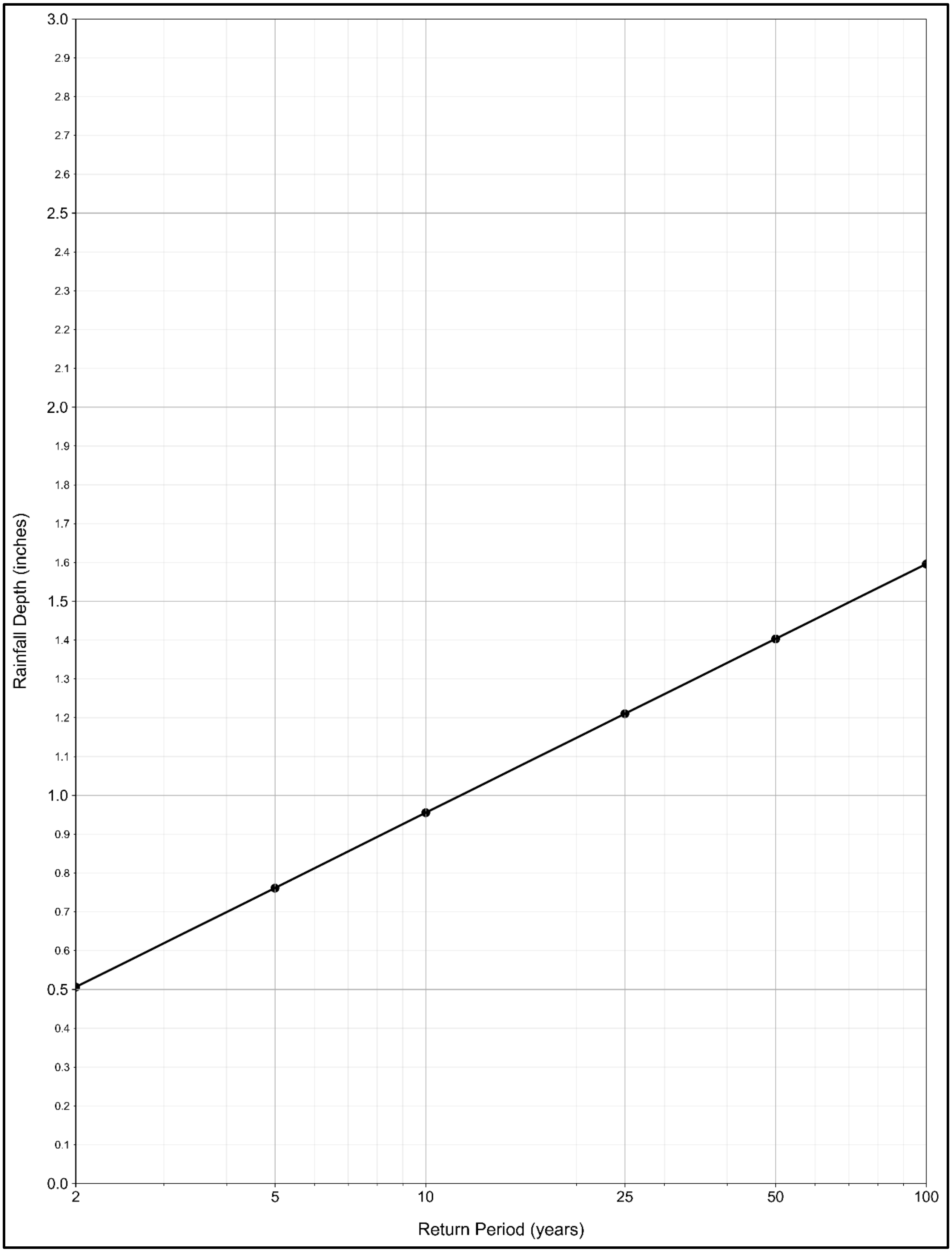


Figure 2-2. Rainfall Depth Versus Return Period

2.6. Random Nature of Point Precipitation Data

When analyzing rain gauge records over time, a highly variable range of fluctuations is observed. These random variations are so pronounced that they obscure any long-term patterns or periodic trends that may exist. As a result, using point precipitation data effectively requires a combination of probabilistic and deterministic approaches.

The duration and magnitude of individual storm events are treated as probabilistic, while the internal structure of a storm may be considered deterministic. Furthermore, the origin of storm events such as convective or orographic processes adds complexity to the development of a comprehensive analytical framework.

2.7. Event Depth-Duration

For most hydrologic studies, the key relationship of interest is the precipitation depth for a given rainfall event duration. As outlined in NOAA precipitation frequency guidance, this relationship includes total precipitation from storms of the specified duration, as well as depths from independent, continuous partial storm durations.

This information is commonly represented through event depth-duration curves, which are constructed by ranking all storm events of a given duration in descending order of rainfall depth. From this ranking, an estimate can be made of the frequency with which a precipitation depth for a given duration will be equaled or exceeded over a specified number of years.

Once event depth-duration curves are developed, a second set of precipitation depth-duration (or intensity-duration) curves can be derived. These curves, for a given return period, represent the maximum precipitation depth (or intensity) that can occur from any storm as a function of duration.

2.8. Intensity-Duration Curves

Intensity-duration data are essential for applying the Rational Method. These data are typically presented as curves depicting rainfall intensity (in inches per hour) versus storm duration (in minutes). For durations under 1 hour, intensity-duration data generally plot as a straight line on log-log paper, with curves for various return periods running parallel to one another.

Intensity-duration curves for a watershed can be developed by estimating the appropriate area-averaged 1-hour point precipitation values from NOAA precipitation data.

Using **Figure 2-3** (log-log paper), the 1-hour point precipitation value is plotted, and a straight line is drawn with a slope of 0.5. However, as with point precipitation data, rainfall records should be carefully analyzed to determine the most appropriate slope for the intensity-duration plot.

Since most rain gauge data interpretations are based on stations with relatively short historical records, caution must be exercised when establishing intensity-duration curves. For Rational Method studies, a minimum storm duration of 5 minutes is used, as indicated in **Figure 2-3**. The validity of the curve shown in **Figure 2-3** was verified based on NOAA Atlas 14 data.

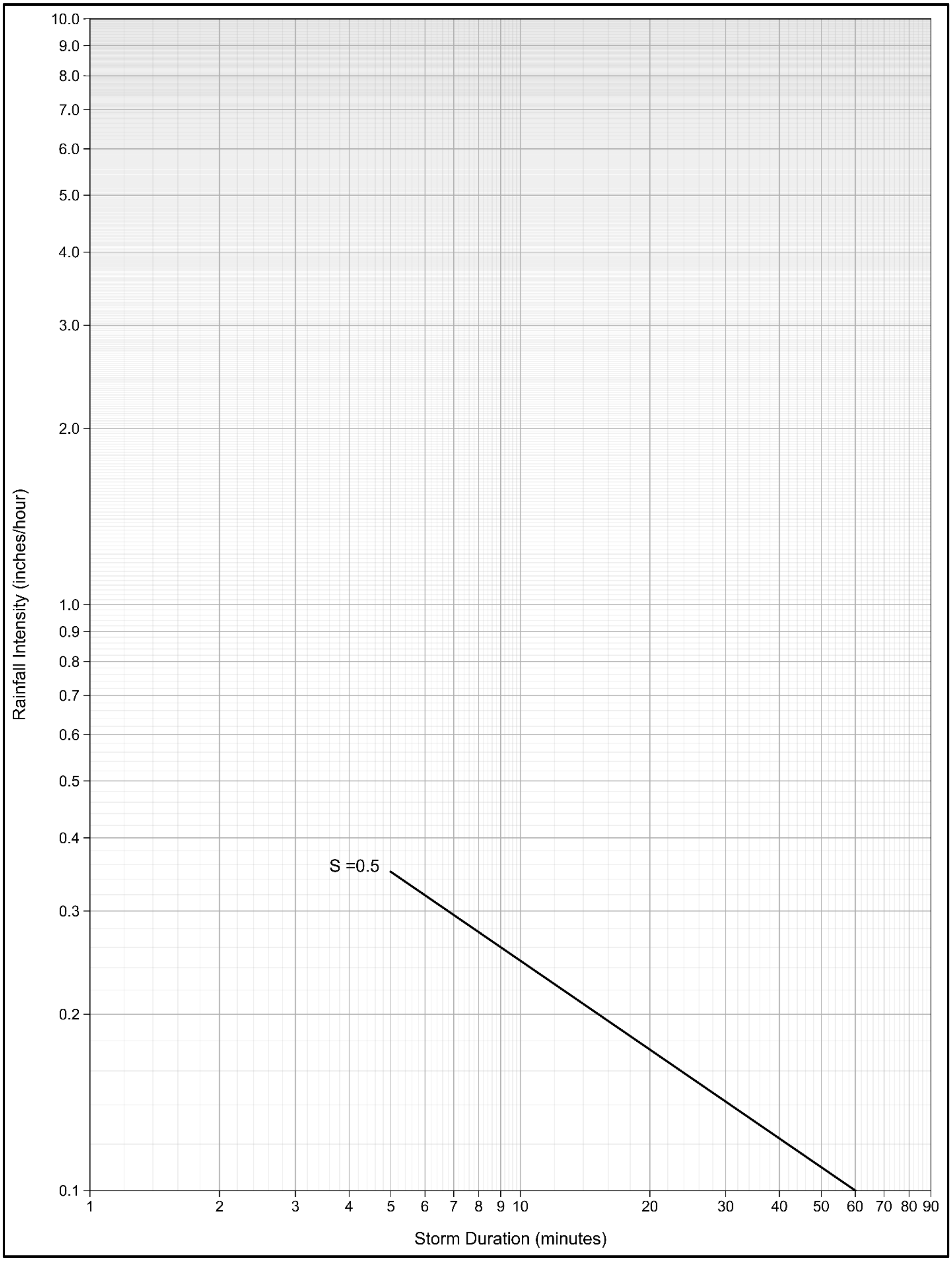


Figure 2-3. Rainfall Intensity Versus Duration Curve

2.9. Synthetic 24-Hour Critical Storm Pattern

The U.S. Department of Agriculture Natural Resources Conservation Service (NRCS) (formerly the Soil Conservation Service (SCS)) developed dimensionless critical storm patterns using rainfall frequency data from the NOAA, National Weather Service (NWS), and other reliable sources. These rainfall frequency data were derived for areas less than 400 sq. mi., for durations up to 24 hours, and for return periods ranging from 1 to 100 years.

These critical storm patterns are based on generalized precipitation depth-duration-frequency (DDF) relationships presented in technical publications by the NWS. Precipitation depths for durations ranging from 1 minute to 24 hours are used to develop the storm patterns. Incremental precipitation depths are calculated using 30-minute intervals. For instance, the 30-minute depth is subtracted from the 1-hour depth, and the 1-hour depth is subtracted from the 1.5-hour depth, and so on.

The storm patterns are structured as follows:

- The maximum 30-minute depth is contained within the maximum 1-hour depth.
- The maximum 1-hour depth is contained within the maximum 1.5-hour depth, and this nesting continues for longer durations.

Since all critical precipitation depths are incorporated within the storm pattern, this method is considered appropriate for hydrologic design in both small and large watersheds.

The County's design storm pattern is based on a modified version of the NRCS 24-hour storm pattern. This pattern is adjusted to more accurately represent local precipitation DDF characteristics, ensuring that the nested intervals align with observed local rainfall records. Additionally, the pattern is further modified to account for watershed area effects, particularly by adjusting shorter-duration areal averaged precipitation values to reflect the DARF.

The procedures used to construct the 24-hour storm pattern and to determine the associated rainfall depths adjusted for depth-area effects follow methodologies established by the USACE, as outlined in the Hydrologic Engineering Center-Hydrologic Modeling System (HEC-HMS) User's Manual (USACE 2024). Further details on the 24-hour storm pattern and the necessary adjustments for depth-area effects are provided in **Chapter 5**.

2.10. Longer Duration Rainfall Data

The County's design storm criteria extend to multiday design storms when necessary to evaluate detention basin characteristics (see **Chapter 8**). The following table provides the ratio of daily rainfall to the peak 24-hour mass rainfall. This ratio should be used when rainfall data are insufficient to directly provide the required quantities. **Table 2-1** presents an average relationship developed from the Claremont Pomona College and Lytle Creek PH rain gauges.

Table 2-1. Multiday Rainfall Mass Ratios

Rainfall Duration	Ratio to Peak 24-Hours
Peak 24-hours	1.00
(Peak 48-hours)-(Peak 24-hours)	0.36
(Peak 72-hours)-(Peak 48-hours)	0.19
(Peak 96-hours)-(Peak 72-hours)	0.15
(Peak 120-hours)-(Peak 96-hours)	0.10

2.11. Spatial Variability

San Bernardino County presents large spatial variability in precipitation, which is demonstrated by the mean annual precipitation data shown in **Figure 2-4** (Daly et al. 2021). Mean annual precipitation varies from 53.1 inches in the vicinity of San Gorgonio Mountain (elev. 11,499 feet) to 2.6 inches in Death Valley. ARs contribute up to 50% of the annual precipitation volumes in the western parts of the county and almost no volume in the eastern parts (Dettinger et al. 2011).

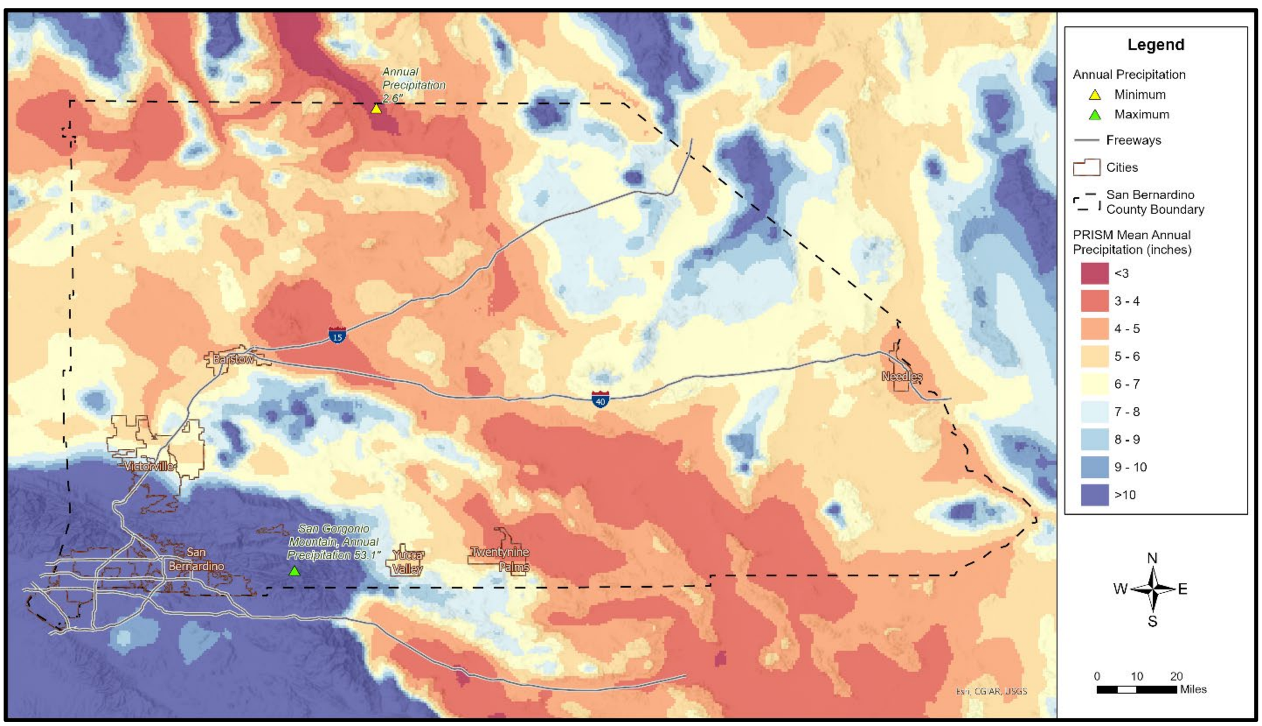


Figure 2-4. Mean Annual Precipitation Map

2.12. Storm Frequency Relationship Ratios

Storm frequency relationship ratios (SFRRs) were established to associate the 100-year storm event to the 2-year, 10-year, 25-year, 50-year, 500-year, and 1,000-year storm events. The SFRRs were determined using NOAA Atlas 14 data (Perica et al. 2011) and are presented in **Table 2-2**.

Table 2-2. 100-Year Ratios

Return Period (years)	SFRR
2	0.32
10	0.57
25	0.73
50	0.86
500	1.37
1,000	1.55

The implementation of these ratios to calculate other return period storm flows requires the 100-year storm event flow; this flow value is then multiplied by the SFRR corresponding to the desired x-year storm event using **Equation 2-2**:

Equation 2-2 100-Year Storm Flow Adjustment

$$Q_{x\text{-Year}} = (SFRR_{x\text{-Year}})(Q_{100\text{-Year}})$$

where:

$Q_{x\text{-Year}}$ = Peak flow rate of desired x-year storm event (cfs)
$SFRR_{x\text{-Year}}$ = Storm Frequency Relationship Ratio (unitless)
$Q_{100\text{-Year}}$ = 100-year peak flow rate (cfs)

The SFRRs are considered approximate and should only be used for planning purposes; they do not ultimately replace the need to perform detailed x-year storm event hydrologic calculations.

2.13. Climate Change

Climate change continues to influence regional hydrologic patterns by altering the intensity, frequency, and duration of precipitation events. These shifts affect watershed response, peak flows, groundwater recharge, and overall flood risk. To ensure hydrologic analyses remain current and scientifically supported, the County adopts the most recent NOAA precipitation frequency data as its official source. NOAA's revisions are based on ongoing analysis of national and local gauge records—including data collected through the County's own gaging network—so each new dataset is automatically incorporated upon release. By codifying this practice in this hydrology manual, the County ensures consistency across all technical studies and design applications. Looking ahead, NOAA is developing precipitation frequency products that more directly evaluate long-term climatic changes, and these will likewise be adopted to maintain alignment with the best available science. This approach ensures that hydrologic evaluations reflect evolving climatic conditions and support resilient infrastructure planning.

2.14. Considerations for Snow

In certain regions of the county, precipitation can fall as snow instead of rain. In these areas, precipitation frequency estimates that do not differentiate between liquid and solid precipitation could potentially result in overestimation or underestimation of design parameters (runoff volume and peak flow), unless the design parameters are highest for rainfall only events. Overestimation happens when precipitation falls as snow that accumulates on the ground and melts later. Underestimation happens in areas that are subject to rain-on-snow events, where snowmelt augments the rainfall rate. High temperatures are possible in early spring, resulting in snowmelt runoff without additional precipitation.

Ideally, snow coverage and snowmelt frequency are determined based on direct snow measurement. However, gauges that differentiate snowfall from rainfall are limited in number. Those gauges also present a limited period of record. Therefore, to evaluate the effects of snow on precipitation design, the County relied on two previous peer reviewed studies that investigated gridded snow data:

- Cho and Jacobs (2020) applied two widely accepted gridded snow products to evaluate the impacts of snow on flood design. The authors performed frequency analysis for two variables: (1) Snow water equivalent (SWE), and (2) 1- and 7-day rainfall combined with snowmelt frequencies across the United States for locations where snowpack is present.

  Figure 2-5 shows the 100-year SWE values in inches for the county based on Cho and Jacobs (2020). The 100-year SWE varies from 0 at low elevations to 93 inches in the highest mountain areas. However, SWE, especially at high elevations, may take weeks or even months to melt. Therefore, snowmelt rates are more relevant than SWE for flood design purposes.

  Figure 2-6 shows the 100-year, 1-day snowmelt plus precipitation rates for the county. Near the peaks of the San Bernardino Mountains, snowmelt combined with precipitation estimates exceeds the NOAA Atlas 14 values by a small amount. The maximum 100-year, 1-day snowmelt plus precipitation rate is 18.5 inches/day, while the maximum NOAA Atlas 14 value is 17.67 inches/day. That relatively small difference (5%) could be the result of using different datasets when developing the estimate. At Big Bear City, the NOAA Atlas 14 100-year, 1-day precipitation value is 11.5 inches compared to the snowmelt plus precipitation value of 10 inches.

  Cho and Jacobs (2020) also provide the 25-year, 1-day, and the 100-year and 25-year, 7-day snowmelt plus precipitation rate maps (not included in this Manual).

- Sun et al. (2022) applied intensity-duration-frequency curves to estimate the contribution of rain, snowmelt, and rain-on-snow events for different durations and return periods in the contiguous United States (CONUS). The authors classified each area in the CONUS based on the different dominant mechanisms that control flood generation. Extreme floods in San Bernardino County are primarily controlled by rainfall events; therefore, estimates provided by NOAA Atlas 14 are appropriate.

Based on these two peer reviewed studies (Cho and Jacobs 2020, Sun et al. 2022), the County concluded that extreme floods in San Bernardino County are mostly controlled by rainfall events. The exceptions are near the highest elevations of the San Bernardino Mountains, where a maximum underestimation due to rain-on-snow events is approximately 15% for a very limited number of locations (less than 1% of the county). Uncertainties of the expected underestimation are high, and they only occur in currently unpopulated areas. Therefore, the County does not currently recommend any corrections to NOAA Atlas 14 due to rain-on-snow processes.

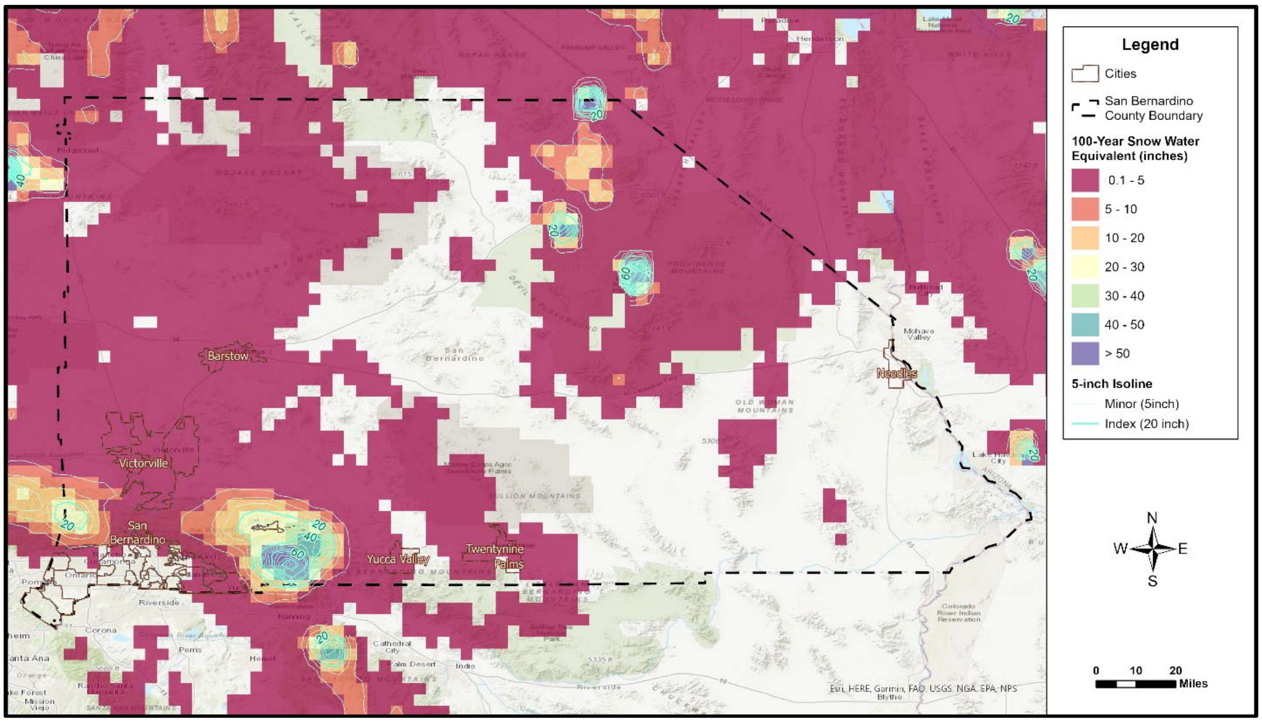


Figure 2-5. 100-Year Snow Water Equivalent

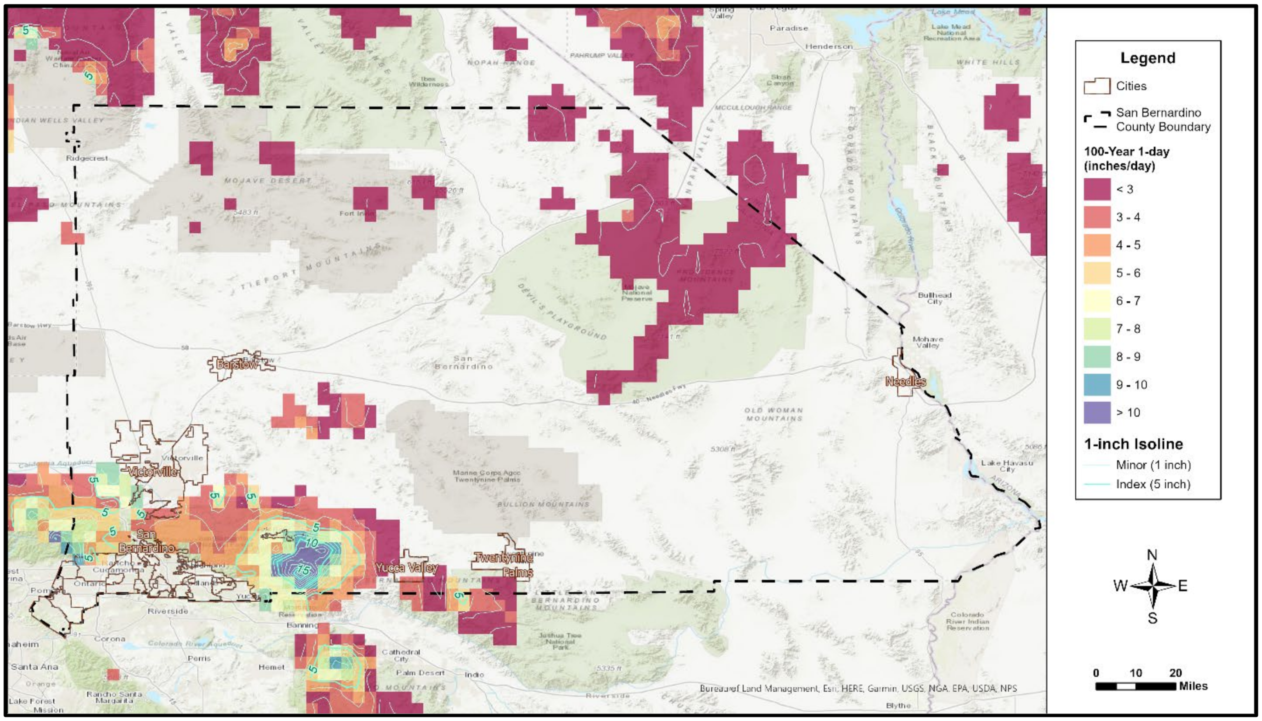


Figure 2-6. 100-Year, 1-Day Snowmelt (Inches/Day)

CHAPTER 3
LOSSES

3.1. Watershed Losses

Watershed precipitation losses primarily include depression storage, vegetation interception and transpiration, minor amounts of evaporation, and infiltration. Infiltration is the process of water entering the soil surface and percolating into the soil. This infiltrated water may then be consumptively used by vegetation, percolate further downward to groundwater storage, or exit the soil surface as seepage. Seepage from streambank storage is the primary source of baseflow in perennial streams.

For hydrologic modeling purposes, watershed losses are grouped into two components: (1) infiltration, and (2) initial abstraction, which includes all the losses except infiltration.

This chapter presents computational watershed loss methods and data to quantify infiltration and initial abstraction.

3.2. Hydrologic Soil Groups (HSGs)

The major factor affecting loss rates is the nature of the soil itself. The soil surface characteristics, its ability to transmit water to subsurface layers, and total storage capacity are all major factors in controlling the infiltration rate and initial abstraction parameter values of a particular soil. Soils are classified into four hydrologic soil groups (HSGs) as follows (McCuen 1982, SCS 1969):

Group A: This group consists of soils with low runoff potential. These soils have high infiltration rates even when thoroughly wetted and consist primarily of deep, well-drained sands or gravels. These soils have a high rate of water transmission.

Group B: These soils have moderate infiltration rates when thoroughly wetted and mainly consist of moderately deep to deep, moderately well to well-drained sandy loam soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.

Group C: These soils have slow infiltration rates when thoroughly wetted and primarily consist of silty loam soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.

Group D: This group includes soils with high runoff potential. They have very slow infiltration rates when thoroughly wetted and primarily consist 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 nearly impervious material. These soils have a very slow rate of water transmission.

3.2.1. Soil Maps

San Bernardino County HSG data are provided in **Figure 3-1**. The data sources for the map include the Natural Resources Conservation Service (NRCS) Soil Survey Geographic, the State Soil Geographic, and the Raster Soil Survey databases, all of which are accessible through the gridded National Soil Survey Geographic Database (gNATSGO) (Soil Survey Staff 2021). In addition, soil data from the 1986 edition of the Manual (SBCFCD 1986) were used in a few locations where more recent data are unavailable. The NRCS sometimes designates multiple HSGs for the same area (e.g., both Group A and Group D). In these cases, the more conservative HSG should be used for hydrologic calculations (i.e., higher runoff potential).

The HSG can be identified using the map provided in this Manual or the digital files available upon request from the County.

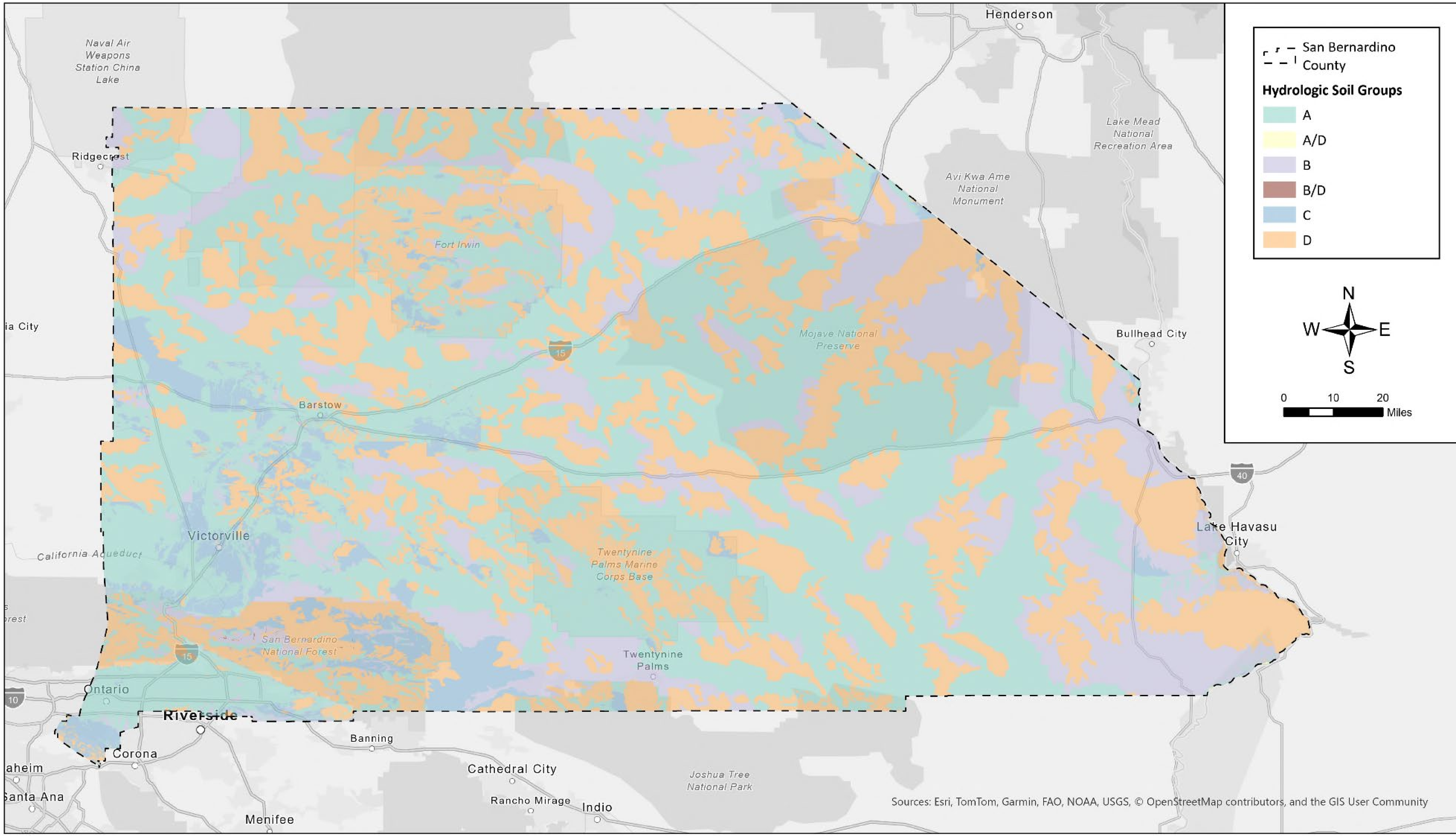


Figure 3-1. Hydrologic Soils Group Map

3.3. Soil Cover and Hydrologic Conditions

The type, quality, and density of vegetation or ground cover within a watershed impact the infiltration rate of soils. In most cases, the existing watershed cover type and quality can be determined using publicly available imagery and/or data gathered during field visits. **Table 3-1** provides definitions of specific cover types. These ground cover types can be further refined based on cover quality as follows:

Poor: Heavily grazed or regularly burned areas. Less than 50% of the ground surface is protected by plant cover or brush and tree canopy.

Fair: Moderate cover with 50% to 75% of the ground surface protected by vegetation.

Good: Heavy or dense cover with more than 75% of the ground surface protected by vegetation. This should be used in the ultimate planned open spaces.

Table 3-2 provides the curve number (CN) values for various types and quality of ground cover and the table footnotes provide additional instruction for their use. Note that impervious areas shall be assigned a CN of 98.

Table 3-1. SCS Soil Cover Type Descriptions

Soil Cover Type	Description
Residential or Commercial Landscaping (Lawn, Shrubs, etc.)	The pervious portions of commercial establishments, single and multiple family dwellings, trailer parks, and schools where the predominant land cover is lawn, shrubbery, and trees.
Row Crops	Lettuce, tomatoes, beets, tulips, or any field crop planted in rows far enough apart that most of the soil surface is exposed to rainfall impact throughout the growing season. At plowing, planting, and harvest times it is equivalent to fallow.
Small Grain	Wheat, oats, barley, flax, etc. planted in rows close enough that the soil surface is not exposed except during planting and shortly thereafter.
Legumes	Alfalfa, sweetclover, timothy, etc. and combinations are either planted in close rows or broadcast.
Fallow	Fallow land is land plowed but not yet seeded or tilled.
Crop Residue	Crop material left over after harvesting activities.
Woodland – Grass	Areas with an open cover of broadleaf or coniferous trees usually live oak and pines, with the intervening ground space occupied by annual grasses or weeds. The trees may occur singly or in small clumps. Canopy density, the amount of ground surface shaded at high noon, is from 20–50%.
Woodland	Areas on which coniferous or broadleaf trees predominate. The canopy density is at least 50%. Open areas may have a cover of annual or perennial grasses or of brush. Herbaceous plant cover under the trees is usually sparse because of leaf or needle litter accumulation.
Chaparral	Land on which the principal vegetation consists of evergreen shrubs with broad, hard, stiff leaves such as manzanita, ceanothus, and scrub oak. The brush cover is usually dense or moderately dense. Diffusely branched evergreen shrubs with fine needle-like leaves, such as chamise and redshank, with dense high growth are also included in this soil cover.
Annual Grass	Land on which the principal vegetation consists of annual grasses and weeds such as annual bromes, wild barley, soft chess, ryegrass, and filaree.
Irrigated Pasture	Irrigated land planted to perennial grasses and legumes for production of forage, and which is cultivated only to establish or renew the stand of plants. Dry land pasture is considered as annual grass.
Meadow	Land areas with seasonally high water table, locally called cienegas. Principal vegetation consists of sod-forming grasses interspersed with other plants.
Orchard (Deciduous)	Land planted to such deciduous trees as apples, apricots, pears, walnuts, and almonds.
Orchard (Evergreen)	Land planted to evergreen trees which include citrus and avocados and coniferous plantings.
Turf	Golf courses, parks, and similar lands where the predominant cover is irrigated mowed close-grown turf grass. Parks in which trees are dense may be classified as woodland.

Table 3-2. Hydrologic Soil Group Curve Numbers for Pervious Areas – AMC II

Cover Type[1]	Average Percent Impervious Area[2]	Quality of Cover[3]	Hydrologic Soil Group			
			A	B	C	D
Natural Covers						
Barren (Rockland, eroded, and graded land)			78	86	91	93
Chaparral, Broadleaf (Manzanita, ceanothus, and scrub oak)		Poor Fair Good	53 40 31	70 63 57	80 75 71	85 81 78
Chaparral, Narrowleaf (Chamise and redshank)		Poor Fair	71 55	82 72	88 81	91 86
Grass, Annual or Perennial		Poor Fair Good	67 50 38	78 69 61	86 79 74	89 84 80
Meadows or Cienegas (Areas with a seasonally high-water table, principal vegetation is sod-forming grass)		Poor Fair Good	63 51 30	77 70 58	85 80 71	88 84 78
Open Brush (Soft wood shrubs: buckwheat, sage, etc.)		Poor Fair Good	62 46 41	76 66 63	84 77 75	88 83 81
Woodland (Coniferous or broadleaf trees predominate, with canopy density at least 50%)		Poor Fair Good	45 36 30	66 60 55	77 73 70	83 79 77
Woodland, Grass (Coniferous or broadleaf trees with canopy density from 20–50%)		Poor Fair Good	57 43 32	73 65 58	82 76 72	86 82 79
Agricultural Covers						
Fallow (Land is plowed, but not tilled or seeded) Bare Soil Crop Residue[4]		 Poor Good	 77 76 74	 86 85 83	 91 90 88	 94 93 90
Legumes, Close Seeded (Alfalfa, sweetclover, timothy, etc.)		Poor Good	66 58	77 72	85 81	89 85
Orchards, Evergreen (Citrus, avocados, etc.)		Poor Fair Good	57 44 33	73 65 58	82 77 72	86 82 79

Cover Type[1]	Average Percent Impervious Area[2]	Quality of Cover[3]	Hydrologic Soil Group			
			A	B	C	D
Pasture, Dryland (Annual grasses)		Poor Fair Good	68 49 39	79 69 61	86 79 74	89 84 80
Pasture, Irrigated (Legumes and perennial grass)		Poor Fair Good	58 44 33	74 65 58	83 77 72	87 82 79
Row Crops (Field crops: tomatoes, sugar beets, etc.)		Poor Good	72 67	81 78	88 85	91 89
Small Grain (Wheat, oats, barley, etc.)		Poor Good	65 63	76 75	84 83	88 87
Developed (e.g., urban) Covers						
Open Space (lawns, parks, golf courses, cemeteries, etc.)		Poor Fair Good	68 49 39	79 69 61	86 79 74	89 84 80
Impervious Areas and Road Surfaces Parking lots, roofs, driveways, etc. (excludes R/W) Paved roads (excludes R/W) Gravel roads (includes R/W) Dirt Roads (includes R/W)			 98 98 76 72	 98 98 85 82	 98 98 89 87	 98 98 91 89
Desert Landscaping Natural (pervious areas only) Artificial (includes impervious weed barrier, desert shrub with 1 to 2-inch sand or gravel mulch overlayment)			 63 96	 77 96	 85 96	 88 96
Business Districts[5] Commercial Industrial	 85 72		 89 81	 92 88	 94 91	 95 93
Residential Districts (by lot size)[5] 1/8-acre or less (e.g., townhouses) 1/4-acre 1/3-acre 1/2-acre 1-acre 2-acre	 65 38 30 25 20 12		 77 61 57 54 51 46	 85 75 72 70 68 65	 90 83 81 80 79 77	 92 87 86 85 84 82
Newly Graded Areas			77	86	91	94

1. See **Table 3-1** for definition of cover types.
2. The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition.
3. Quality of cover definitions:
   - Poor – Heavily grazed, regularly burned areas, or areas of high burn potential. Less than 50% of the ground surface is protected by plant cover or brush and tree canopy.
   - Fair – Moderate cover with 50–75% of the ground surface protected.
   - Good – Heavy or dense cover with more than 75% of the ground surface protected.
4. Crop residue cover only applies if residue covers 5% (minimum) of the surface throughout the year.
5. The CN values for business or residential districts may be used as presented above. Alternatively, a composite CN can be calculated based on the directly connected impervious area (assumed CN = 98) and the CN for pervious areas (e.g., open space in good condition). The impervious percentage may be estimated using this table or determined manually from digital data (e.g., impervious polygon shapefiles).

3.4. Watershed Development Conditions

For proposed condition analyses, land use should be based on the ultimate development of the watershed. The ultimate development of the watershed should normally be assumed, as watershed urbanization is reasonably likely within the expected life of most hydraulic facilities. Long-range master plans for the County and incorporated cities should be reviewed to ensure reasonable land use assumptions are implemented for the ultimate development of the watershed.

Performing comprehensive desktop analyses and site reconnaissance to confirm existing use and drainage patterns is recommended. These analyses should include particular attention to existing and proposed conditions (e.g., landscape practices in which some areas commonly use ornamental gravels underlain by impervious plastic materials in place of lawns and shrubs). The impervious cover values in **Table 3-2** are intended to be used as a guideline, and adjustments based on desktop analyses or site reconnaissance are encouraged.

3.5. Antecedent Moisture Condition (AMC)

Antecedent moisture condition (AMC) describes the relative wetness or dryness of a watershed before a storm event. Consequently, the AMC assumptions play a significant role in determining runoff peak flow rates and volumes.

The definitions for the AMC classifications are:

> **AMC I:** Lowest runoff potential. The watershed soils are dry enough to allow satisfactory grading or cultivation to take place.
>
> **AMC II:** Moderate runoff potential; represents average moisture conditions of watersheds.
>
> **AMC III:** Highest runoff potential. The watershed is practically saturated from antecedent rains. Heavy rainfall or light rainfall and low temperatures have occurred within the last 5 days.

The selection of an AMC for a hydrologic study will be based on the location of the watershed area; the AMC classification map is shown in **Figure 3-2**. If a watershed area straddles two AMC zones, the higher AMC will be applied if at least 30% of the area falls within that zone; otherwise, the lower AMC should be used.

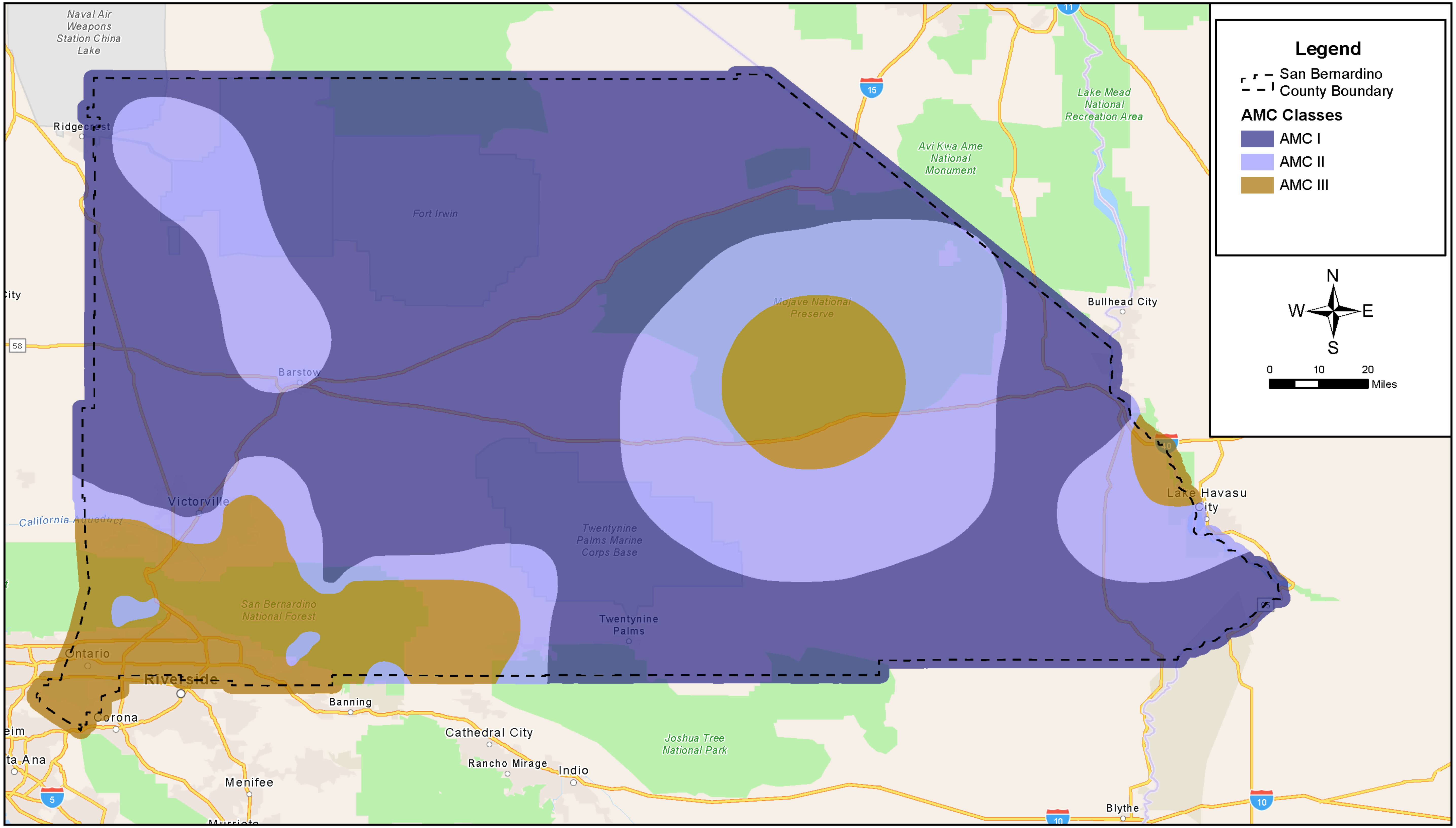


Figure 3-2. Antecedent Moisture Condition Map

3.5.1. Adjustment of Curve Numbers (CNs) for AMC

The CN values selected for a particular soil cover type and quality also depend on the assumed AMC.

The CN values listed in **Table 3-2** correspond to AMC II and require adjustments to represent either AMC I or AMC III.

Table 3-3 provides the necessary CN adjustments (using interpolation for intermediate CN values) to account for AMC changes for hydrologic studies in San Bernardino County.

Table 3-3. Curve Number for AMC Classification

CN for AMC Condition II	Corresponding CN for AMC Classification	
	I	III
100	100	100
95	87	99
90	78	98
85	70	97
80	63	94
75	57	91
70	51	87
65	45	83
60	40	79
55	35	75
50	31	70
45	27	65
40	23	60
35	19	55
30	15	50
25	12	45
20	9	39
15	7	33
10	4	26
5	2	17
0	0	0

3.6. Estimation of Loss Rates

In estimating loss rates for design hydrology, a watershed CN is determined for each soil cover within the watershed using **Table 3-2** and **Table 3-3**. A low CN indicates low runoff potential (i.e., high infiltration), and a high CN indicates high runoff potential (i.e., low infiltration).

The CN accounts for the major pervious surface loss rate factors such as:

- Hydrologic soil group (HSG)
- Land Cover type and quality
- Antecedent moisture condition (AMC)

Also included in the CN selection are the effects of “initial abstraction” (I_a), which represents the combined effects of other rainfall losses including depression storage, vegetation interception, evaporation, and transpiration, among other factors.

3.6.1. Estimation of Initial Abstraction (I_a)

The initial abstraction (I_a) for an area is a function of land use, treatment and condition, interception, infiltration, depression storage, and AMC. I_a can be estimated using the following equations (SCS 1969):

Equation 3-1 Initial Abstraction

$$I_a = 0.2S$$

where:

S = Total soil capacity estimate given by **Equation 3-2**

Equation 3-2 Total Soil Capacity

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

where:

CN = Area curve number

3.6.2. Estimation of Storm Runoff Yield

This section explores the method for computing the storm runoff yield fraction (Y_j), where the term “yield” refers to the effective storm runoff after losses. Storm runoff yield is typically determined using hydrologic models such as HEC-HMS rather than manual calculations. However, the County’s low loss method (see **Section 3.6.3**) requires additional calculations before implementation in HEC-HMS.

Given the CN for a watershed subarea (A_j), the corresponding x-hour storm runoff yield fraction, Y_j, is estimated using **Equation 3-3**:

Equation 3-3 X-Hour Storm Runoff Yield Fraction

$$Y_j = \frac{(P_x - I_a)^2}{(P_x - I_a + S)P_x}$$

where:

Y_j = X-hour storm runoff yield fraction for subarea A_j
P_x = X-hour storm rainfall (inches)
I_a = Initial abstraction from **Equation 3-1**. If I_a is greater than P_x then Y_j is defined to be zero.
S = Total Soil Capacity from **Equation 3-2**

If the study area contains several CN designations, then the yield (Y) for the total area must represent the net effect of those curve numbers. This is accomplished by weighting each of the subarea yield values according to the respective areas as shown in **Equation 3-4**:

Equation 3-4 Runoff Yield Weighting

$$Y = (Y_1A_1 + \ldots + Y_mA_m)/(A_1 + A_2 + \ldots + A_m)$$

where:

Y = Weighted runoff yield fraction
Each Y_j from **Equation 3-3**
A_j = Watershed subarea

3.6.3. Low Loss Rate (F*)

In design storm runoff hydrograph studies, **Equation 3-5** is used to estimate the portion of rainfall attributed to watershed losses:

Equation 3-5 Runoff Losses

$$\bar{Y} = 1 - Y$$

where:

$\bar{Y}$ = Watershed low loss fraction
Y = Watershed x-hour storm runoff yield fraction computed from **Equation 3-3** or runoff yield weighting **Equation 3-4**

Using the low loss fraction, $\overline{Y}$, the corresponding low loss rate, F*, is given by:

Equation 3-6 Low Loss Rate

$$F^{*} = \overline{Y}I$$

where:

F^* = Low loss rate (inches/hour)
$\overline{Y}$ = Watershed low loss fraction
I = Rainfall intensity (inches/hour)

3.6.4. Infiltration Rates

Soil infiltration rates have been estimated for each of the soil groups by laboratory studies and measurements. These findings indicate that when initially dry soil is wetted, its infiltration rate decreases over time. Under continuous heavy rainfall, this rate approaches a minimum (typically within about 30 minutes), representing the soil's infiltration capacity.

When sufficient stream gauge data are available, infiltration rates for unit hydrograph hydrology can be estimated from a study of rainfall-runoff relationships of major storms. For small area applications, onsite infiltration testing may be adequate. When this information is not available, infiltration rates for pervious areas can be estimated as a function of CN using **Figure 3-3**. Loss rates for pervious areas estimated from the **Figure 3-3** curves are generally consistent with values developed from rainfall-runoff reconstitution studies in San Bernardino County watersheds.

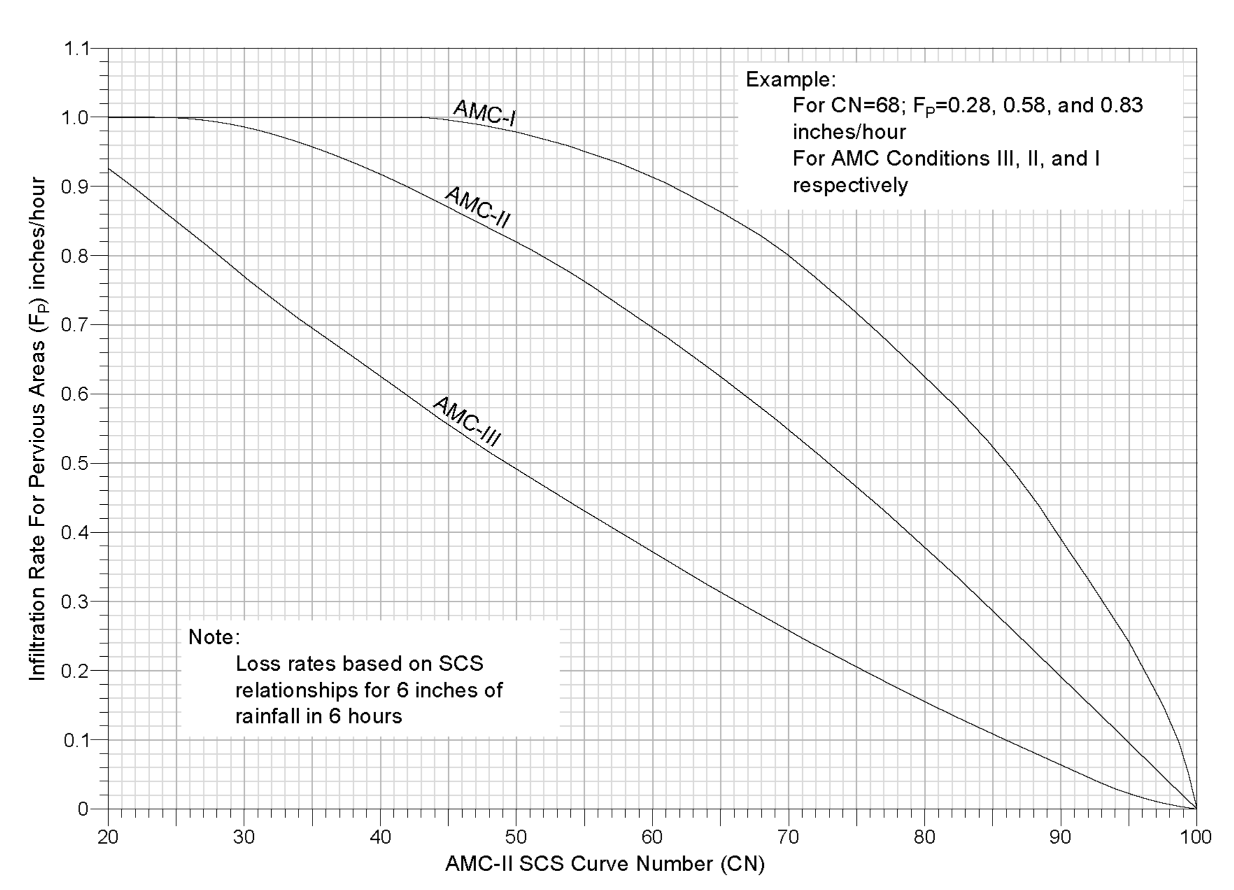


Figure 3-3. Infiltration Rate for Pervious Areas vs. SCS Curve Number

3.6.5. Estimation of Watershed Maximum Loss Rates (F_m)

The infiltration rate selected from **Figure 3-3** applies to the pervious area fraction of the watershed. The infiltration rate assumed for an impervious surface is 0.0 inches/hour. The maximum loss rate, F_m, for a watershed subarea is, therefore, given by:

Equation 3-7 Maximum Loss Rate

$$F_m = a_P F_P$$

where:

F_m = Maximum loss rate (inches/hour)
a_p = Pervious area fraction ($a_p = 1 - a_i$; where a_i = Impervious area fraction)
F_p = Infiltration rate for the pervious area (inches/hour)

If a watershed or subarea contains several F_p values, the composite F_m value is determined as a simple area average of all the F_m values.

3.6.6. Design Storm Loss Rates

In design storm runoff hydrograph analyses, a 24-hour storm hyetograph is used to define the temporal distribution of effective rainfall across the watershed. The effective rainfall quantities are determined by subtracting the watershed losses from the design storm rainfall.

The loss rate used for a study watershed considers both the maximum loss rate (F_m) and the low loss rate (F*) at each time step of the storm hyetograph. For most time steps, the low loss rate (F*) is used as the loss rate unless it exceeds the maximum loss rate (F_m). In that case, the maximum loss rate (F_m) is used as the loss rate. In other words, the lower value is used as the loss rate and (F_m) serves as the maximum loss rate.

Typically, in less frequent storm events (e.g., 100-year), F* will serve as the loss rate for the entire storm hyetograph except in the most intense time steps (i.e., typically around the peak), where F_m becomes the controlling value. However, for more frequent events (e.g., 2-year), F* often applies for the entire storm hyetograph. **Figure 3-4** illustrates the loss rate function used with the design storm.

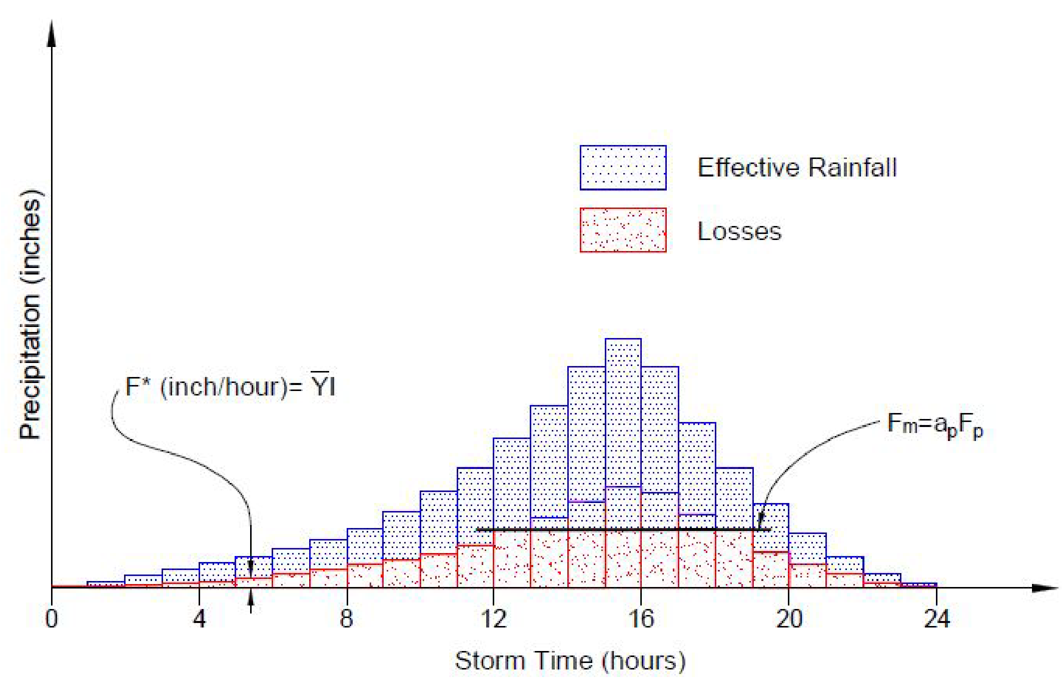


Figure 3-4. Design Storm Loss Function

3.7. Desert Hydrology Loss Rates

For desert watersheds, a field investigation is recommended for hydrologic studies to determine the pertinent drainage area characteristics. The extent of the field study will depend on the size of the drainage area and the complexity of the drainage problem. Many features, such as channel-flow patterns, flow distribution, channel diversions, and the type and density of vegetative cover, may not be apparent from topographic data or aerial imagery. The field study may also indicate the need for low-level aerial imagery to solve complex drainage problems.

3.7.1. Hydrologic Soil Cover Complex

A determination of vegetative cover types, quality and condition, and HSG must be made for the watershed area. Accordingly, **Section 3.7.8** and **Section 3.7.9** present additional desert cover types and densities. The most commonly encountered hydrologic soil cover complexes in desert areas are shown in **Figure 3-5**, along with their associated CN. Vegetation type, vegetative cover density, and HSG influence CNs, which in turn affect direct runoff.

3.7.2. Hydrologic Cover Types

Vegetation types are divided into the following groups:

Desert Brush: Includes such plants as mesquite, creosote bush, black bush, catclaw, cactus, etc. Typical of lower elevations and low annual rainfall.

Herbaceous: Includes short desert grasses with some brush. Typical of intermediate elevations and higher annual rainfall than desert areas.

Mountain Brush: Includes mixtures of oak, aspen, mountain mahogany, manzanita, bitter brush, maple, etc. Typical of intermediate elevations and generally higher annual rainfall than herbaceous areas.

Juniper-Grass: Includes juniper areas mixed with varying grass cover that is generally heavier than desert grasses due to higher annual precipitation. Typical of higher elevations.

Ponderosa Pine: Includes ponderosa pine forests. Typical of high elevations and high annual precipitation.

3.7.3. Hydrologic Cover Density

Hydrologic cover density is defined as the percentage of the ground surface covered by the crown canopy of live plants and litter.

Three broad ranges of vegetative cover density have been established.

Poor: 0–20% vegetative cover

Fair: 20%–40% vegetative cover

Good: Over 40% vegetative cover

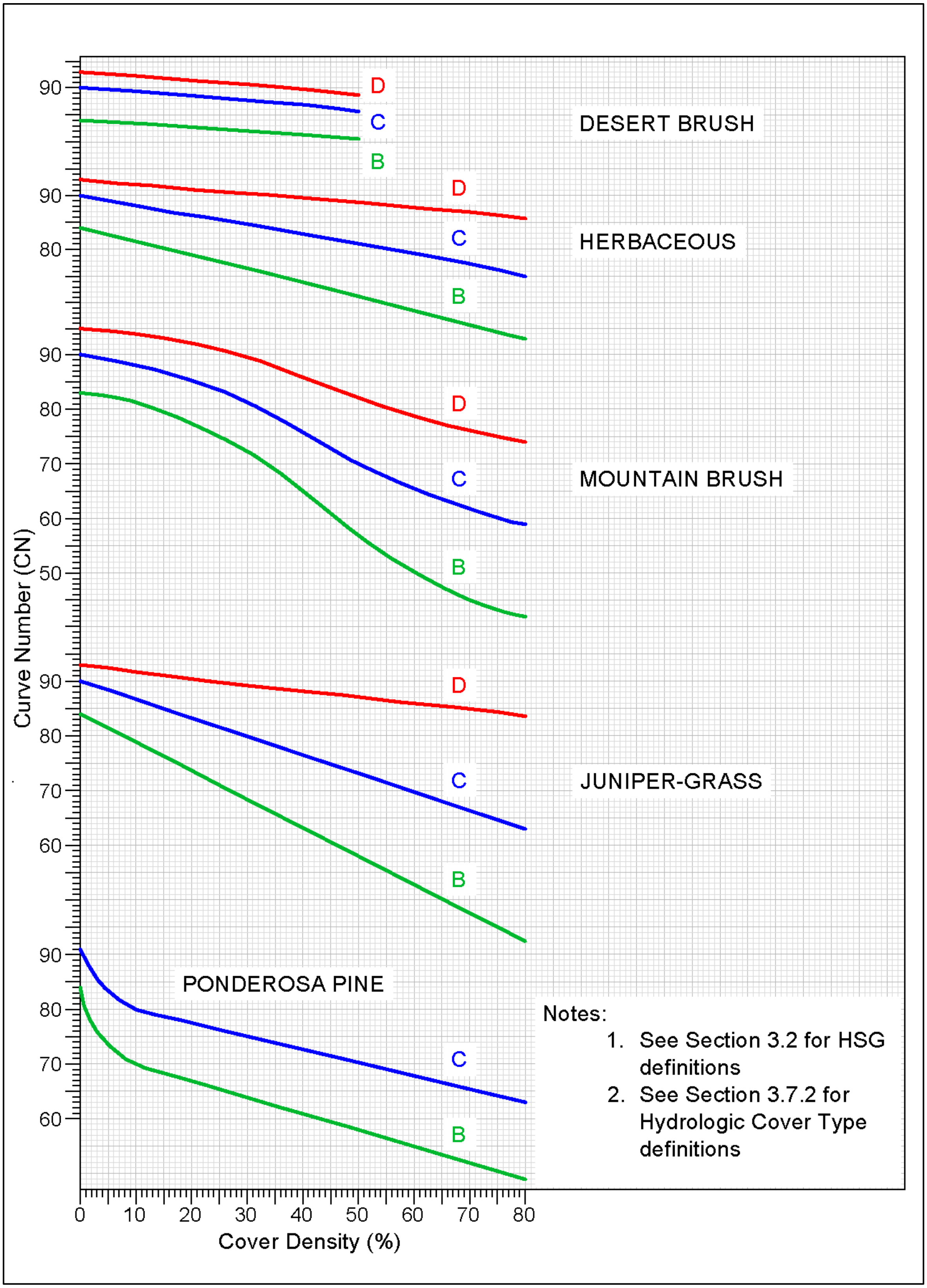


Figure 3-5. Associated Curve Number based on Cover Type Density and HSG

3.8. Post-Fire Hydrology

Wildfires can have a significant but temporary impact on the physical processes that influence a subarea’s CN. When vegetation burns, it changes the area's hydrologic condition, and the resulting hydrophobic soils can affect how water infiltrates compared to the original HSG. After a wildfire, watersheds go through a recovery process that can take anywhere from a few years to several decades. The most noticeable changes in hydrologic function typically occur in the first few years following the fire, gradually decreasing over time.

Wildfires affect the CN of burned areas; for further details on adjusting the CN after a wildfire and understanding post-fire hydrology, refer to **Chapter 6**.

CHAPTER 4
RATIONAL METHOD

4.1. Standard Rational Method Equation

The Rational Method was originally developed to estimate peak discharges from small urban and developed areas (less than 1 sq. mi. or 640 acres); therefore, its use should normally be limited to those conditions. The standard Rational Method equation relates rainfall intensity, a runoff coefficient, and watershed area to the peak runoff at a selected concentration point as shown in **Equation 4-1**:

Equation 4-1 Standard Rational Method Equation

$$Q = CIA$$

where:

Q = The peak discharge (cfs)
C = A runoff coefficient representing the ratio of runoff depth to rainfall depth (dimensionless)
I = The time-averaged rainfall intensity for a storm duration equal to the time of concentration (inches/hour)
A = Watershed area (acres)

Essentially, the standard Rational Method equation is a unit conversion yielding 1.008 cubic feet per second (cfs) when 1 inch per hour of rainfall is applied uniformly over a 1-acre watershed; the 0.8% difference introduced by unit conversions is typically ignored.

The values of the runoff coefficient (C) and the rainfall intensity (I) are based on a study of the watershed characteristics such as the type and condition of the runoff surfaces and the time of concentration. Watershed area (A) may be determined using various methods (e.g., GIS) and a suitable topographic map of the study area. These factors and the limitations of the Rational Method equation are discussed in the following sections.

Computing peak discharge using the Rational Method requires the following data:

Runoff Coefficient (C): Represents the ratio of runoff depth to rainfall depth.

Rainfall Intensity (I): The time-averaged rainfall intensity for a specified storm duration and selected design frequency.

Watershed Area (A) Characteristics: For example, area size (A), shape, slope, and surface roughness.

4.1.1. Rational Method Limitations

The Rational Method equation is valid if certain assumptions are reasonably correct and limitations of the method are observed. Two basic assumptions are:

1. The frequency of storm runoff is the same as the frequency of the rainfall producing this runoff (e.g., 100-year storm event rainfall will produce 100-year runoff).
2. The peak runoff occurs when all parts of the watershed contribute to the runoff.

The Rational Method equation inherently assumes that rainfall intensity (I) is uniformly distributed over the watershed at a uniform rate throughout the duration of the storm event. This condition is generally assumed to be applicable for areas less than 640 acres. Beyond this limit (i.e., watersheds greater than 640 acres), rainfall depths may vary, thus introducing additional uncertainty into calculations, potentially making the Rational Method inappropriate.

The selection of the runoff coefficient (C) is another major limitation for the Rational Method equation. For small urban and developed areas, the runoff coefficient can be reasonably estimated from field and aerial photo studies. For larger areas, where the determination of the runoff coefficient is to be based on vegetation type, cover density, infiltration capacity of the ground surface, and slope of the drainage area, estimating the runoff coefficient may involve greater error due to the variability of the watershed characteristics. Rainfall losses due to evaporation, transpiration, depression, and channel storage are inadequately evaluated, and may significantly impact the estimate of the watershed's peak rate of runoff, particularly in natural cover and desert areas.

The intensity-duration curve used for Rational Method studies also does not account for the effects of depth-area-duration factors. For large watershed areas, the absence of depth-area adjustments can result in significant differences in the estimate of the precipitation depths.

The above limitations indicate that an estimate of the peak flow rate becomes less reliable as the watershed area increases; therefore, the Rational Method equation should generally not be used for watershed areas greater than 640 acres.

4.1.2. Time of Concentration

The critical duration of the storm rainfall required in the Rational Method equation is based on the time of concentration of the drainage area.

The time of concentration (T_c) is defined as the interval of time required for the flow at a concentration point (a given point) to become a maximum (peak flow) under a uniform rainfall intensity over the subject watershed. Because the Rational Method assumes that the entire watershed contributes flow to the concentration point, T_c is typically defined as the time from the beginning of rainfall for water to travel from the most hydraulically remote portion of the watershed to reach the concentration point. The time of concentration (T_c) is a function of many variables, including the length of the flow path from the most remote point of an area to the concentration point, the slope and other characteristics of natural and improved channels in the area, the loss rate characteristics of the soil, and the extent and type of land development.

For Rational Method studies based on this Manual, the T_c of an initial subarea may be estimated from the nomograph in **Figure 4-1**. The initial subarea T_c estimation is often the most significant component of the watershed T_c computation. Small development studies typically use only initial subarea estimations due to the small subarea sizes. Larger study areas generally show high sensitivity to the initial subarea T_c. Consequently, judgment is needed when developing initial subarea T_c estimates. The **Figure 4-1** nomograph is based on the Kirpich formula and relates an initial subarea T_c to subarea slope and development type. It is assumed in the nomograph that overland flow effects dominate the travel time hydraulics.

As shown in **Equation 4-2**, the time of concentration for the next downstream subarea is computed by adding the initial time of concentration (T_i) to the time required for the computed peak flow to travel to the next concentration point (travel time, T_t). Time of concentration is computed for each subsequent subarea by computing the peak flow rate travel time between subareas and adding to the cumulative total.

Equation 4-2 Time of Concentration

$$T_c = T_i + T_t$$

where:

T_c = Time of concentration (minutes or hours)
T_i = Initial time (minutes or hours); typically characterized as sheet flow near the watershed headwaters and estimated from **Figure 4-1**
T_t = Travel time (minutes or hours); often occurring as concentrated flow (e.g., within a street gutter, swale, channel, or pipe); estimated from **Equation 4-4**

When the flow concentrates in street gutters, drainage channels, or closed conduits, the flow velocity should be estimated by Manning's equation (**Equation 4-3**):

Equation 4-3 Manning's Equation

$$V = \frac{1.49}{n} R^{2/3} S^{1/2}$$

where:

V = Mean velocity (feet per second (fps))
n = Manning's roughness coefficient
R = Hydraulic radius (feet)
S = Energy slope, which equals the conduit invert slope for uniform flow

When Manning's equation is not appropriate, due to irregular geometry or insufficient channel detail, as is often the case in natural mountain and valley channels, the following empirical equations are used to estimate the flow velocity (V).

Equation 4-4 should be used when analyzing flow in steep, rough mountain streams:

Equation 4-4 Natural Mountain Channel Equation

$$V = 5.6Q^{1/3}S^{1/2}$$

where:

V = Mean velocity (fps)
Q = Peak discharge (cfs)
S = Channel slope (feet/feet)

Use **Equation 4-5** when analyzing flow in wide, less steep, valley streams:

Equation 4-5 Natural Valley Channel Equation

$$V = (7.0+8.0Q^{0.352})S^{1/2}$$

where:

V = Mean velocity (fps)
Q = Peak discharge (cfs)
S = Channel slope (feet/feet)

The travel time will then be the flow distance divided by the mean velocity, as shown in **Equation 4-6**.

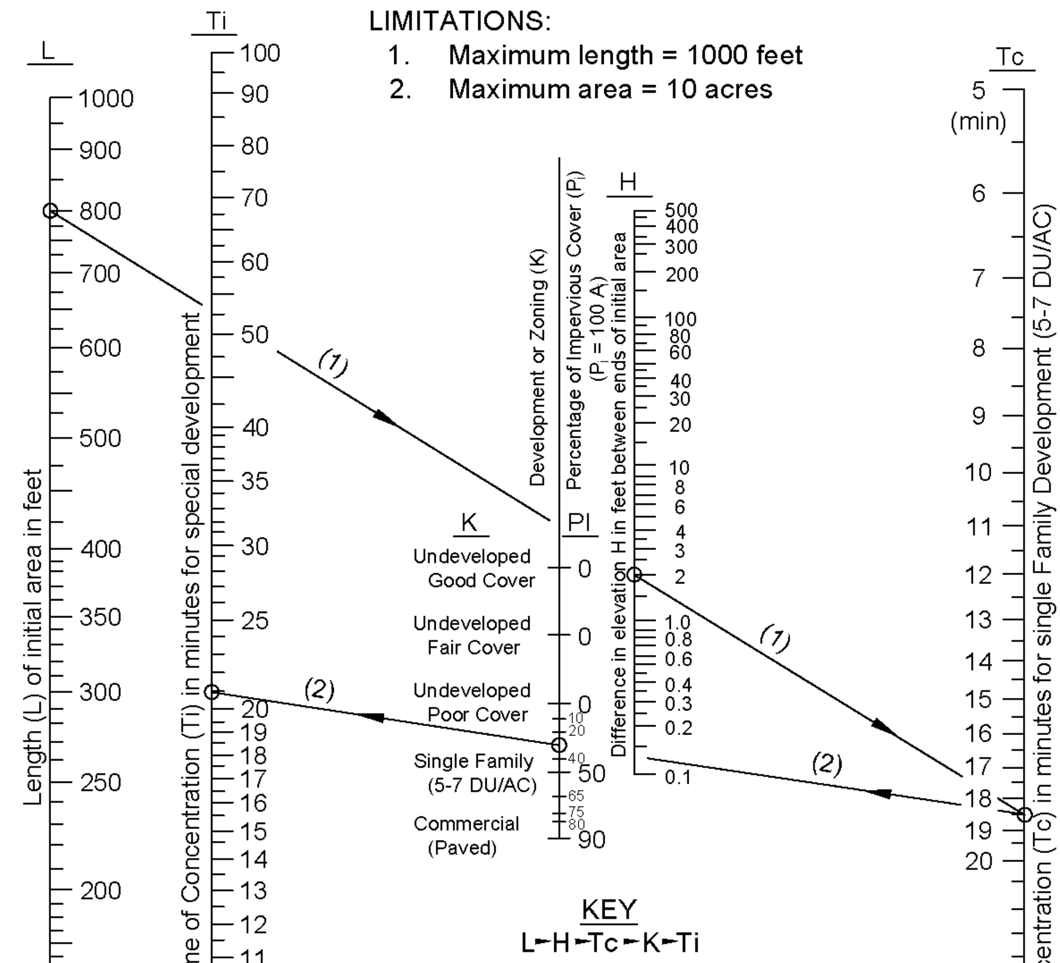
Equation 4-6 Travel Time Equation

$$T_t = L/V$$

where:

T_t = Travel time (minutes or hours)
V = Mean velocity (fps)
L = Flow path length (feet)

Proceeding downstream through a watershed, the time of concentration (T_c) will typically increase due to the addition of the travel time (T_t) values. In open channel flow (e.g., street conveyance), additional runoff is often introduced between concentration points (i.e., upstream and downstream nodes), which can increase the peak flow rate. This added flow affects the velocity, consequently influencing the Rational Method flow calculation at the next downstream concentration point. Therefore, iterative calculations are required. These iterations involve estimating the average flow between concentration points and then back-checking the estimate for adequacy; an example of this is shown in **Section 4.4**.



PI	Development
85	Commercial
72	Industrial
65	1/8-acre lot (or less)
38	1/4-acre lot
30	1/3-acre lot
25	1/2-acre lot
20	1-acre lot
12	2-acre lot

Example:

(1) L = 800', H = 2.0', K = Single Family Res. (5-7 DU/AC), Tc = 18.5 min.

(2) L = 800', H = 2.0', K = Single Family Res. ($\frac{1}{3}$-acre), Ti = 21.0 min.

Figure 4-1. Initial Subarea Time of Concentration Nomograph

4.1.3. Rainfall Intensity

Rainfall intensity (I) is determined using intensity-duration curves which are appropriate for the study watershed.

The intensity-duration curves for a particular study watershed can be developed using the log-log paper of **Figure 2-3** (see **Chapter 2**), plotting the 1-hour point rainfall value for the desired return period, and then drawing a straight line through the point with a slope of 0.5, extending to the 5-minute minimum duration (see **Section 2.8**).

Alternatively, NOAA rainfall data can be applied in combination with **Figure 2-3** to develop approximate intensities for various storm durations. Rainfall records should be carefully analyzed to determine the most appropriate slope for the intensity-duration plot. All supporting data and analytical methods must be submitted to the County for review and approval.

4.1.4. Runoff Coefficient

The runoff coefficient (C) represents the ratio of the runoff rate to the rainfall rate at an average rainfall intensity (I) when the total watershed contributes flow. The runoff coefficient value is dependent on rainfall intensity, drainage area slope, type and density of vegetative cover, infiltration capacity of the ground surface, and various other factors.

The runoff coefficient is assumed to be a function of the impervious and pervious watershed area fractions, pervious area infiltration rate, and the effects of watershed detention. These parameters are numerically related to the runoff coefficient (C) using **Equation 4-7**:

Equation 4-7 Runoff Coefficient Equations

$$C = \begin{cases} 0.90\left(a_i + \frac{(I-F_p)a_p}{I}\right), \text{ for I greater than } F_p \\ 0.90\, a_i, \text{ for I less than or equal to } F_p \end{cases}$$

where:

0.90 is a calibration constant determined by an average fit between the Rational Method and design storm unit hydrograph (see **Chapter 5**) peak flow rate estimates, and where:

C	=	Runoff coefficient
I	=	Rainfall intensity (inches/hour)
F_p	=	Infiltration rate for pervious areas (inches/hour) (see **Section 3.6.4**)
a_i	=	Ratio of impervious area to total area (decimal fraction)
a_p	=	Ratio of pervious area to total area (decimal fraction), ($a_p = 1 - a_i$)

4.1.5. Watershed Area

The contributing watershed area to a concentration point may be determined by using software (e.g., GIS or AutoCAD tools) or manual methods from digital elevation models (DEMs), topographic contour maps, aerial photos, and field surveys. When using these tools to draw watershed divides—or at a smaller scale, sub-watershed and subareas (i.e., smaller areas within a watershed generating a portion of the watershed outlet flow)—care should be taken to ensure that the entire area contributing flow to a designated concentration point is captured. When delineating watersheds and subareas, other considerations should include:

- **Initial Flow Path Length:** Per **Figure 4-1**, the maximum allowable flow path length for the initial subarea is 1,000 feet.
- **Travel Time Flow Paths:** Since Rational Method studies are typically dependent on the node-to-node average slope, flow paths should be defined with respect to inflection points to avoid introducing uncertainty into the calculations.
- **Subarea Delineation:** Subarea boundaries should be drawn to minimize the heterogeneity of characteristics important to Rational Method calculations. These include, but are not limited to: land use, vegetation type and density, hydrologic soil types, general pervious and impervious area percentages and connectivity to the main flow path, and slope (area and main flow path).
- **Subarea Size:** Per **Figure 4-1**, the initial subarea size should not exceed 10 acres. The Rational Method is highly sensitive to the time of concentration (T_c); therefore, large differences in successive subarea size, shape, and associated flow path lengths (especially in the most upstream reaches of the watershed) may introduce error into the flow calculations. Subarea sizes should be limited to allow for a gradual increase as the analysis progresses downstream. Concentration points should be selected downstream of the initial subarea such that subarea travel times are less than 3 minutes for T_c values of 30 minutes (or shorter), and less than 5 minutes for T_c values between 30 minutes and 60 minutes. After a T_c of 1 hour (or longer), subarea travel times should be limited to less than 10 minutes.
- **Cross-Watershed Spillover:** In locations where mild lateral and transverse slopes constitute the watershed or subarea divide (e.g., a low relief natural divide or roadway crown), runoff from within the topographic watershed or subarea may spill into an adjacent watershed. The watershed/subarea boundary should be adjusted as necessary to account for this condition.

4.1.6. Peak Flow Rate Equation

Combining **Equation 4-1** and **Equation 4-7**, the peak flow rate estimate is shown in **Equation 4-8**:

Equation 4-8 Peak Flow Rate Equation

$$Q = 0.90(I - F_m)A$$

where:

F_m = a_pF_p (see **Section 3.6.5**), and where it is understood that rainfall intensity (I) is greater than F_p (otherwise if $I \le F_p$, $Q = 0.90a_iIA$)

4.2. Confluence Analysis

In most studies, the calculation of peak flow rates along a main channel or stream involves only the direct application of **Equation 4-8**; typically these studies involve the inclusion of subarea runoff to the stream where the effect on the stream peak flow rate is relatively minor.

However, at the confluence of two or more streams, estimating the peak flow requires a more detailed analysis that accounts for the interaction of tributary inflows. **Equation 4-9** presents the general case of a two-stream confluence; this equation set adjusts flows from streams with longer time of concentration (T_c) values by applying a combined factor: the ratio of time of concentration (e.g., T_x/T_{x+1}) and the corresponding ratio of rainfall intensity (e.g., I_x/I_{x+1}), further modified by the area-averaged maximum loss rate (F_m). Flows from streams with shorter time of concentration (T_c) values are adjusted using only the rainfall intensity ratio (e.g., $T_{x+1}/T_x = 1$).

Equation 4-9 General Confluence Equation Set

$$Q_{p(x)} = Q_x + \frac{T_x}{T_{x+1}} \frac{\left(I_x - F_{m(x+1)}\right)}{\left(I_{x+1} - F_{m(x+1)}\right)} Q_{x+1}$$

$$Q_{p(x+1)} = Q_{x+1} + \frac{T_{x+1}}{T_x} \frac{\left(I_{x+1} - F_{m(x)}\right)}{\left(I_x - F_{m(x)}\right)} Q_x$$

where:

Stream x

T_x = Time of concentration (minutes)

Q_x = Peak flow rate (cfs)

I_x = Intensity (inches/hour)

$F_{m(x)}$ = Area-averaged maximum loss rate (inches/hour)

Stream x+1

T_{x+1} = Time of concentration (minutes)

Q_{x+1} = Peak flow rate (cfs)

I_{x+1} = Intensity (inches/hour)

$F_{m(x+1)}$ = Area-averaged maximum loss rate (inches/hour)

Confluenced Stream

$Q_{p(x)}$ or $Q_{p(x+1)}$ = Confluenced peak flow rates (cfs)

Note: The T_c ratio is limited to a maximum value of 1 (e.g., $\frac{T_x}{T_{x+1}} \leq 1$)

The confluenced peak flow rates are calculated for each stream (e.g., $Q_{p(x)}$, $Q_{p(x+1)}$, etc.) using **Equation 4-9**. The largest Q_p and its associated time of concentration (T_c) are then designated as the governing values and are used to continue the hydrologic calculations downstream (if needed).

The total catchment area (A_t) is the sum of all individual catchment areas (e.g., $A_t = A_x + A_{x+1}$). However, to better represent the influence of each tributary based on its timing and hydrologic response, an effective contributing area (A_{eff}) should be calculated as a function of the time of concentration ratio using **Equation 4-10**. The effective area associated with the peak flow rate from the governing stream is used for downstream hydrologic calculations (if needed).

Equation 4-10 General Effective Area Equation Set

$$A_{eff(x)} = A_x + \left(\frac{T_x}{T_{x+1}}\right) A_{x+1}$$

$$A_{eff(x+1)} = A_{x+1} + \left(\frac{T_{x+1}}{T_x}\right) A_x$$

where:

Stream x

T_x = Time of concentration (minutes)

A_x = Tributary watershed area (acres)

Stream x+1

T_{x+1} = Time of concentration (minutes)

A_{x+1} = Tributary watershed area (acres)

Confluenced Stream

$A_{eff(x)}$ or $A_{eff(x+1)}$ = Effective area (acres)

Note: The T_c ratio is limited to a maximum value of 1 (e.g., $\frac{T_x}{T_{x+1}} \leq 1$)

In cases where more than two streams confluence (e.g., Q_{x+2}, Q_{x+3}, etc.), additional equations and terms are added, as necessary, to **Equation 4-9** and **Equation 4-10**. **Section 4.4** provides an example of a three-stream confluence calculation.

If the time of concentration values are equal (e.g., $T_x = T_{x+1}$), the tributary peak flows and contributing watershed areas can be summed directly (e.g., $Q_p = Q_x + Q_{x+1}$) before continuing with downstream calculations (if needed).

4.3. Instructions for Rational Method Hydrology Calculations

The following steps are general guidelines for preparing Rational Method calculations.

1. Delineate the study watershed (limited to approximately 640 acres), drainage system (e.g., Flow Path), and designate subareas tributary to concentration points (see **Section 4.1.5**).
2. Determine the initial time of concentration (T_i) using **Figure 4-1**. The initial subarea should generally be the most upstream portion of the watershed drainage system, with an area less than 10 acres and a flow path of less than 1,000 feet.
3. Using the T_i value from Step 2 for the initial subarea (or new T_c calculated from Step 9 for any subsequent subareas), determine the rainfall intensity (I) from the developed intensity-duration curve using **Figure 2-3** (see **Chapter 2**).
4. Determine the subarea size (A) tributary to the point of concentration. Care should be taken to minimize heterogeneity of characteristics such as land use, vegetation type and density, hydrologic soil types, general pervious and impervious area percentages, connectivity to the main flow path, and slope (subarea and main flow path).
5. Calculate the area-averaged maximum loss rate (F_m) of the subarea per **Equation 3-7**.

6. Compute the peak flow rate (Q) for the point of concentration using **Equation 4-8**. If the calculated Q is less than the previous upstream point of concentration Q, use the upstream Q value.
7. Measure the flow path length (L) that the peak runoff must travel to the concentration point of the next downstream subarea. Determine the mean velocity (V) of the flow in this reach using **Equation 4-3**. If open channel flow occurs, iterative calculations may be required to estimate the average flow through the subarea.
8. Use the velocity (V) from Step 7 to calculate the travel time (T_t) with **Equation 4-6**.
9. Calculate the T_c by either:
   - Adding T_i from Step 2 and T_t from Step 8 using **Equation 4-2** for the subarea downstream of the initial subarea; or
   - Adding the calculated T_t from Step 8 to the previously calculated T_c for the subsequent downstream subareas.
10. Using the T_c from Step 9, calculate flow (Q) for the new point of concentration following Steps 3 through Step 6.
11. The flow calculations will progress downstream until the flow path reaches the watershed outlet or a junction of independent streams (confluence point). If flow reaches a confluence point, the Rational Method procedures should be performed starting with Step 1 at the upstream end of the new stream.
12. After all the independent stream results are calculated at the junction, use **Equation 4-9** and **Equation 4-10** to estimate the confluenced Q, associated T_c, and total confluenced tributary area. The calculations can then continue downstream, if needed.

The blank form in **Figure 4-2** can be used to summarize the input data and calculate flow values at concentration points within a study watershed.

SAN BERNARDINO COUNTY
HYDROLOGY MANUAL

STUDY NAME:

______-YEAR STORM 1-HOUR RAINFALL (INCH) = ______

ANTECEDENT MOISTURE CONDITION (AMC) = ______

Calculated by ____________ Date ________
Checked by ____________ Date ________
Page _____ of _____

Concentration Point	Area (Acres)		Soil Type	Dev. Type	$T_{i\ or\ t}$ min.	T_c min.	I in./hr.	F_m in./hr.	F_m avg.	Q Total	L feet	Slope ft./ft.	V_{ave} ft./sec.	Notes
	Subarea	Total												

Figure 4-2. Rational Method Blank Worksheet

4.4. Rational Method Example

Determine the 100-year peak flow rate at various points of concentration within the example watershed (see **Figure 4-3** and **Table 4-1**); assume the 1-hour point rainfall is 1.49 inches.

1. **Delineate the study watershed (limited to approximately 640 acres), drainage system (e.g., Flow Path), and designate subareas tributary to concentration points (see Section 4.1.5).**

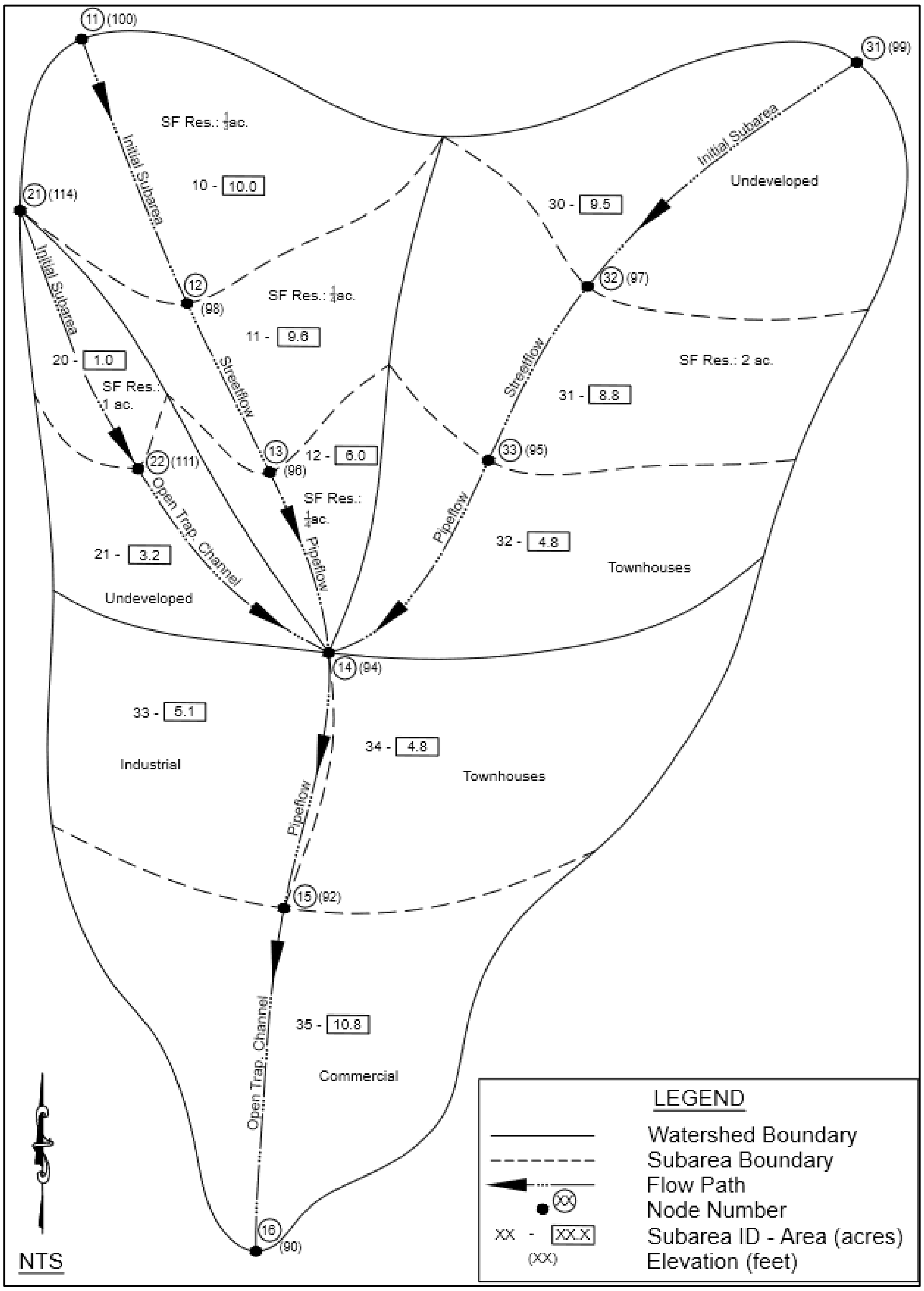


Figure 4-3. Rational Method Example Study Area

Table 4-1. Summary Table of Rational Method Example Study Area

Subarea[1]	Area A (acres)	Development Type[2] K	Flow Path Length L (feet)	Elevation Difference[3] H (feet)	Slope[4] S (feet/feet)	Flow Conveyance Type
10 (11 → 12)	10.0	Residential SF $\frac{1}{3}$ - acre	800	2 (100-98)	0.0025	Initial Subarea
11 (12 → 13)	9.6	Residential SF $\frac{1}{4}$ - acre	350	2 (98-96)	0.0057	Open Channel Flow (Street)
12 (13 → 14)	6.0	Residential SF $\frac{1}{4}$ - acre	650	2 (96-94)	0.0031	Pipe Flow (Closed Conduit)
20 (21 → 22)	1.0	Residential SF 1 - acre	400	3 (114-111)	0.0075	Initial Subarea
21 (22 → 14)	3.2	Undeveloped (Grass-Fair)	850	17 (111-94)	0.020	Open Channel Flow (Trapezoid)
30 (31 → 32)	9.5	Undeveloped (Grass-Good)	750	2 (99-97)	0.0027	Initial Subarea
31 (32 → 33)	8.8	Residential SF 2 - acre	550	2 (97-95)	0.0036	Open Channel Flow (Street)
32 (33 → 14)	4.8	Townhomes	700	1 (95-94)	0.0014	Pipe Flow (Closed Conduit)
33 (Ind.)/34 (TH) (14 → 15)	5.1	Industrial	550	2 (94-92)	0.0036	Pipe Flow (Closed Conduit)
	4.8	Townhomes				
35 (15 → 16)	10.8	Commercial	700	2 (92-90)	0.0029	Open Channel Flow (Trapezoid)

1. The upstream and downstream nodes are in parentheses (e.g., upstream node → downstream node).
2. All development types were assigned type "B" Hydrologic Soil Group (HSG).
3. The elevation difference was calculated by subtracting the downstream node elevation from the upstream node elevation; the node elevations are in parentheses (e.g., upstream node elevation - downstream node elevation).
4. The slope was calculated by dividing the elevation difference, H, by the flow path length, L, for the respective subarea.

2. **Determine the initial time of concentration (T_i) using Figure 4-1. The initial subarea should generally be the most upstream portion of the watershed drainage system, with an area less than 10 acres and a flow path of less than 1,000 feet.**

   This example contains three initial subareas; however, this step focuses on the initial Subarea 10 between Node 11 and Node 12 (note that the other initial subarea calculations would be similar).

   With the flow path length (L), elevation difference (H), and development type (K) defined in **Table 4-1**, **Figure 4-1** is used to determine the initial time of concentration. From **Figure 4-1**, a straight line (Path 1) is drawn from the L data point (800 feet), through the H data point (2 feet), resulting in a T_c of approximately 18.5 minutes. Then, another straight line (Path 2), is drawn from the T_c data point through the K data point (single family residential, 1/3-acre) with the associated impervious cover of 30% assumed from **Table 3-3**. The resulting initial time of concentration is estimated to be 21.0 minutes.

3. **Using the T_i value from Step 2 for the initial subarea, (or new T_c calculated from Step 9 for any subsequent subareas), determine the rainfall intensity (I) from the developed intensity-duration curve using Figure 2-3 (see Chapter 2).**

To develop an intensity-duration curve (IDC), the 1-hour point rainfall (inches) for the respective design storm frequency must be determined and the 0.5 log-log slope applied. In this example, the 1-hour point rainfall (100-year storm) is 1.49 inches (i.e., 1.49 inches/hour).

The developed IDC (see **Figure 4-4**) is then used to determine the rainfall intensity from the 21.0-minute time of concentration (i.e., storm duration) from Step 2.

$$I_{(11\rightarrow 12)} = 2.52\ \frac{\text{inches}}{\text{hour}}$$

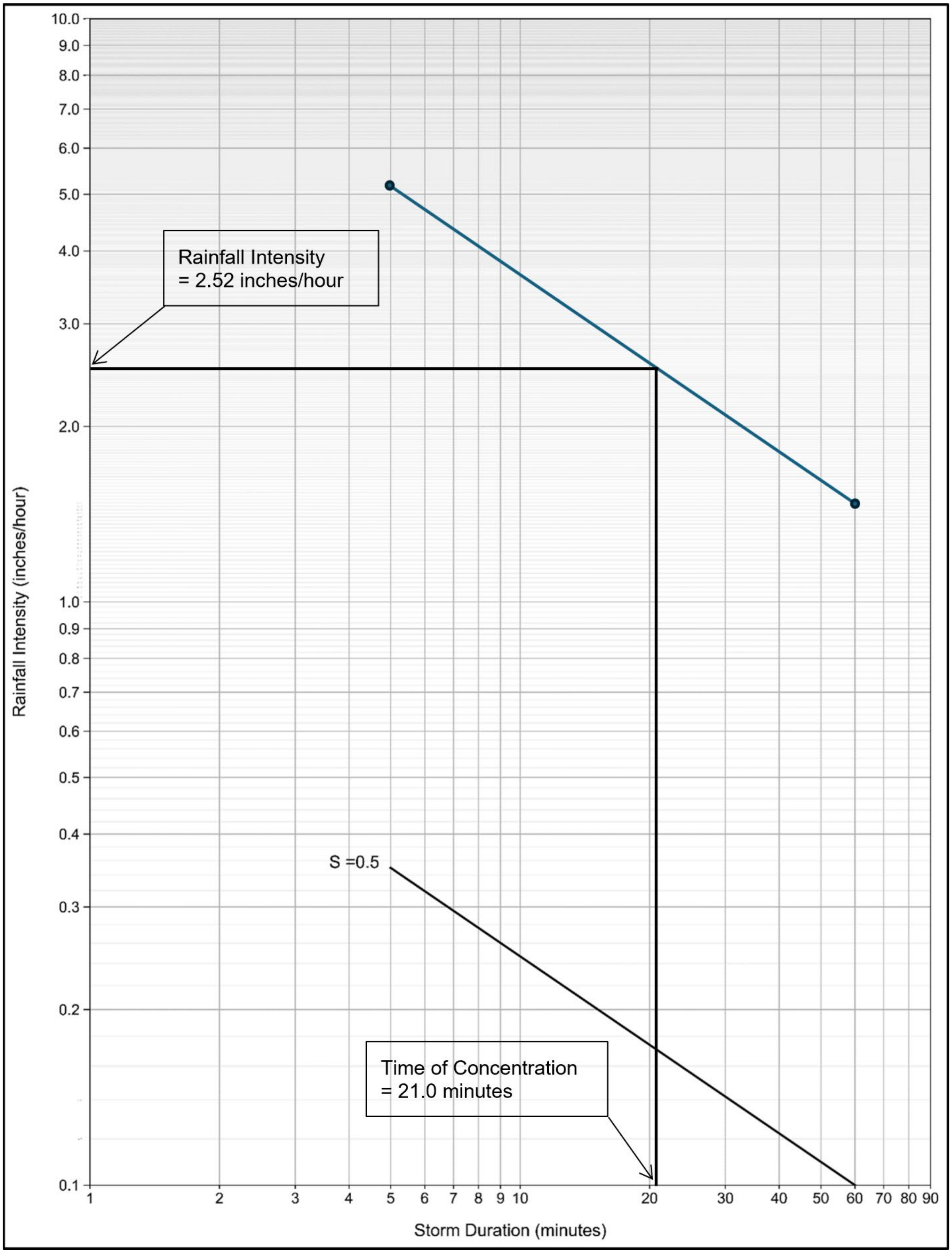


Figure 4-4. Intensity-Duration Curve for Rational Method Example

4. **Determine the subarea size (A) tributary to the point of concentration. Care should be taken to minimize heterogeneity of characteristics such as land use, vegetation type and density, hydrologic soil types, general pervious and impervious area percentages, connectivity to the main flow path, and slope (subarea and main flow path).**

   The subarea size tributary to the point of concentration (Node 12) is 10 acres. Other values are as follows (see **Table 4-1**):

Cover type:	SF Residential - 1/3-acre
Hydrologic soil group (HSG):	B
Quality of cover:	N/A
Curve number (CN):	72 (AMC II)

5. **Calculate the area-averaged maximum loss rate (F_m) of the subarea per Equation 3-7.**

   From **Table 3-3**, the impervious area decimal fraction (a_i) is estimated; the pervious area decimal fraction (a_p) can then be determined:

Impervious area fraction (a_i):	0.30
Pervious area fraction (ap):	0.70

   Using **Figure 3-3**, the infiltration rate for pervious areas (F_p) based on the parameters outlined above is determined:

   Infiltration Rate for the Pervious Areas (F_p): 0.52 inches/hour

   Because this is an initial subarea, the area-averaged maximum loss rate is the direct result from **Equation 3-7** (i.e., $F_m = F_{m\ avg}$).

$$F_m = F_p a_p = 0.52 \frac{\text{inches}}{\text{hour}} \text{ x } 0.70 \approx 0.36 \frac{\text{inches}}{\text{hour}}$$

$$F_{m\ (11 \rightarrow 12)} = 0.36 \frac{\text{inches}}{\text{hour}}$$

6. **Compute the peak flow (Q) for the point of concentration using Equation 4-8. If the calculated Q is less than the previous upstream point of concentration Q, use the upstream Q value (excluding the initial subarea).**

   The peak flow rate is calculated for the subarea as follows:

$$Q = 0.90\ (I\text{-}F_m)A = 0.90 \text{ x } (2.52 \frac{\text{inches}}{\text{hour}} - 0.36 \frac{\text{inches}}{\text{hour}}) \text{ x } 10 \text{ acres} \approx 19.4 \text{ cfs}$$

$$Q_{(11 \rightarrow 12)} = 19.4 \text{ cfs}$$

7. **Measure the flow path length (L) that the peak runoff must travel to the concentration point of the next downstream subarea. Determine the mean velocity (V) of flow in this reach using Equation 4-3. If open channel flow occurs, iterative calculations may be required to estimate the average flow through the subarea.**

   This step determines the travel time for the flow routed from Node 12 to Node 13 (Subarea 11) as street flow. Node 13 and Node 12 are the downstream and upstream concentration points, respectively. Because Subarea 11 is not a closed conduit, an average flow is used to estimate the mean velocity. This approach accounts for additional flow assumed to be added during the flow routing process. This requires iterative calculations to converge on a solution (shown in Steps 7 through 10):

   A unit flow rate is assumed: A general guideline is to assume that the unit flow rate is the upstream flow divided by the upstream area.

$$q_{assumed} = \frac{Q_{(11\rightarrow12)}}{A_{(11\rightarrow12)}} = \frac{19.4\text{ cfs}}{10.0\text{ acres}} \approx 2.0\ \frac{\text{cfs}}{\text{acre}}$$

   It is assumed that half of the subarea contributes flow to the open channel:

$$Q_{avg} = Q_{(11\rightarrow12)} + \left(q_{assumed}\frac{A_{(12\rightarrow13)}}{2}\right) = 19.4\text{ cfs} + \left(2.0\frac{\text{cfs}}{\text{acre}} \text{x} \frac{9.6\text{ acres}}{2}\right) \approx 29.0\text{ cfs}$$

   The following channel characteristics are assumed:

Channel type:	Open channel street flow
Channel shape:	Rectangular
Bottom width (B):	20 feet
Side slope (Z):	0
Channel slope (S):	0.0057 feet/feet
Manning's n (n):	0.018
Length (L):	350 feet

   To calculate the mean velocity using **Equation 4-3** (Manning's Equation), the following relationships are used:

   Continuity: $Q = AV$

   (where: Q is the flow rate, A is the flow area, and V is the mean velocity)

   Flow area (A): $A = BD_n$

   (where: B is channel bottom width and D_n is the normal flow depth)

   Hydraulic Radius (R): $R = \frac{A}{P}$

   (where: P is the wetted perimeter based on D_n)

   Since both the flow area (A) and the wetted perimeter (P) are dependent upon the flow depth, an iterative process is used to estimate the normal depth (D_n) that satisfies the given flow conditions using **Equation 4-3**:

$$V = \frac{1.49}{n} R^{2/3} S^{1/2}$$

Substitute A/P for R (hydraulic radius):

$$\frac{Q}{A} = \frac{1.49}{n} \left(\frac{A}{P}\right)^{2/3} S^{1/2}$$

Substitute BD_n for A (flow area) and $B+2D_n$ for P (wetted perimeter):

$$\frac{Q}{BD_n} = \frac{1.49}{n} \left(\frac{BD_n}{B+2D_n}\right)^{2/3} S^{1/2}$$

$$\frac{29 \text{ cfs}}{20 \text{ feet x } D_n} = \frac{1.49}{0.018} \text{x} \left(\frac{20 \text{ feet x } D_n}{20 \text{ feet} + 2D_n}\right)^{2/3} (0.0057 \text{ feet/feet})^{1/2}$$

$$D_n \approx 0.42 \text{ feet}$$

The corresponding flow area (A) and wetted perimeter (P) values are then calculated to compute the hydraulic radius (R):

$$R = \frac{A}{P} = \frac{BD_n}{B+2D_n} = \frac{20 \text{ feet x } 0.42 \text{ feet}}{(20 \text{ feet} + (2 \text{ x } 0.42 \text{ feet}))} \approx 0.4 \text{ feet}$$

The mean velocity is then calculated using **Equation 4-3**:

$$V = \frac{1.49}{n} R^{2/3} S^{1/2} = \frac{1.49}{0.018} (0.4 \text{ feet})^{2/3} (0.0057)^{1/2} \approx 3.4 \text{ fps}$$

8. Use the velocity (V) from Step 7 to calculate the travel time (T_t) with Equation 4-6.

$$T_t = L/V = \frac{350 \text{ feet}}{3.4 \ \frac{\text{feet}}{\text{second}}} \text{ x } \frac{1 \text{ minute}}{60 \text{ seconds}} \approx 1.7 \text{ minutes}$$

$$T_{t\,(12\rightarrow13)} = 1.7 \text{ minutes}$$

9. Calculate the T_c by either:

- Adding T_i from Step 2 and T_t from Step 8 using **Equation 4-2** for the subarea downstream of the initial subarea; or
- Adding the calculated T_t from Step 8 to the previously calculated T_c for the subsequent downstream subareas.

$$T_{c\,(12\rightarrow13)} = T_{i\,(11\rightarrow12)} + T_{t\,(12\rightarrow13)} = 21.0 \text{ minutes} + 1.7 \text{ minutes} = 22.7 \text{ minutes}$$

10. Using the T_c from Step 9, calculate flow (Q) for the new point of concentration following Steps 3 through Step 6.

Following Steps 3 through Step 6, the Subarea 11 parameters are used to calculate the peak flow rate at Node 13:

Rainfall intensity (I):	2.42 inches/hour
Curve number (CN):	75

Pervious area fraction (a_p):	0.62
Infiltration rate for the pervious area (F_p):	0.46 inches/hour
Maximum loss rate (F_m):	0.29 inches/hour
Area ($A_{(11\rightarrow13)}$):	19.6 acres

The area-averaged maximum loss rate ($F_{m\ avg}$) is determined:

$$F_{m\ avg} = \frac{\sum_{i=1}^{n}(F_{m\ i}A_i)}{\sum_{i=1}^{n}A_i)} = \frac{\left(0.36\frac{\text{inches}}{\text{hour}} \text{ x } 10 \text{ acres}\right) + \left(0.29\frac{\text{inches}}{\text{hour}} \text{ x } 9.6 \text{ acres}\right)}{19.6 \text{ acres}} \approx 0.33\frac{\text{inches}}{\text{hour}}$$

Equation 4-8 is used to determine the peak flow rate at Node 13 for the total area (Subareas 10 and 11) contributing flow:

$$Q_{iteration\ (12\rightarrow13)} = 0.90 \text{ x } \left(2.42\frac{\text{inches}}{\text{hour}} - 0.33\frac{\text{inches}}{\text{hour}}\right) \text{ x } 19.6 \text{ acres} \approx 36.9 \text{ cfs}$$

Using the following equation (or another acceptable procedure), the estimated average flow is back-checked to confirm it is adequate:

$$Q_{back\text{-}check} = Q_{Upstream} + \frac{(Q_{iteration} - Q_{Upstream})}{2} = 19.4 \text{ cfs} + \frac{(36.9 \text{ cfs} - 19.4 \text{ cfs})}{2} \approx 28.2 \text{ cfs}$$

$$28.2 \text{ cfs} \neq 29.0 \text{ cfs}$$

Because the assumed average flow is determined to be inadequate, the iteration process restarts at Step 7 and continues through Step 10 until the assumed average flow is acceptable.

Since the back-check flow is less than the estimated average flow, the assumed unit flow rate is reduced from 2.0 cfs/acre to 1.8 cfs/acre. The following values are then determined following Steps 7 through 10:

Assumed unit flow rate ($q_{assumed}$):	1.8 cfs/acre
Average peak flow (Q_{avg}):	28.0 cfs
Mean Velocity (V):	3.4 fps
Travel Time (T_i):	1.7 minutes
Time of Concentration (T_c):	22.7 minutes
Rainfall intensity (I):	2.42 inches/hour
Area-averaged maximum loss rate ($F_{m\ avg}$):	0.33 inches/hour
Contributing Area ($A_{(11\rightarrow13)}$):	19.6 acres
Calculated Peak Flow ($Q_{iteration}$):	36.9 cfs
Back-check Flow ($Q_{back\text{-}check}$):	28.2 cfs

The back-check flow (28.2 cfs) is now closer to the average peak flow (28.0 cfs). Therefore, the peak flow at Node 13 is 36.9 cfs and Steps 7 through 10 are determined to be complete.

In this example, the iterative process is sensitive to the initial estimate of the average peak flow and less sensitive to the remaining parameters determined in Steps 7 through 10. This is observed in many of the parameter values and the resultant peak flow rate not significantly changing. However, this procedure is outlined to show an approach to the iterative process for open channel flows.

11. **The flow calculations will progress downstream until the flow path reaches the watershed outlet or a junction of independent streams (confluence point). If flow reaches a confluence point, the Rational Method procedures should be performed starting with Step 1 at the upstream end of the new stream.**

    A junction of three independent streams occurs at Node 14, therefore, Steps 1 through 10 should be repeated to determine the peak flow rate (Q), rainfall intensity (I), time of concentration (T_c), area (A), and area-averaged maximum loss rate (F_m) of each independent stream before performing the confluence analysis. The independent stream results are shown in **Table 4-2**:

Table 4-2. Stream Summary at Node 14 Junction

Stream Number (U/S Node XX → D/S Node XX)	**Total Area** A_x (acres)	**Time of Concentration** T_x (minutes)	**Rainfall Intensity** I_x (inches/hour)	**Area-Averaged Maximum Loss Rate** $F_{m(x)}$ (inches/hour)	**Peak Flow Rate** Q_x (cfs)
Stream 1 (11→14)	25.6	24.4	2.34	0.32	46.5
Stream 2 (21→14)	4.2	17.0	2.80	0.54	8.5
Stream 3 (31→14)	23.1	49.3	1.64	0.51	23.5

12. **After all the independent stream results are calculated at the junction, use Equation 4-9 and Equation 4-10 to estimate the confluenced Q, associated T_c, and total confluenced tributary area. The calculations can then continue downstream, if needed.**

    Let Stream 1 = Q_1, Stream 2 = Q_2, and Stream 3 = Q_3 as defined in **Table 4-2**, along with the other associated parameters. The junction equation essentially computes the potential peak flow rate at the time of concentration for each of the independent streams.

<u>Stream 1:</u>

$$Q_{p(1)} = Q_1 + \left[\cancelto{1}{\frac{(T_1)}{(T_2)}}\frac{(I_1\text{-}Fm_2)}{(I_2\text{-}Fm_2)}Q_2\right] + \left[\frac{(T_1)}{(T_3)}\frac{(I_1\text{-}Fm_3)}{(I_3\text{-}Fm_3)}Q_3\right]$$

$$Q_{p(1)} = 46.5\text{ cfs} + \left[\frac{\left(2.34\frac{\text{inches}}{\text{hour}}\text{-}0.54\frac{\text{inches}}{\text{hour}}\right)}{\left(2.80\frac{\text{inches}}{\text{hour}}\text{-}0.54\frac{\text{inches}}{\text{hour}}\right)}8.5\text{ cfs}\right] +$$

$$\left[\frac{(24.4\text{ minutes})}{(49.3\text{ minutes})}\frac{\left(2.34\frac{\text{inches}}{\text{hour}}\text{-}0.51\frac{\text{inches}}{\text{hour}}\right)}{\left(1.64\frac{\text{inches}}{\text{hour}}\text{-}0.51\frac{\text{inches}}{\text{hour}}\right)}23.5\text{ cfs}\right] \approx 72.1\text{ cfs}$$

$$A_{eff(1)} = A_1 + \cancelto{1}{\left(\frac{T_1}{T_2}\right)} A_2 + \left(\frac{T_1}{T_3}\right) A_3 = 25.6 \text{ acres} + 4.2 \text{ acres} + \left(\frac{24.4 \text{ minutes}}{49.3 \text{ minutes}}\right) \text{x } 23.1 \text{ acres} \approx 41.2 \text{ acres}$$

$$Q_{p(1)} = 72.1 \text{ cfs}$$
$$A_{eff(1)} = 41.2 \text{ acres}$$
$$T_1 = 24.4 \text{ minutes}$$

<u>Stream 2:</u>

$$Q_{p(2)} = Q_2 + \left[\frac{(T_2)}{(T_1)}\frac{(I_2 - Fm_1)}{(I_1 - Fm_1)} Q_1\right] + \left[\frac{(T_2)}{(T_3)}\frac{(I_2 - Fm_3)}{(I_3 - Fm_3)} Q_3\right]$$

$$Q_{p(2)} = 8.5 \text{ cfs} + \left[\frac{(17.0 \text{ minutes})}{(24.4 \text{ minutes})}\frac{\left(2.80 \frac{\text{inches}}{\text{hour}} - 0.32 \frac{\text{inches}}{\text{hour}}\right)}{\left(2.34 \frac{\text{inches}}{\text{hour}} - 0.32 \frac{\text{inches}}{\text{hour}}\right)} 46.5 \text{ cfs}\right] +$$

$$\left[\frac{(17.0 \text{ minutes})}{(49.3 \text{ minutes})}\frac{\left(2.80 \frac{\text{inches}}{\text{hour}} - 0.51 \frac{\text{inches}}{\text{hour}}\right)}{\left(1.64 \frac{\text{inches}}{\text{hour}} - 0.51 \frac{\text{inches}}{\text{hour}}\right)} 23.5 \text{ cfs}\right] \approx 64.7 \text{ cfs}$$

$$A_{eff(2)} = A_2 + \left(\frac{T_2}{T_1}\right) A_1 + \left(\frac{T_2}{T_3}\right) A_3 = 4.2 \text{ acres} + \left(\frac{17.0 \text{ minutes}}{24.4 \text{ minutes}}\right) \text{ x } 25.6 \text{ acres} + \left(\frac{17.0 \text{ minutes}}{49.3 \text{ minutes}}\right) \text{x } 23.1 \text{ acres}$$

$$\approx 30.0 \text{ acres}$$

$$Q_{p(2)} = 64.7 \text{ cfs}$$
$$A_{eff(2)} = 30.0 \text{ acres}$$
$$T_2 = 17.0 \text{ minutes}$$

<u>Stream 3:</u>

$$Q_{p(3)} = Q_3 + \left[\cancelto{1}{\frac{(T_3)}{(T_1)}}\frac{(I_3 - Fm_1)}{(I_1 - Fm_1)} Q_1\right] + \left[\cancelto{1}{\frac{(T_3)}{(T_2)}}\frac{(I_3 - Fm_2)}{(I_2 - Fm_2)} Q_2\right]$$

$$Q_{p(3)} = 23.5 \text{ cfs} + \left[\frac{\left(1.64 \frac{\text{inches}}{\text{hour}} - 0.32 \frac{\text{inches}}{\text{hour}}\right)}{\left(2.34 \frac{\text{inches}}{\text{hour}} - 0.32 \frac{\text{inches}}{\text{hour}}\right)} 46.5 \text{ cfs}\right] +$$

$$\left[\frac{\left(1.64 \frac{\text{inches}}{\text{hour}} - 0.54 \frac{\text{inches}}{\text{hour}}\right)}{\left(2.80 \frac{\text{inches}}{\text{hour}} - 0.54 \frac{\text{inches}}{\text{hour}}\right)} 8.5 \text{ cfs}\right] \approx 58.0 \text{ cfs}$$

$$A_{eff(3)} = A_3 + \left(\frac{T_3}{T_1}\right)A_1 + \left(\frac{T_3}{T_2}\right)A_2 = 23.1 \text{ acres} + 4.2 \text{ acres} + 25.6 \text{ acres} = 52.9 \text{ acres}$$

$Q_{p(3)}$ = 58.0 cfs
$A_{eff(3)}$ = 52.9 acres
T_3 = 49.3 minutes

As shown in **Table 4-3**, Stream 1 produces the largest peak flow rate; it is then selected to continue the Rational Method calculations downstream to the watershed outlet. **Figure 4-5** and **Figure 4-6** show the completed Rational Method worksheet.

Table 4-3. Junction (Node 14) Analysis Summary

Stream	Peak Flow Rate $Q_{p(x)}$ (cfs)	Effective Area $A_{eff(x)}$ (acres)	Time of Concentration T_x (minutes)
Stream 1	72.1	41.2	24.4
Stream 2	64.7	30.0	17.0
Stream 3	58.0	52.9	49.3

Notes:

- The $F_{m\ avg}$ value must be determined for the subareas contributing flow to the confluence point (Node 14). The area-averaged F_m ($F_{m\ avg}$) value is determined to be 0.42 inches/hour and uses the sum of the total upstream area (52.9 acres), not the effective area (41.2 acres).
- The effective area of 41.2 acres, plus the additional subareas, is used to calculate the peak flow rates continuing downstream from the Node 14 confluence.

SAN BERNARDINO COUNTY HYDROLOGY MANUAL	STUDY NAME: EXAMPLE PROBLEM 100-YEAR STORM 1-HOUR RAINFALL (INCH) = 1.49 ANTECEDENT MOISTURE CONDITION (AMC) = AMC-II	Calculated by Date Checked by ____________ Date ________ Page 1 of 2

Concentration Point	Area (Acres) Subarea	Area (Acres) Total	Soil Type	Dev. Type	T_c min.	I in./hr.	F_m in./hr.	F_m avg.	Q Total
12 (11 → 12)	10.0	10.0	B	SF(1/3)	21.0	2.52	0.36	0.36	19.4
13 (12 → 13)	9.6	19.6	B	SF(1/4)	22.7	2.42	0.29	0.33	36.9
14 (13 → 14)	6.0	25.6	B	SF(1/4)	24.4	2.34	0.29	0.32	46.5
14 (11 → 14)		**25.6**			**24.4**				**46.5**
22 (21 → 22)	1.0	1.0	B	SF(1)	13.7	3.12	0.46	0.46	2.4
14 (22 → 14)	3.2	4.2	B	UNDEV (GRASS)	17.0	2.80	0.56	0.54	8.5
14 (21 → 14)		**4.2**							**8.5**
32 (31 → 32)	9.5	9.5	B	UNDEV (GRASS)	42.0	1.78	0.68	0.68	9.4
33 (32 → 33)	8.8	18.3	B	SF(2)	46.2	1.70	0.55	0.62	17.8
14 (33 → 14)	4.8	23.1	B	TH	49.3	1.64	0.10	0.51	23.5
14 (31 → 14)		**23.1**			**49.3**				**23.5**

$T_{i\ or\ t}$ min.	L feet	Slope ft./ft.	V_{ave} ft./sec	Notes
-	800	0.0025	-	INITIAL SUBAREA
1.7	350	0.0057	3.4	Street Flow n=0.018, B=20ft, Z=0
1.7	650	0.0031	6.2	Pipe Flow 39" RCP, n=0.013
				STREAM SUMMARY
-	400	0.0075	-	INITIAL SUBAREA
3.3	850	0.0200	4.3	Trapezoidal Channel n=0.025, B=0.5ft, Z=2
				STREAM SUMMARY
-	750	0.0027	-	INITIAL SUBAREA
4.2	550	0.0036	2.2	Street Flow n=0.018, B=20ft, Z=0
3.1	700	0.0014	3.8	Pipe Flow 36" RCP, n=0.013
				STREAM SUMMARY
				CONFLUENCE ANALYSIS FOR NODE 14
				CONFLUENCE RESULTS
	Use Q_{p1}	Use T_{c1}	Use A_{p1}	Stream 1 Dominates
	72.1 cfs	24.4 min	41.2 acre	$F_{m\ avg}$ = 0.42

$$Q_{p(1)} = 46.5\text{ cfs} + \left[\frac{\left(2.34\frac{\text{inches}}{\text{hour}} - 0.54\frac{\text{inches}}{\text{hour}}\right)}{\left(2.80\frac{\text{inches}}{\text{hour}} - 0.54\frac{\text{inches}}{\text{hour}}\right)} 8.5\text{ cfs}\right] + \left[\frac{(24.4\text{ minutes})}{(49.3\text{ minutes})}\frac{\left(2.34\frac{\text{inches}}{\text{hour}} - 0.51\frac{\text{inches}}{\text{hour}}\right)}{\left(1.64\frac{\text{inches}}{\text{hour}} - 0.51\frac{\text{inches}}{\text{hour}}\right)} 23.5\text{ cfs}\right] \approx 72.1\text{ cfs}$$

$$Q_{p(2)} = 8.5\text{ cfs} + \left[\frac{(17.0\text{ minutes})}{(24.4\text{ minutes})}\frac{\left(2.80\frac{\text{inches}}{\text{hour}} - 0.32\frac{\text{inches}}{\text{hour}}\right)}{\left(2.34\frac{\text{inches}}{\text{hour}} - 0.32\frac{\text{inches}}{\text{hour}}\right)} 46.5\text{ cfs}\right] + \left[\frac{(17.0\text{ minutes})}{(49.3\text{ minutes})}\frac{\left(2.80\frac{\text{inches}}{\text{hour}} - 0.51\frac{\text{inches}}{\text{hour}}\right)}{\left(1.64\frac{\text{inches}}{\text{hour}} - 0.51\frac{\text{inches}}{\text{hour}}\right)} 23.5\text{ cfs}\right] \approx 64.7\text{ cfs}$$

$$Q_{p(3)} = 23.5\text{ cfs} + \left[\frac{\left(1.64\frac{\text{inches}}{\text{hour}} - 0.32\frac{\text{inches}}{\text{hour}}\right)}{\left(2.34\frac{\text{inches}}{\text{hour}} - 0.32\frac{\text{inches}}{\text{hour}}\right)} 46.5\text{ cfs}\right] + \left[\frac{\left(1.64\frac{\text{inches}}{\text{hour}} - 0.54\frac{\text{inches}}{\text{hour}}\right)}{\left(2.80\frac{\text{inches}}{\text{hour}} - 0.54\frac{\text{inches}}{\text{hour}}\right)} 8.5\text{ cfs}\right] \approx 58.0\text{ cfs}$$

$$A_{eff(1)} = A_1 + \left(\frac{T_1}{T_2}\right)^{\nearrow 1} A_2 + \left(\frac{T_1}{T_3}\right) A_3 = 25.6\text{ acres} + 4.2\text{ acres} + \left(\frac{24.4\text{ minutes}}{49.3\text{ minutes}}\right) 23.1\text{ acres} \approx 41.2\text{ acres}$$

Figure 4-5. Rational Method Example Worksheet (Page 1 of 2)

SAN BERNARDINO COUNTY HYDROLOGY MANUAL	STUDY NAME: 100-YEAR STORM 1-HOUR RAINFALL (INCH) = 1.49 ANTECEDENT MOISTURE CONDITION (AMC) = AMC-II	Calculated by ________ Date ________ Checked by ________ Date ________ Page 2 of 2

Concentration Point	Area (Acres) Subarea	Area (Acres) Total	Soil Type	Dev. Type	$T_{i \text{ or } t}$ min.	T_c min.	I in./hr.	F_m in./hr.	F_m avg.	Q Total	L feet	Slope ft./ft.	V_{ave} ft./sec	Notes
15 (14 → 15)	5.1	46.3	B	INDUS	1.2	25.6	2.28	0.06	0.39	78.8	550	0.0036	7.7	Pipe Flow 48" RCP, n=0.013
15 (14 → 15)	4.8	51.1	B	TH	-	25.6	2.28	0.10	0.37	87.8	Same Concentration Point as the Subarea Above			
16 (15 → 16)	10.8	61.9	B	COMM	2.8	28.4	2.17	0.02	0.32	103.1	700	0.0029	6.2	Trapezoidal Channel n=0.025, B=2ft, Z=2
16 (Outlet)		**61.9**				**28.4**				**103.1**				**RATIONAL METHOD SUMMARY**

Figure 4-6. Rational Method Example Worksheet (Page 2 of 2)

4.5. Rational Method Hydrograph Calculations

In certain instances (e.g., unsteady flow hydraulic modeling), a runoff hydrograph may be required based on the results from Rational Method calculations. The following variables, T_c, I, F_m, $\overline{Y}$, F*, and A, are obtained as outlined below.

- Obtain the time of concentration (T_c) from the Rational Method study results at the location of interest; this value will be used as the unit interval. This ensures that the peak flow rate of the hydrograph matches the Rational Method peak flow rate.
- Calculate the rainfall intensity, I, following the procedures in **Section 4.1.3** at each time step.
- Determine the maximum loss rate, F_m, for the watershed using **Equation 3-7**.
- Determine watershed low loss fraction, $\overline{Y}$, and the low loss rate, F*, for the watershed using **Equation 3-3** through **Equation 3-6**.
- Obtain the watershed area (A) from the Rational Method study results at the concentration point of interest.

The steps for constructing a Rational Method hydrograph are as follows:

1. Create the cumulative hyetograph by placing rainfall depths at each peak rainfall unit number (PRUN). From 5 minutes to 60 minutes, the rainfall depths can be calculated from the Rational Method study IDC (i.e., **Figure 2-3**). If the hyetograph must be extended beyond 60 minutes, use NOAA precipitation data and perform log-log interpolation to estimate the intermediate rainfall depths between known values.
2. Create the incremental hyetograph (Column 4) from the cumulative hyetograph developed in Step 1.
3. Convert the incremental rainfall depths from Step 2 to incremental rainfall intensities (Column 5).
4. At each PRUN, calculate the low loss rate (F*) (Column 6) and the maximum loss rate (Fm) (Column 7), as presented in **Section 3.6.3** and **Section 3.6.5**, respectively.
5. Determine the governing loss rate (Column 8) from Step 4 and calculate the effective rainfall intensity (Column 9) at each PRUN.
6. Calculate the flow rate (Q) at each PRUN using **Equation 4-8**. Note that the “I-Fm” term was determined in Step 5.
7. Based on the engineer-specified total storm duration (e.g., 24 hours), develop the hydrograph using the 2/3, 1/3 placement scheme as illustrated in **Figure 4-8**.

4.6. Rational Method Hydrograph Example

The following data for a Rational Method study are assumed:

Watershed area (A):	8 acres
Time of concentration (T_c):	10 minutes (unit interval)
Maximum loss rate (F_m):	0.12 inches/hour
Low loss fraction ($\bar{Y}$):	0.35
NOAA Atlas 14 Rainfall Data:	The rainfall depths are 0.98 inches for a 60-minute duration and 1.31 inches for a 120-minute duration.

1. **Create the cumulative hyetograph by placing rainfall depths at each PRUN. From 5 minutes to 60 minutes, the rainfall depths can be calculated from the Rational Method study IDC (i.e., Figure 2-3). If the hyetograph must be extended beyond 60 minutes, use NOAA precipitation data and perform log-log interpolation to estimate the intermediate rainfall depths between known values.**

   To create the cumulative rainfall depths (Column 3 of **Table 4-4**; note that all column references are to **Table 4-4**), the IDC from the Rational Method study is used, whereby the cumulative rainfall intensities are obtained at each PRUN (see **Figure 4-7**) up to 60 minutes (Column 2). The extracted cumulative intensity values are then converted into cumulative rainfall depths.

   An example of these calculations is shown for PRUNs 1 and 2:

$$\text{Cumulative Rainfall Depth (inches)} = \left(\text{Cum. Rainfall Intensity}\left(\frac{\text{inches}}{\text{hour}}\right)\right)\left(\frac{\text{Cum. Storm Duration (minutes)}}{\text{60 minutes}}\right)$$

   PRUN1 (10-minute duration):

$$\text{Cumulative Rainfall Depth (inches)} = \left(2.40\,\frac{\text{inches}}{\text{hour}}\right)\left(\frac{\text{10 minutes}}{\text{60 minutes}}\right) = 0.40\text{ inches}$$

   PRUN2 (20-minute duration):

$$\text{Cumulative Rainfall Depth (inches)} = \left(1.70\,\frac{\text{inches}}{\text{hour}}\right)\left(\frac{\text{20 minutes}}{\text{60 minutes}}\right) = 0.57\text{ inches}$$

   At the 120-minute duration (PRUN12) and 180-minute duration (PRUN18) points, the NOAA Atlas 14 values of 1.31 inches and 1.56 inches, respectively, are input. To obtain rainfall depths beyond 60 minutes, log-log interpolation should be performed between the known rainfall depths.

   An example of this calculation is shown for PRUN7 at the 70-minute duration. From the known rainfall depths at 60 minutes (0.98 inches) and 120 minutes (1.31 inches), the log-log slope is calculated:

$$\text{Log-Log slope} = \left(\frac{\log_{10}\left(\frac{\text{1.31 inches}}{\text{0.98 inches}}\right)}{\log_{10}\left(\frac{\text{120 minutes}}{\text{60 minutes}}\right)}\right) = 0.42$$

Using the log-log slope of 0.42, the cumulative rainfall depth at PRUN7 is interpolated:

$$0.42 = \left(\frac{\log_{10}\left(\frac{\text{PRUN7 inches}}{\text{0.98 inches}}\right)}{\log_{10}\left(\frac{\text{70 minutes}}{\text{60 minutes}}\right)} \right)$$

$$\text{PRUN7} \approx 1.05 \text{ inches}$$

2. **Create the incremental hyetograph (Column 4) from the cumulative hyetograph developed in Step 1.**

   The cumulative rainfall depth (Column 3) is converted into incremental rainfall depth (Column 4) by subtracting the cumulative rainfall depth value from the preceding cumulative rainfall depth value (e.g., for PRUN5, cumulative rainfall depth: 0.89 - 0.80 = 0.09 inches).

3. **Convert the incremental rainfall depths from Step 2 to incremental rainfall intensities (Column 5).**

   The incremental rainfall depths (Column 4) are converted to rainfall intensities (Column 5) by dividing by the unit interval of 10 minutes and converting the minutes into hours. An example of this conversion is shown for PRUN5:

$$\text{Rainfall Intensity}\left(\frac{\text{inches}}{\text{hour}}\right) = \frac{\text{Incremental Rainfall Depth (inches)}}{T_c\ \text{(minutes)}\ \frac{\text{1 hour}}{\text{60 minutes}}} = \frac{\text{0.09 inches}}{\text{10 minutes x}\ \frac{\text{1 hour}}{\text{60 minutes}}}$$

$$= 0.54\,\frac{\text{inches}}{\text{hour}}$$

4. **At each PRUN, calculate the low loss rate (F*) (Column 6) and the maximum loss rate (F_m) (Column 7), as presented in Section 3.6.3 and Section 3.6.5, respectively.**

   The low loss rate (F*) is determined by using the respective incremental rainfall intensity (Column 5), low loss fraction ($\bar{Y}$ = 0.35), and applying **Equation 3-6**. An example of this calculation is shown for PRUN5:

$$F^* = \bar{Y}I = 0.35 \text{ x } 0.54\ \frac{\text{inches}}{\text{hour}} \approx 0.19\ \frac{\text{inches}}{\text{hour}}$$

   The maximum loss rate (F_m) is assumed from the Rational Method study (F_m = 0.12 inches/hour) as shown in Column 7. Note that in this example, $F_m = F_{m\ avg}$.

5. **Determine the governing loss rate (Column 8) from Step 4 and calculate the effective rainfall intensity (Column 9) at each PRUN.**

   The governing loss rate is the low loss rate unless the low loss rate exceeds the maximum loss rate; in that case, the maximum loss rate is used. Once the governing loss rate is determined (F* or F_m), the effective rainfall intensity (I_e) is calculated by subtracting the governing loss rate (Column 8) from the incremental rainfall intensity (Column 5), as shown in Column 9.

6. **Calculate the flow rate (Q) at each PRUN using Equation 4-8. Note that the "I-F_m" term was determined in Step 5.**

   The peak flow rate is determined for each respective PRUN using **Equation 4-8**.

7. **Based on the engineer-specified total storm duration (e.g., 24 hours), develop the hydrograph using the 2/3, 1/3 placement scheme as illustrated in Figure 4-8.**

   The peak flow rate results in Column 10 are used to create the total storm duration hydrograph (see **Figure 4-8**). This is accomplished by using the flow value at PRUN1 (i.e., maximum flow rate) and placing it at the 2/3 total storm duration location. Next, the two subsequent peak flow rates (i.e., PRUNs 2 and 3) are placed on the left side of the hydrograph and the following peak flow rate (i.e., PRUN4) on the right side of the hydrograph. This alternating placement (i.e., left, left, right, left, left, right, etc.) continues until all calculated flows have been assigned along the hydrograph.

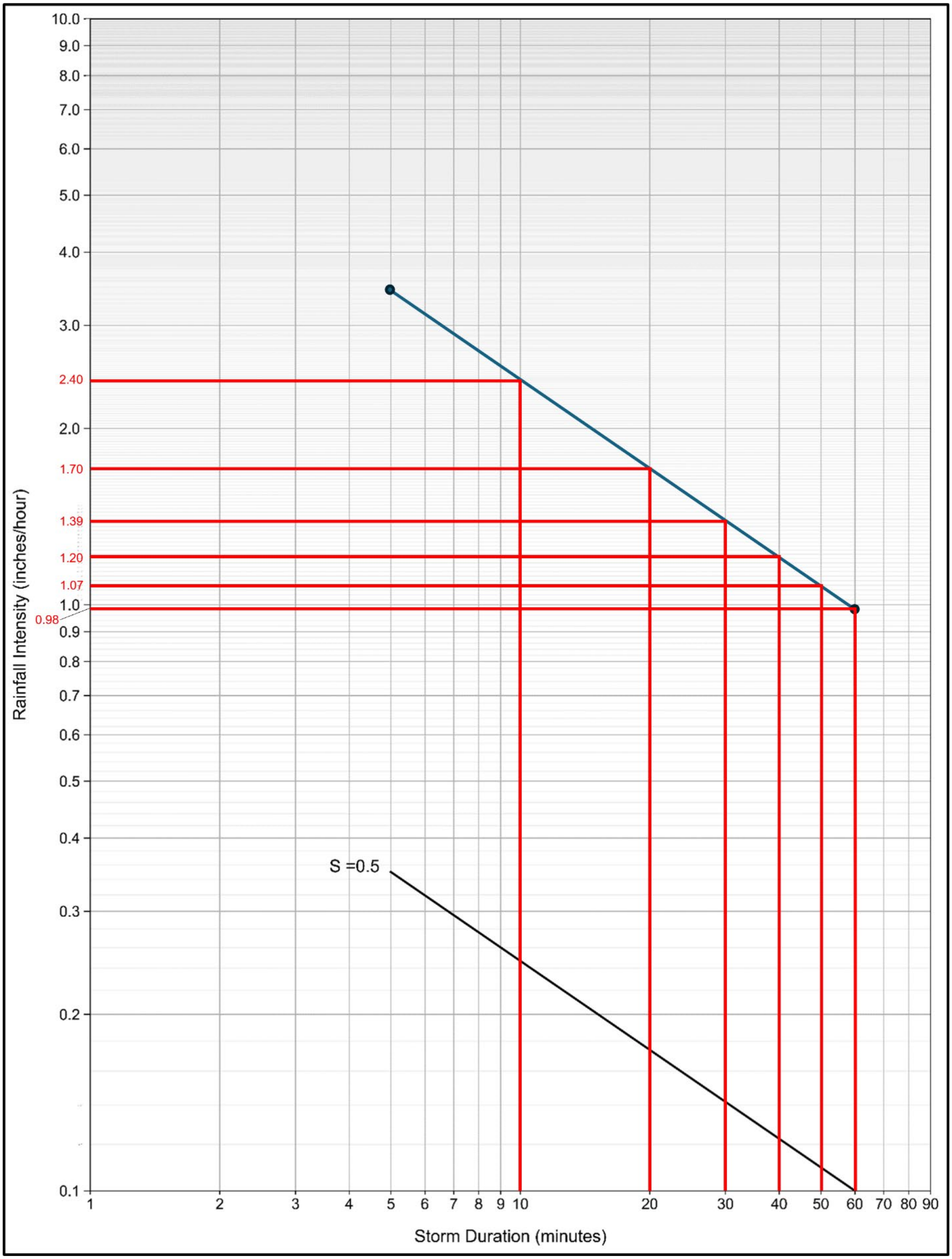


Figure 4-7. Intensity-Duration Curve for Rational Method Hydrograph Example

Table 4-4. Rational Method Hydrograph Example Tabular Results

Peak Rainfall Unit Number	Cumulative Rainfall Intensity I_c (inches/hour)	Cumulative Rainfall Depth (inches)	Incremental Rainfall Depth (inches)	Incremental Rainfall Intensity I (inches/hour)	Low Loss Rate F* (inches/hour)	Maximum Loss Rate F_m (inches/hour)	Governing Loss Rate F* or F_m (inches/hour)	Effective Rainfall Intensity I (inches/hour)	Flow Rate Q(cfs)
1	2	3	4	5	6	7	8	9	10
1	2.40	0.40	0.40	2.40	0.84	0.12	0.12	2.28	16.42
2	1.70	0.57	0.17	1.01	0.35	0.12	0.12	0.89	6.41
3	1.39	0.70	0.13	0.76	0.27	0.12	0.12	0.64	4.61
4	1.20	0.80	0.11	0.63	0.22	0.12	0.12	0.51	3.67
5	1.07	0.89	0.09	0.54	0.19	0.12	0.12	0.42	3.02
6	0.98	0.98	0.09	0.54	0.19	0.12	0.12	0.42	3.02
7	NA	1.05	0.07	0.42	0.15	0.12	0.12	0.30	2.16
8	NA	1.11	0.06	0.36	0.13	0.12	0.12	0.24	1.73
9	NA	1.16	0.05	0.30	0.11	0.12	0.11	0.19	1.37
10	NA	1.22	0.06	0.36	0.13	0.12	0.12	0.24	1.73
11	NA	1.26	0.04	0.24	0.08	0.12	0.08	0.16	1.15
12	NA	1.31	0.05	0.30	0.11	0.12	0.11	0.19	1.37
13	NA	1.36	0.05	0.30	0.11	0.12	0.11	0.19	1.37
14	NA	1.40	0.04	0.24	0.08	0.12	0.08	0.16	1.15
15	NA	1.44	0.04	0.24	0.08	0.12	0.08	0.16	1.15
16	NA	1.48	0.04	0.24	0.08	0.12	0.08	0.16	1.15
17	NA	1.52	0.04	0.24	0.08	0.12	0.08	0.16	1.15
18	NA	1.56	0.04	0.24	0.08	0.12	0.08	0.16	1.15

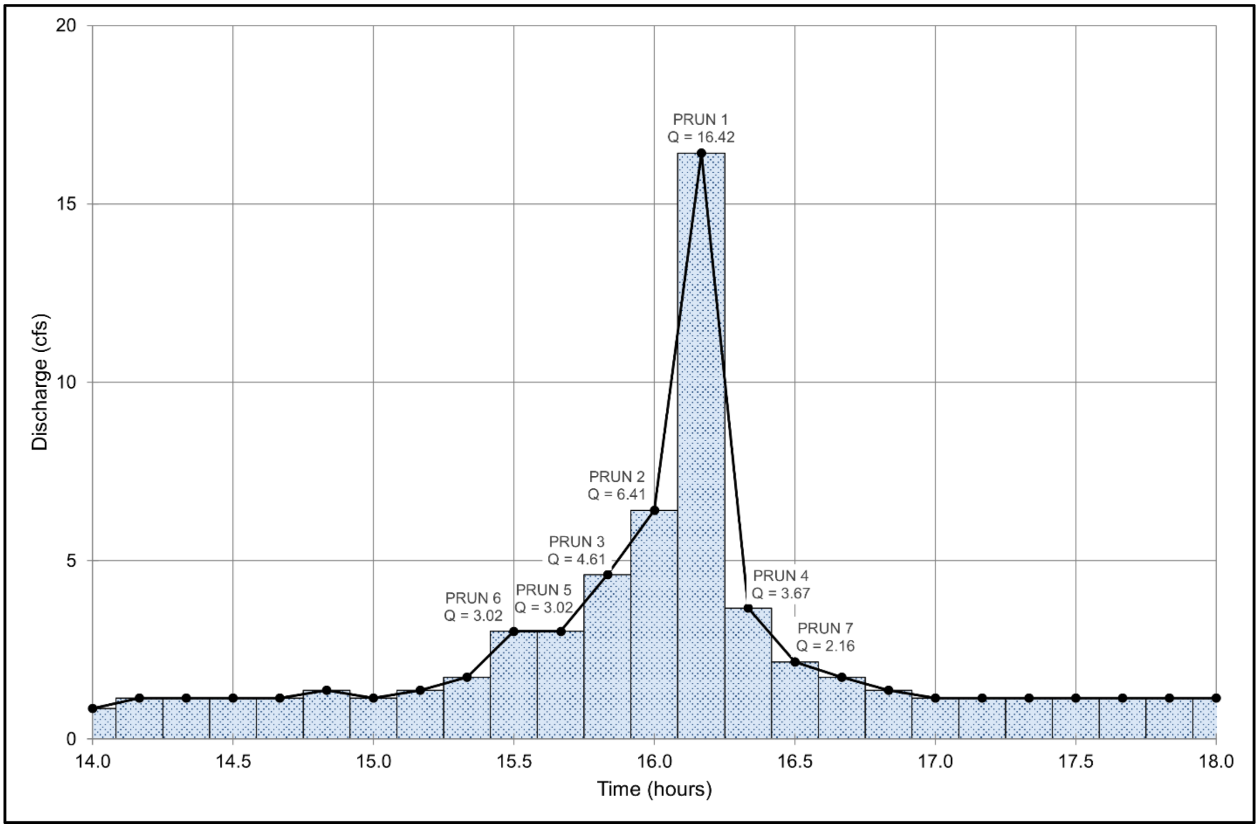


Figure 4-8. Rational Method Hydrograph Example Flow Placement

CHAPTER 5
UNIT HYDROGRAPH METHOD

5.1. Unit Hydrograph Method

The unit hydrograph (UH) method assumes that the watershed discharge relates to the total volume of runoff, that the time factors affecting the UH shape do not vary, and that the watershed area, slope, and shape factors characterize the watershed storm rainfall-runoff relationships. The UH method is often used to estimate the time distribution of watershed runoff where stream gauge information is either unavailable or inadequate to justify statistical interpretation.

For the purposes of this Manual, the UH method will be used for watershed areas larger than approximately 1 sq. mi. (640 acres).

5.2. Terminology

The following definitions are used in the discussion of UH and runoff hydrograph calculations:

Effective Rainfall: Total rainfall less infiltration, evaporation, transpiration, interception, and storage.

Unit Hydrograph: A time distribution of runoff flow rates resulting from 1 inch of effective rainfall occurring uniformly over the tributary watershed upstream of a point of concentration, during a specified unit time period. The unit effective rainfall is generally assumed to occur as an equivalent constant rainfall intensity during a specified unit period of time.

Distribution Graph: A unit hydrograph whose ordinates are expressed in terms of percent of ultimate discharge. A distribution graph is generally developed as a block graph with each block representing its associated percentage of unit runoff which occurs during the specified unit time period. The unit time period used in the distribution graph is identical to the unit time period specified for the unit hydrograph.

Summation Hydrograph: A curve illustrating the time distribution of runoff rates resulting from a continuous series of unit period effective rainfalls over the tributary watershed upstream of the point of concentration. The ordinates of the summation hydrograph are expressed in percent of the ultimate discharge.

Lag: The time from the beginning of a continuous series of effective unit period rainfall over a watershed and the point at which the runoff rate at the concentration point reaches 50% of its peak value.

Ultimate Discharge: The maximum rate of watershed runoff resulting from a specified effective rainfall intensity. In this Manual, for an effective rainfall rate of 1 inch occurring over a unit period of 1 hour, the ultimate discharge is 645 cfs for every sq. mi. of watershed area. For other unit periods, the ultimate discharge is calculated by dividing 645 by the unit period in hours and multiplying by the watershed area in square miles.

S-Graph: A summation hydrograph developed by plotting watershed discharge (percent of ultimate discharge) as a function of time (percent of lag).

5.3. Runoff Hydrograph Development Using Synthetic Unit Hydrographs (UHs)

To calculate a runoff hydrograph from a UH at a concentration point using this Manual, the primary information requirements include the: (1) synthetic UH for the tributary watershed, (2) effective hyetograph for the tributary watershed, and (3) unit time period ordinate hydrograph convolution results. The following sections describe these processes in further detail.

5.3.1. Synthetic Unit Hydrograph (UH) Development

To generate a synthetic UH for a study watershed, the tributary watershed area and lag, discussed in **Section 5.3.1.1** and **Section 5.3.1.2**, respectively, need to be calculated. Additionally, an S-graph (see **Section 5.3.1.3**) representative of the watershed must also be selected.

5.3.1.1. Watershed Area

While there are no specified maximum restrictions for watershed area as there are for Rational Method calculations, increasing the homogeneity of input parameter values by using watershed subarea delineations is highly recommended. Improving delineation granularity by considering characteristics—including, but not limited to, land use, vegetation type and density, hydrologic soil types, general pervious and impervious area percentages and connectivity to the main flow path, slope (area and main flow path), and surface roughness (flow path and subarea)—helps reduce the hydrologic calculation uncertainty. As discussed in **Section 4.1.5**, accounting for cross-watershed/subarea spillover is also necessary where applicable.

5.3.1.2. Lag

Lag time for a watershed is defined as the elapsed time from the beginning of effective rainfall to the point at which the direct runoff hydrograph at the watershed outlet reaches 50% of its peak discharge. When lag times derived from hydrograph analyses of several gauged watersheds are correlated with their hydrologic characteristics, empirical relationships often emerge. These relationships can be applied to estimate lag times for ungauged drainage areas with known characteristics. By comparing lag values obtained from rainfall-runoff data with time of concentration (T_c) estimates derived using the Rational Method, a relationship—such as **Equation 5-1**—is developed. This equation is commonly used in UH studies where sufficient topographic data are available to compute T_c.

Equation 5-1 Tc to Lag Conversion

$$T_{lag} = 0.8T_c$$

where:

T_{lag} = Lag time (hours)
T_c = Time of concentration (hours)

The Rational Method time of concentration calculation methodology needed to use **Equation 5-1** is presented in **Chapter 4**. Because the time of concentration is related to the flow rate through a watershed, the results will vary based on the storm frequency and duration; for example, the time of concentration for a 2-year storm will typically be longer than a 100-year storm. Therefore, the Rational Method time of concentration storm frequency and duration must correspond to the lag storm frequency and duration of interest.

For the case of large-scale natural condition watershed studies, particularly those that exceed the applicability of the Rational Method (see **Chapter 4**), **Equation 5-2**, developed by USACE is recommended to calculate lag:

Equation 5-2 USACE Lag

$$T_{lag} = C_t\left((LL_{ca})/S^{0.5}\right)^m$$

where:

T_{lag} = Lag time (hours)
C_t = 24 $\bar{n}$; a constant determined by regional flood reconstitution studies ($\bar{n}$ is the visually estimated basin factor of all collection streams and watershed channels, see **Table 5-1**)
L = Length of longest watercourse (miles)
L_{ca} = Length along longest watercourse, measured from the outlet upstream to a point opposite center of area (miles)
S = Overall slope of drainage area between the headwaters and the collection point (feet per mile)
m = 0.38; a constant determined by regional flood reconstitution studies

Table 5-1. Basin Factor Descriptions

$\bar{n}$	Drainage Area Slopes	Drainage Improvements	Groundcover	Main Watercourse
0.015	Fairly uniform, gentle slopes	Most watercourses either improved or along paved streets	Some grasses; large % of area is impervious	Improved channel or conduit
0.020	Some graded and non-uniform, gentle slopes	Over half of the area watercourses are improved or paved streets	Equal amounts grasses and impervious area	Partly improved channel or conduit and partly greenbelt (see $\bar{n}$ = 0.025)
0.025	Generally rolling with gentle side slopes	Some drainage improvements in the area—streets and canals	Mostly scattered brush and grass; small % impervious	Straight channels that are turfed or with stony beds and weeds on earth bank (greenbelt type)
0.030	Generally rolling with rounded ridges and moderate side slopes	No drainage improvements exist in the area	Includes scattered brush and grasses	Meandering in fairly straight, unimproved channels with some boulders and lodged debris
0.040	Steep upper canyons with moderate slopes in lower canyons	No drainage improvements exist in the area	Mixed brush and trees with grasses in lower canyons	Meandering with moderate bends and moderately impeded by boulders and debris
0.050	Quite rugged with sharp ridges and steep canyons	No drainage improvements exist in the area	Excluding small areas of rock outcrops, includes many trees and considerable underbrush	Meandering around sharp bends and over large boulders; considerable debris obstruction
0.200	Comparatively uniform slopes	No drainage improvements exist in the area	Cultivated crops or substantial growth of grass and fairly dense small shrubs, cacti, or similar vegetation	Surface characteristics are such that channelization does not occur

5.3.1.3. S-Graphs

S-graphs are summation hydrographs modified so that the percent of ultimate discharge is related to time expressed in percent of lag. The derivation of an S-graph is identical to that of a summation hydrograph, except that time is normalized by the lag value. In accordance with the definition of lag, each S-graph reaches 50% of ultimate discharge at 100% of the lag time.

San Bernardino County uses five standardized S-graphs for UH development, corresponding to characteristic watershed types. These S-graphs are:

Valley–Developed: Applies to watershed conditions where prismatic channels exist or are proposed for design storm conveyance. This S-graph represents fully urbanized watersheds with existing or planned free-draining storm drains and flood control channels, and no detention or retention facilities. See **Figure 5-1**.

Valley–Undeveloped: Applies to natural watersheds whose channels are not sharply incised (e.g., anticipated floodplain conveyance). Additionally, if the watershed is developed, but interior drainage experiences significant ponding and storage impacts, this S-graph may be more appropriate than the Valley–Developed. See **Figure 5-2**.

Foothill: Used for sharply incised channels in natural watersheds (e.g., canyon bottoms where overbank flow is largely confined to the main channel). See **Figure 5-3**.

Mountain: Appropriate for mountainous watersheds characterized by steep natural channels with numerous plunging flow reaches and lodged boulders/debris. See **Figure 5-4**.

Desert: Generally applicable to watersheds located in areas characterized by desert environments. See **Figure 5-5**.

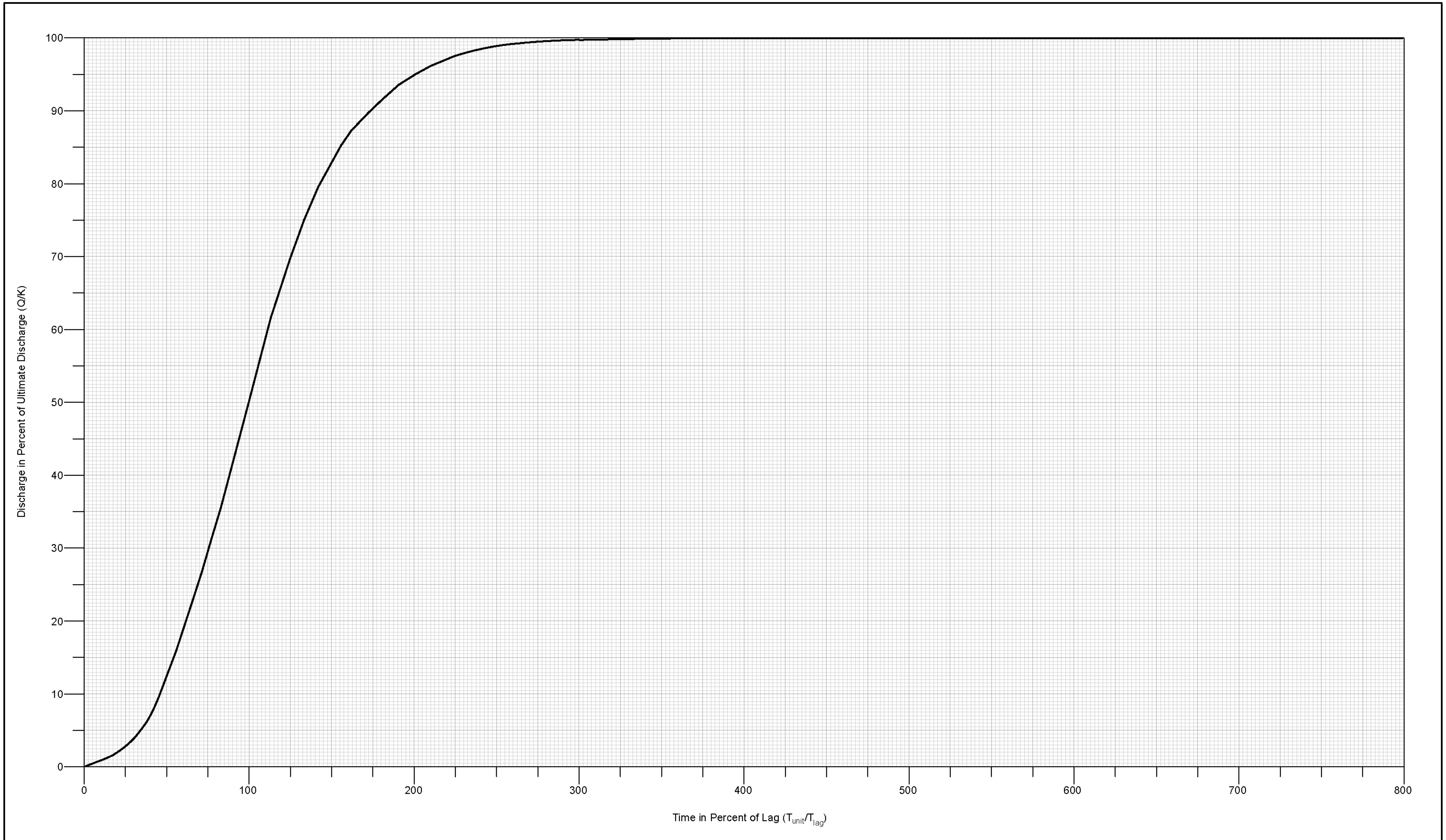


Figure 5-1. S-Graph for Valley–Developed

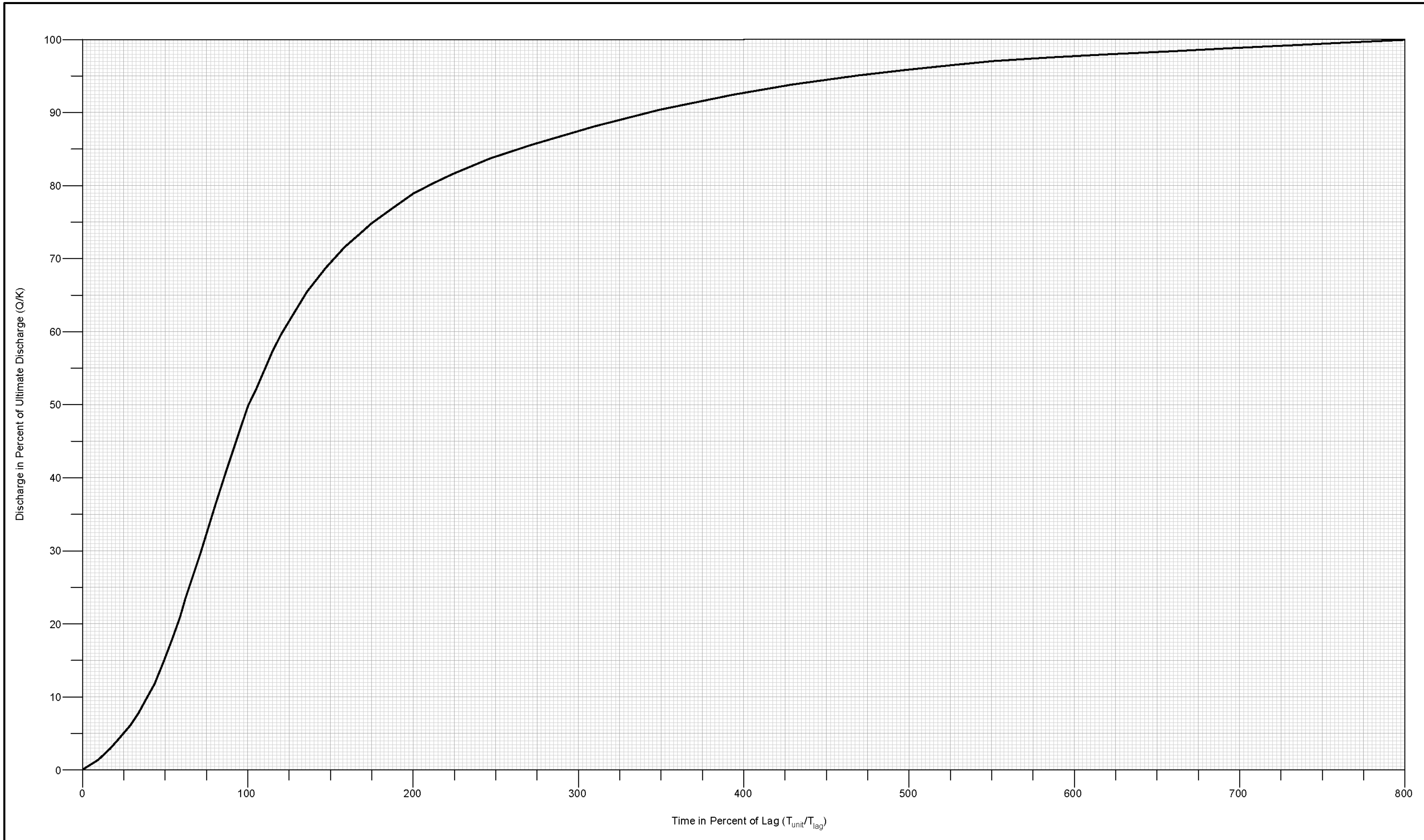


Figure 5-2. S-Graph for Valley–Undeveloped

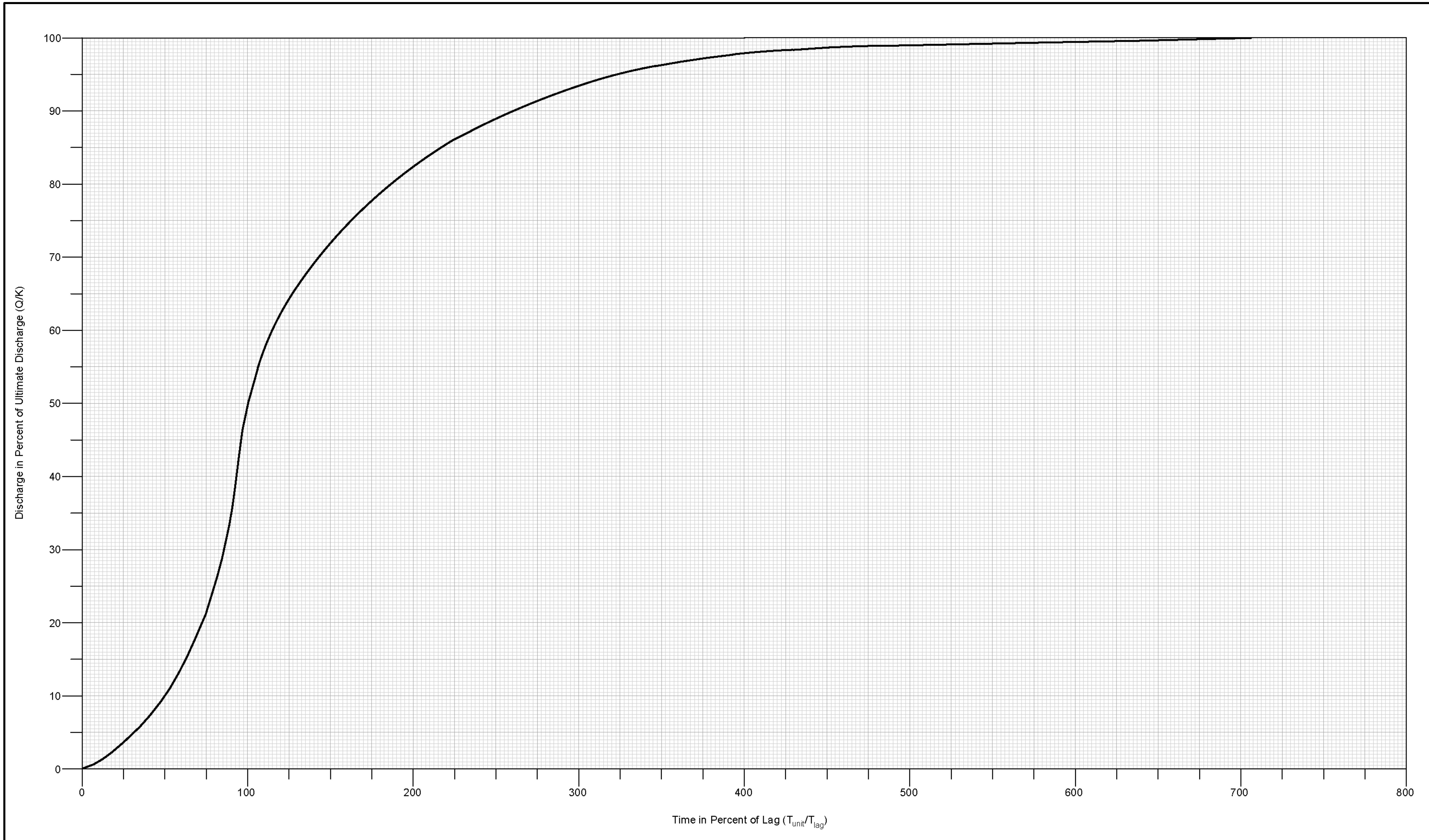


Figure 5-3. S-Graph for Foothill Areas

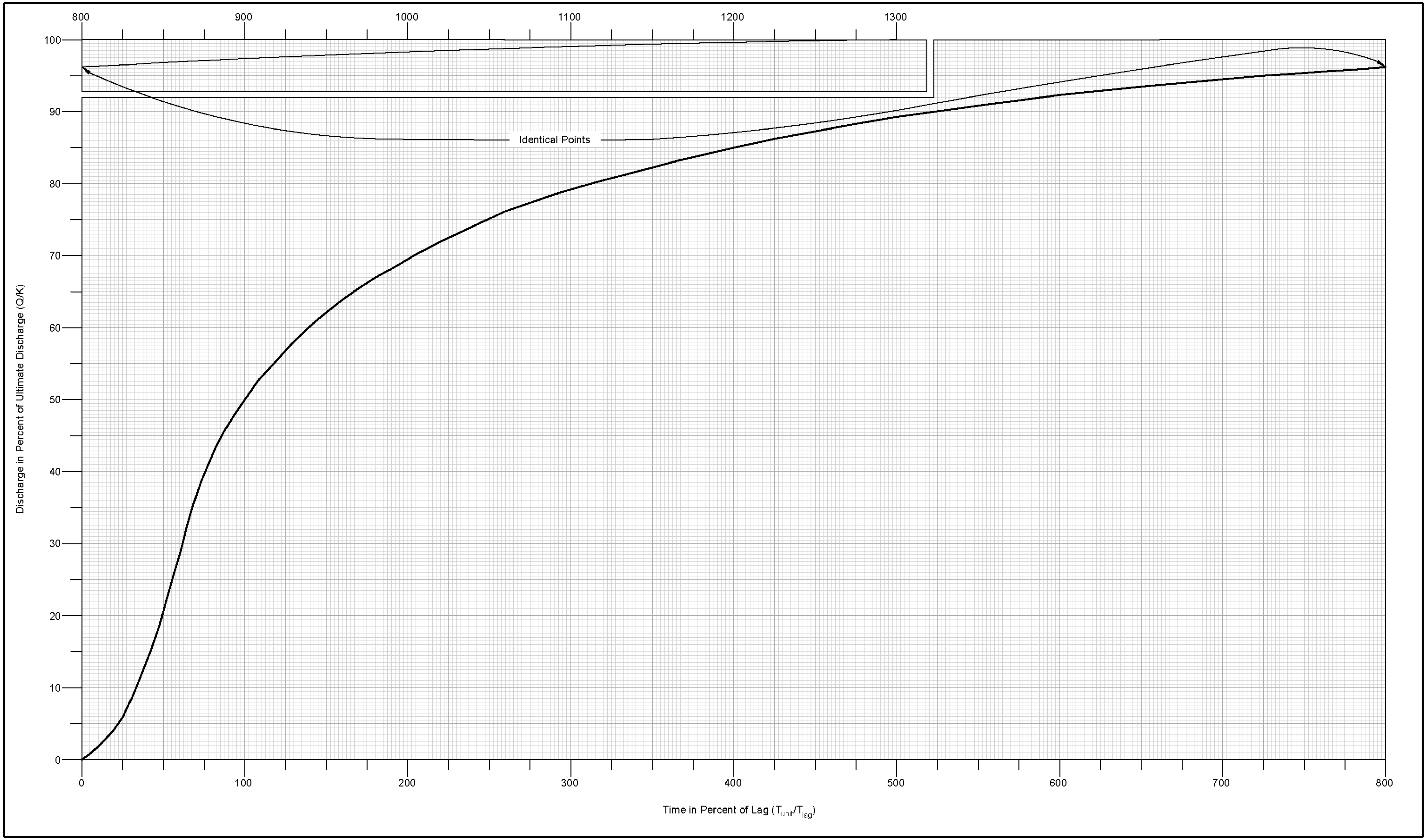


Figure 5-4. S-Graph for Mountain Areas

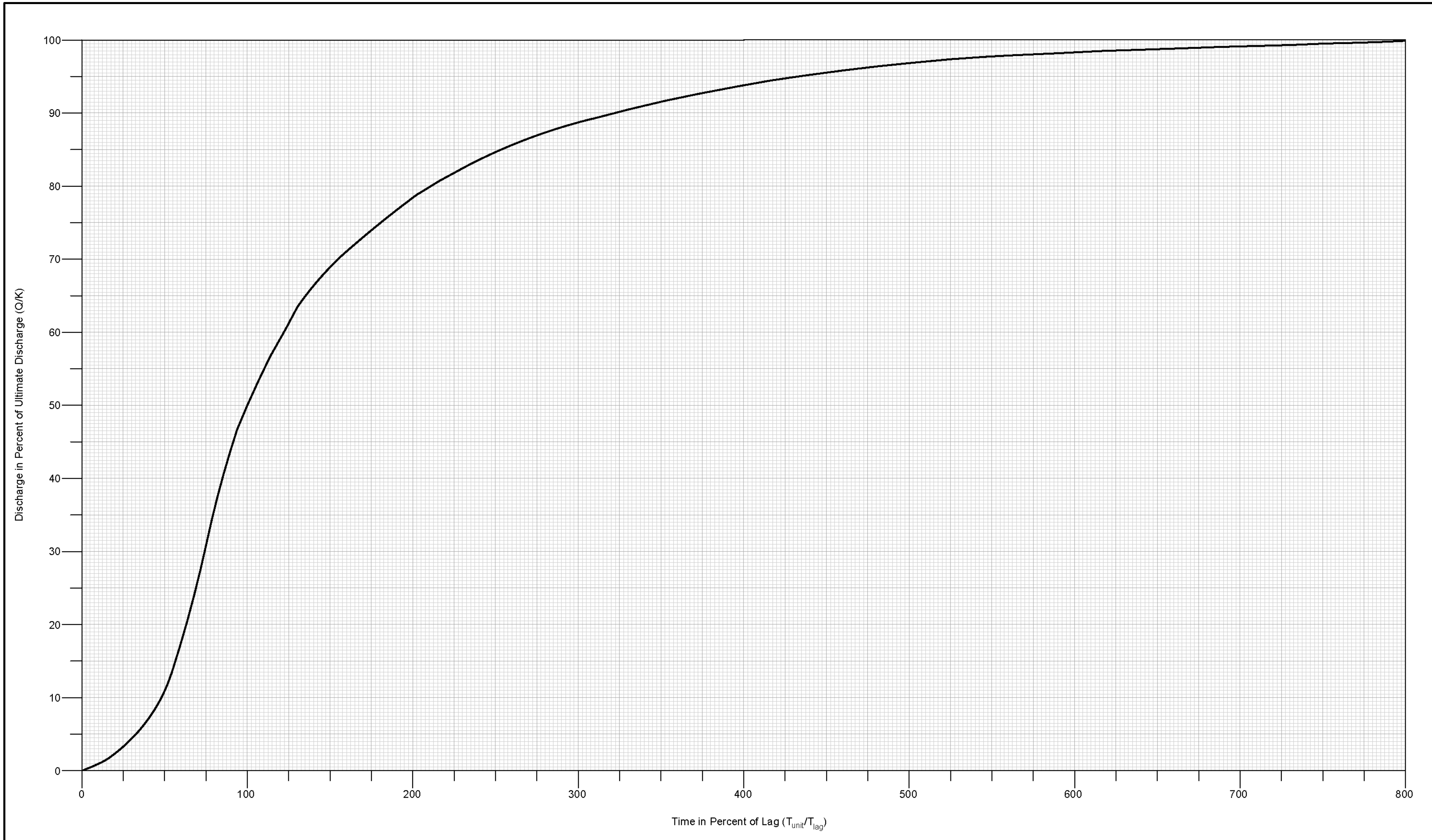


Figure 5-5. S-Graph for Desert Areas

5.3.1.4. Synthetic Unit Hydrograph (UH) Development

The following steps are general guidelines for developing a synthetic UH. **Section 5.3.1.5** provides examples illustrating the use of these guidelines.

1. Delineate the study watershed area (see **Section 5.3.1.1**) contributing to the point of concentration, using the best available surveying data and tools. Acceptable methods include digital delineation using software (e.g., GIS or AutoCAD tools) or manual methods based on digital elevation models (DEMs), topographic contour maps, aerial photos, and field surveys.
2. Calculate the watershed lag time (T_{lag}) using the most applicable of either **Equation 5-1** or **Equation 5-2**.
3. Select a unit time period (T_{unit}) approximately 15% to 25% of the T_{lag}. If possible, use the unit time period of the synthetic critical storm pattern of 5 minutes.
4. At each unit time period ordinate (UTPO), calculate the cumulative unit time period expressed as a percentage of lag (i.e., $\sum(T_{unit}/T_{lag})$).
5. Choose the appropriate S-graph for the watershed type (see **Section 5.3.1.3**).
6. Using the cumulative unit time period in percent of lag from Step 4 and the S-graph from Step 5, determine the corresponding average percentage of ultimate discharge at each UTPO.
7. Calculate the incremental discharge percentages by differencing the cumulative discharge values from Step 6 at each UTPO.
8. Compute the ultimate watershed discharge using **Equation 5-3**.

Equation 5-3 Ultimate Discharge

$$K = 645\,(A/T_{unit})$$

where:

K	=	The ultimate discharge (cfs)
A	=	Watershed area (sq. mi.)
T_{unit}	=	Unit time period (hours)

9. Multiply the incremental ultimate discharge percentage ordinate from Step 7 by the ultimate watershed discharge (K) calculated in Step 8 to obtain the UH ordinates (in flow units) at each UTPO.

5.3.1.5. Synthetic Unit Hydrograph (UH) Example

1. **Delineate the study watershed area (see Section 5.3.1.1) contributing to the point of concentration, using the best available surveying data and tools. Acceptable methods include digital delineation using software (e.g., GIS or AutoCAD tools) or manual methods based on DEMs, topographic contour maps, aerial photos, and field surveys.**

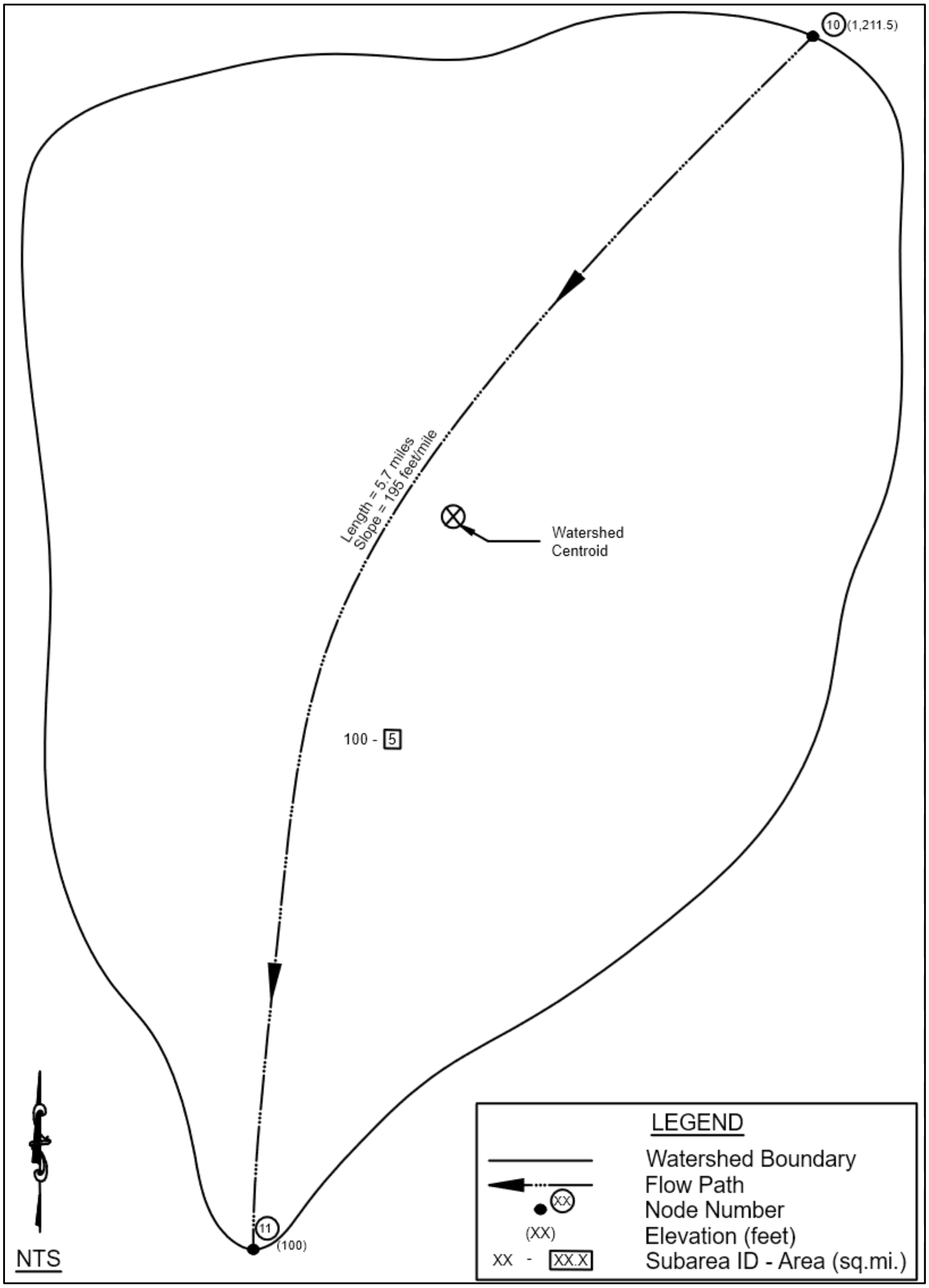


Figure 5-6. Example Study Watershed

2. **Calculate the watershed lag time (T_{lag}) using the most applicable of either Equation 5-1 or Equation 5-2.**

   The time of concentration (T_c) was not previously determined for this study watershed area; therefore, **Equation 5-1** cannot be used to convert T_c to lag time (T_{lag}). T_{lag} is calculated using **Equation 5-2** (USACE Lag). The watershed parameters needed to calculate T_{lag} can usually be estimated from the same data used to delineate the study watershed area and **Table 5-1** (Basin Factor Descriptions). The watershed parameters are:

   Length of the longest watercourse (L): 5.7 miles
   Length along the longest watercourse to the centroid (L_{ca}): 3.6 miles
   (measured from the outlet going upstream to a point opposite of the centroid)
   Slope (S): 195 feet/mile
   (Overall slope of the drainage area between the headwaters and the collection point)
   Basin Factor ($\bar{n}$): 0.03
   Constant (C_t, where C_t = 24 $\bar{n}$): 0.72
   Constant (m): 0.38
   (Determined by regional flood reconstitution studies)

   Equation 5-2 is used to calculate T_{lag}:

$$T_{lag} = C_t\left((L \times L_{ca})/S^{0.5}\right)^m$$

$$T_{lag} = 0.72\left((5.7 \text{ miles} \times 3.6 \text{ miles})/(195\frac{\text{feet}}{\text{mile}})^{0.5}\right)^{0.38} = 0.83 \text{ hours}$$

$$T_{lag} = 0.83 \text{ hours} \approx 50 \text{ minutes}$$

3. **Select a unit time period (T_{unit}) approximately 15% to 25% of the T_{lag}. If possible, use the unit time period of the synthetic critical storm pattern of 5 minutes.**

   For this example, a unit time period (T_{unit}) equal to 5 minutes is selected, even though it is outside of the approximate range of 15% to 25% of T_{lag} of 7.5 to 12.5 minutes. A unit time period of 5 minutes will match the unit time period of the synthetic critical storm pattern and provide better resolution for the runoff hydrograph.

$$T_{unit} = 5 \text{ minutes} \therefore \frac{T_{unit}}{T_{lag}} = \frac{5 \text{ minutes}}{50 \text{ minutes}} \times 100\% = 10\% \text{ of } T_{lag}$$

4. **At each UTPO, calculate the cumulative unit time period expressed as a percentage of lag (i.e., $\Sigma(T_{unit}/T_{lag})$).**

   To calculate the cumulative time in percent of lag at each UTPO (Column 3 of **Table 5-2**; note that all column references apply to **Table 5-2**), the T_{unit} in percent of the T_{lag} value determined in Step 3 (10%) is multiplied by the UTPO (Column 1) up to 360% (180 minutes).

5. **Choose the appropriate S-graph for the watershed type (see Section 5.3.1.3).**

   The Valley–Developed (**Figure 5-1**) was selected for this watershed.

6. **Using the cumulative unit time period in percent of lag from Step 4 and the S-graph from Step 5, determine the corresponding average percentage of ultimate discharge at each UTPO.**

   Vertical lines (blue) were drawn at each cumulative time percentage (Column 3). Horizontal lines (green) are drawn to balance the areas bounded between two adjacent vertical lines, the S-graph curve, and the respective horizontal line. The average percentage of ultimate discharge for the respective UTPO is where the green line intersects the y-axis. Column 4 shows the cumulative (average) percentage of ultimate discharge for each UTPO as shown in **Figure 5-7**.

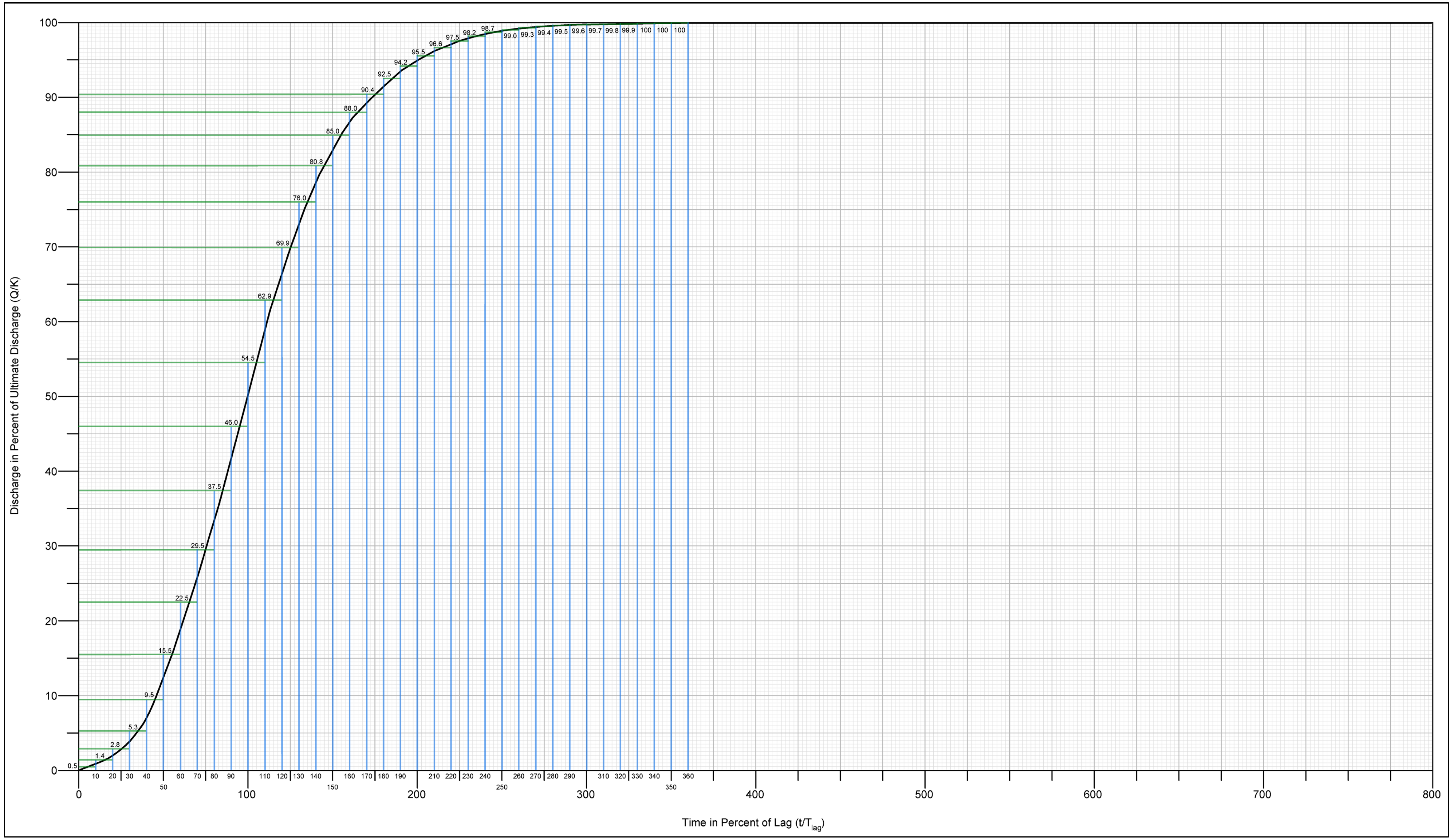


Figure 5-7. Discretized Valley–Developed S-Graph

7. **Calculate the incremental discharge percentages by differencing the cumulative discharge values from Step 6 at each UTPO.**

   The cumulative ultimate discharge percentage values (Column 4) are converted to incremental values (Column 5) at each UTPO by subtracting the cumulative ultimate discharge percentages between the UTPO of interest and the previous UTPO. UTPO 1 is the only exception (i.e., incremental value = cumulative value). An example of this calculation is shown for UTPO 5:

$$\text{Incremental Ultimate Discharge Percentage} \left(\frac{Q}{K}\right) = 9.5\% - 5.3\% = 4.2\%$$

8. **Compute the ultimate watershed discharge using Equation 5-3.**

   Watershed Area (A): 5.0 sq. mi.
   Unit time period (T_{unit}): $0.0833\overline{3}$ hours (5 minutes)

   The ultimate discharge is calculated using **Equation 5-3**:

$$K = 645\left(\frac{A}{T_{unit}}\right) = 645\left(\frac{5 \text{ sq. mi.}}{0.0833\overline{3} \text{ hours}}\right) = 38{,}700 \text{ cfs}$$

$$K = 38{,}700 \text{ cfs}$$

9. **Multiply the incremental ultimate discharge percentage ordinate from Step 7 by the ultimate watershed discharge (K) calculated in Step 8 to obtain the UH ordinates (in flow units) at each UTPO.**

   The ultimate discharge (K = 38,700 cfs) is multiplied by the incremental ultimate discharge percentage value (Column 5) at each UTPO to determine the synthetic UH discharge distribution. An example of this calculation is shown for UTPO 5:

$$\text{Synthetic Unit Hydrograph (cfs)} = 4.2\% \times 38{,}700 \text{ cfs} = 1{,}625.4 \text{ cfs}$$

   See **Table 5-2** and **Figure 5-8** for the synthetic UH example results.

Table 5-2. Synthetic Unit Hydrograph Example Results

Unit Time Period Ordinate	Cumulative Time		Cumulative Percentage of Ultimate Discharge	Incremental Ultimate Discharge Percentage	Synthetic Unit Hydrograph Discharge
	(minutes)	(in percent lag, T_{unit}/T_{lag})	(Q/K)	(Q/K)	(cfs)
1	2	3	4	5	6
1	5	10	0.5	0.50	193.5
2	10	20	1.4	0.90	348.3
3	15	30	2.8	1.40	541.8
4	20	40	5.3	2.50	967.5
5	25	50	9.5	4.20	1,625.4
6	30	60	15.5	6.00	2,322.0
7	35	70	22.5	7.00	2,709.0
8	40	80	29.5	7.00	2,709.0
9	45	90	37.5	8.00	3,096.0
10	50	100	46.0	8.50	3,289.5
11	55	110	54.5	8.50	3,289.5
12	60	120	62.9	8.40	3,250.8
13	65	130	69.9	7.00	2,709.0
14	70	140	76.0	6.10	2,360.7
15	75	150	80.8	4.80	1,857.6
16	80	160	85.0	4.20	1,625.4
17	85	170	88.0	3.00	1,161.0
18	90	180	90.4	2.40	928.8
19	95	190	92.5	2.10	812.7
20	100	200	94.2	1.70	657.9
21	105	210	95.5	1.30	503.1
22	110	220	96.6	1.10	425.7
23	115	230	97.5	0.90	348.3
24	120	240	98.2	0.70	270.9
25	125	250	98.7	0.50	193.5
26	130	260	99.0	0.30	116.1
27	135	270	99.3	0.30	116.1
28	140	280	99.4	0.10	38.7
29	145	290	99.5	0.10	38.7
30	150	300	99.6	0.10	38.7
31	155	310	99.7	0.10	38.7
32	160	320	99.8	0.10	38.7
33	165	330	99.9	0.10	38.7
34	170	340	100	0.10	38.7
35	175	350	100	0.00	0.0
36	180	360	100	0.00	0.0

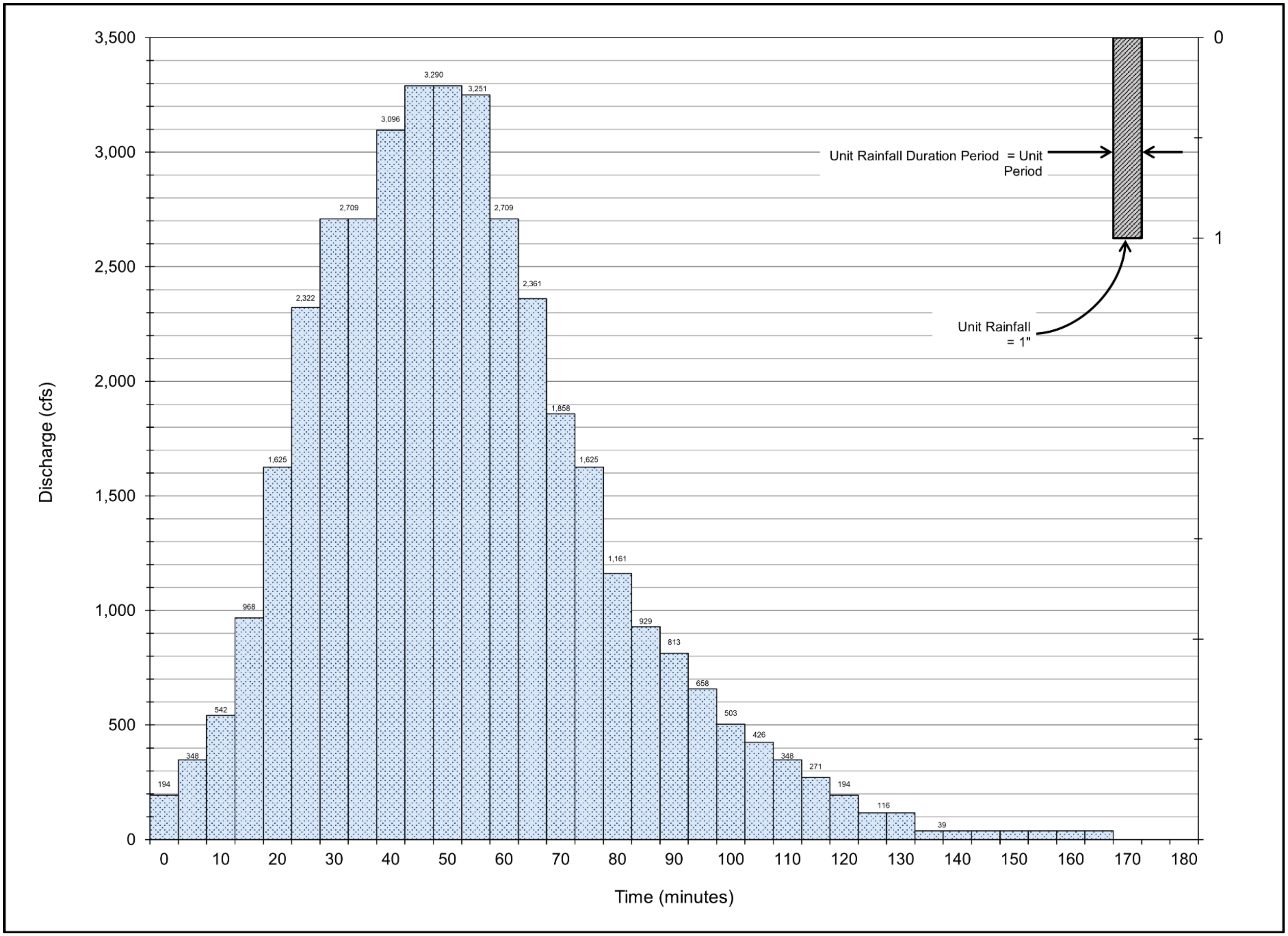


Figure 5-8. Example Unit Distribution Graph (Unit Time Period = 5 minutes)

5.3.2. Effective Design Storm Hyetographs

Effective rainfall is defined as the gross (or unadjusted) precipitation (see **Chapter 2**) minus losses (see **Section 5.3.2.3** and **Chapter 3**). The development of the design storm hyetograph incorporates the temporal distribution of the effective rainfall as described in **Section 5.3.2.1**. Additionally, as discussed in **Section 5.3.2.2**, the spatial variability of rainfall should be evaluated, and if warranted, adjusted using a DARF.

5.3.2.1. Precipitation Time Series Development

Based on the storm event frequency and duration of interest (e.g., 100-year, 24-hour storm), precipitation data are obtained following the methods outlined in **Section 2.5**. To develop the design storm hyetograph, the total precipitation depth needs to be distributed over time to reflect the storm's temporal characteristics.

As described in **Section 2.9**, the County's design storm pattern is based on a modified version of the Natural Resources Conservation Service 24-hour storm pattern. This pattern incorporates adjustments for DARFs. The critical synthetic storm pattern includes maximum precipitation depths for multiple durations up to 24 hour storm events; for example, within a 24-hour storm, the 1-hour, 3-hour, 6-hour, and 12-hour precipitation depths are nested within the 24-hour total, as shown in **Figure 5-9**.

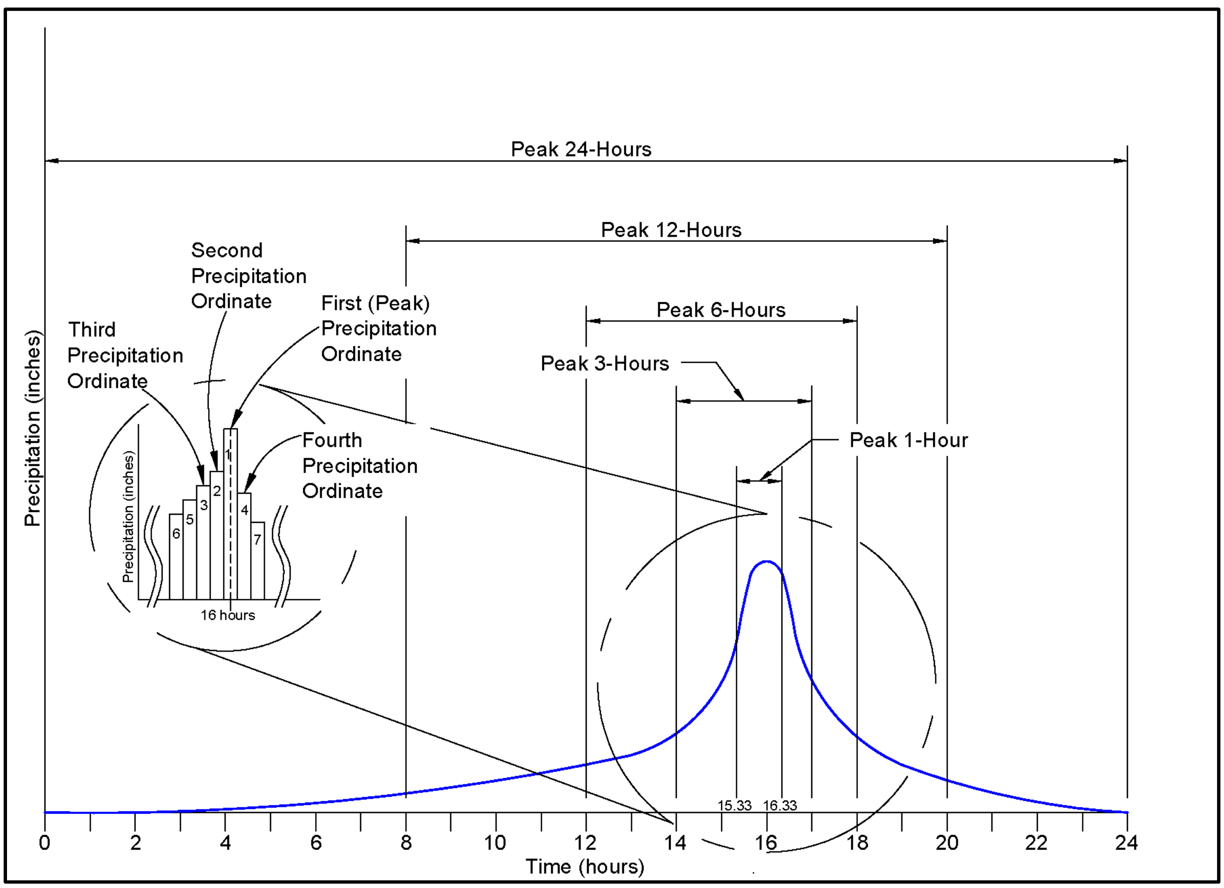


Figure 5-9. 24-Hour Design Storm Peak – Temporal Distribution

All durations are based on the principle of a 2/3, 1/3 placement scheme (resulting in a peak around hour 16 as shown in **Figure 5-9**). Descending rainfall depths are placed in the following repeating sequence: 2 ordinates to the left of the peak, then 1 ordinate to the right of the peak (continuing as, left, left, right, left, left, right, etc.), until the duration of the design storm is covered.

The 1-hour design storm is based on rainfall intensities for durations from 5 minutes to 60 minutes. These rainfall depths could be generated by a Rational Method intensity-duration curve plotted using **Figure 2-4**. Alternatively, National Oceanic and Atmospheric Administration (NOAA) precipitation data provides estimates for the 5-, 10-, 15-, 30-, and 60-minute durations, whereby intermediate rainfall values can be determined using one of the following methods:

- Log-log interpolation techniques performed manually (**Figure 5-10** can be used as an aid).
- The USACE HEC-HMS frequency storm meteorologic model component.

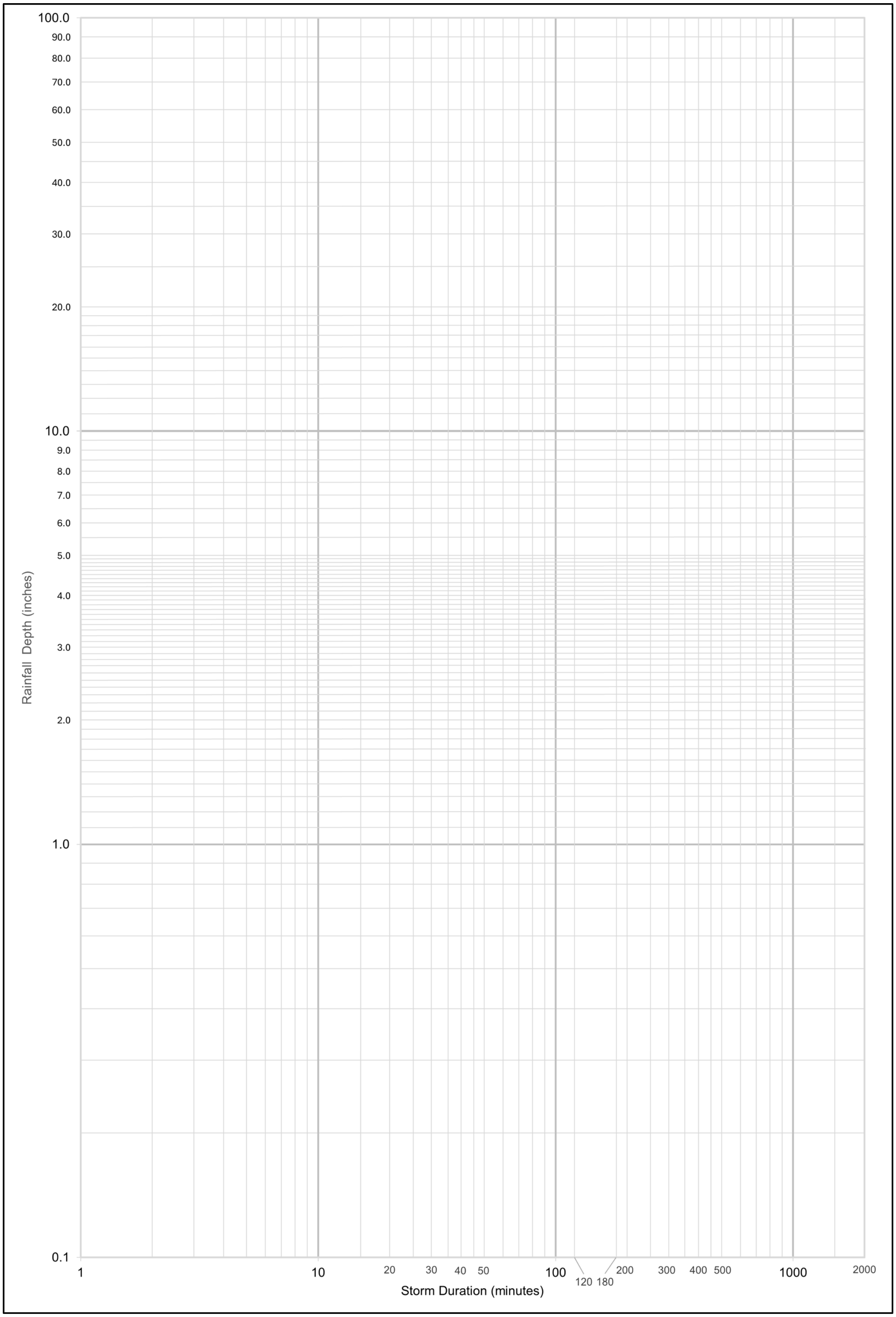


Figure 5-10. Blank Log-Log Interpolation Graph

The 3-hour design storm is constructed in a similar manner as the 1-hour storm, using either Rational Method or NOAA precipitation data for the peak 1-hour portion of the storm, followed by the 120- and 180-minute duration rainfall depths from NOAA data. Intermediate rainfall ordinate depths beyond 60 minutes can be calculated using log-log interpolation techniques applied manually or using software (e.g., USACE HEC-HMS).

The 6-, 12-, and 24-hour design storms are constructed in a similar fashion as the 1-hour and 3-hour storms described above.

5.3.2.2. Depth-Area Reduction Factor (DARF)

For watersheds that exceed 1 sq. mi., precipitation depths must be adjusted using a DARF. The DARF is necessary because precipitation is highly variable in space. A storm with a specific frequency (e.g., 100-year) is unlikely to produce uniform rainfall over a large watershed; rather it is expected to be concentrated over a smaller area, and not necessarily over the entirety of the watershed. The DARF represents that adjustment and corrects the precipitation by reducing the point rainfall depths when it is applied over a larger area.

The DARF also varies by both the duration of the storm and watershed area; consequently, **Figure 5-11** and **Equation 5-4a** through **Equation 5-4f** show the DARF curves and equations, respectively, that should be applied for various precipitation durations along with the watershed area. For storm durations that exceed 24 hours (e.g., 5-day storm), a DARF correction should not be applied, regardless of the watershed size.

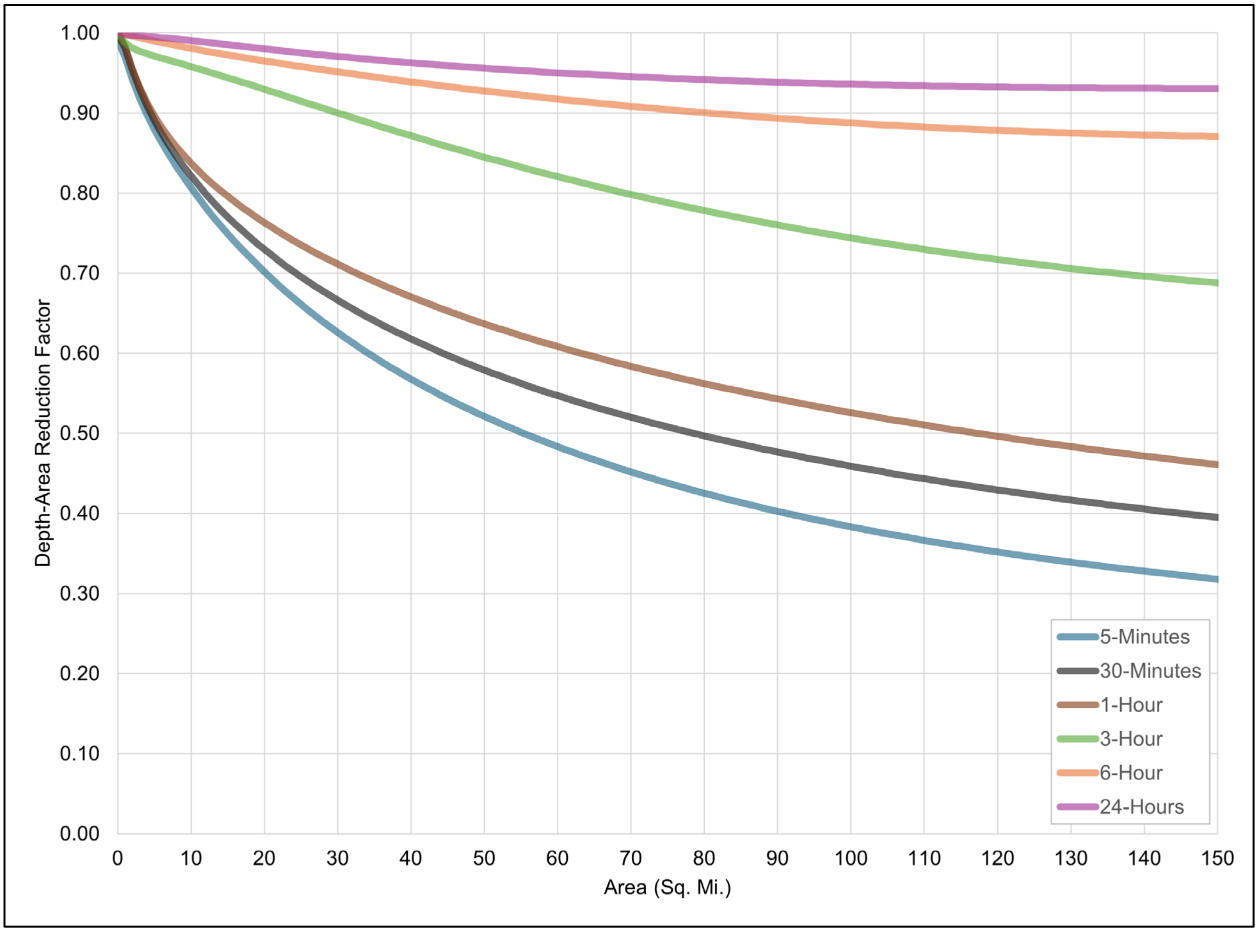


Figure 5-11. Depth-Area Reduction Factor (%)

Equation 5-4 DARF[1] Equation Set

a 5-Minutes

$$D = -0.0001905\alpha^6 + 0.003482\alpha^5 - 0.022455\alpha^4 + 0.0646\alpha^3 - 0.1094\alpha^2 + 0.024\alpha + 0.99$$

$$\alpha = \ln(A+1.01)$$

b 30-Minutes

$$D = -0.0001006\alpha^6 + 0.00194838\alpha^5 - 0.0136345\alpha^4 + 0.0452\alpha^3 - 0.095\alpha^2 + 0.026\alpha + 0.9975$$

$$\alpha = \ln(A+1.01)$$

c 1-Hour

$$D = -0.000085\alpha^6 + 0.00184\alpha^5 - 0.014574\alpha^4 + 0.05382\alpha^3 - 0.1096\alpha^2 + 0.0328\alpha + 0.999$$

$$\alpha = \ln(A+1.01)$$

d 3-Hour

$$D = -0.10629\alpha^5 + 1.92912\alpha^4 - 12.09185\alpha^3 + 34.6926\alpha^2 - 46.9964\alpha + 25.4646$$

$$\alpha = \ln[(A+1.01)^{0.5}+5]$$

e 6-Hour

$$D = 0.811\alpha^6 + 2.907\alpha^5 + 3.97665\alpha^4 + 2.62939\alpha^3 + 0.7387\alpha^2 + 0.06038\alpha + 0.9977$$

$$\alpha = \ln[(A+4.25)^{-0.25}]$$

f 24-Hours

$$D = 51513.09644\alpha^6 - 48749\alpha^5 + 19069\alpha^4 - 3949.8\alpha^3 + 454\alpha^2 - 26.567\alpha + 1.539$$

$$\alpha = \ln[(A+15)^{-0.5} + 1]$$

where:

A = Watershed Area (sq. mi.)
α = Area Transformation
D = Depth-Area Reduction Factor (DARF)

[1]Valid for $0 \leq A \leq 150$ sq. mi.

5.3.2.3. Losses

A detailed discussion of precipitation losses and applicable equations used to calculate effective precipitation in accordance with this Manual's methodologies is presented in **Chapter 3**. Relevant equations and sections from **Chapter 3** will be referenced, where applicable, to aid in the effective design storm hyetograph development methodological description (**Section 5.3.2.4**) and the associated example calculations (**Section 5.3.2.5**).

5.3.2.4. Effective Design Storm Hyetograph Development

The following steps provide general guidelines for developing an effective design storm hyetograph:

1. Identify the storm event frequency (e.g., 100-year) and duration (e.g., 24-hours) for the design storm of interest; then obtain the associated precipitation data following guidance in **Section 2.5.1.1** or **Section 2.5.1.2**.
2. Apply the DARF, presented in **Section 5.3.2.2**, to the precipitation data from Step 1 if applicable based on the watershed area and storm duration.
3. Using the adjusted precipitation (if applicable) from Step 2, or the unadjusted precipitation from Step 1, develop the precipitation time series distribution following the guidance in **Section 5.3.2.1**.
4. Estimate the watershed (or subarea) low loss and maximum loss rates following the guidance presented in **Section 3.6**.
5. Apply the governing loss rates from Step 4 to the precipitation hyetograph developed in Step 3 to compute the effective rainfall hyetograph.

5.3.2.5. Effective Design Storm Hyetograph Example

The example from **Section 5.3.1.5** is continued below to calculate the effective design storm hyetograph.

1. **Identify the storm event frequency (e.g., 100-year) and duration (e.g., 24-hours) for the design storm of interest; then obtain the associated precipitation data following guidance in Section 2.5.1.1 or Section 2.5.1.2.**

   The 100-year storm event frequency and 3-hour storm duration were used for this example. Since the example study watershed is greater than 1 sq. mi., the precipitation data are obtained from NOAA following the guidance in **Section 2.5.1.2** (Areal Average Precipitation); the 5-, 10-, 15-, 30-, 60-, 120-, and 180-minute point precipitation values are shown in Column 2 of **Table 5-3** (note that column references for Step 1 and Step 2 are to **Table 5-3**).

2. **Apply the DARFs, presented in Section 5.3.2.2, to the precipitation data from Step 1 if applicable based on the watershed area and storm duration.**

   The 5-minute, 30-minute, 1-hour, and 3-hour DARF equations are used to calculate the reduction factors for the point precipitation values at the corresponding durations; linear interpolation is used for the intermediate durations. The only watershed parameter needed is the watershed area, which was determined in Step 1 of the synthetic UH example (Area = 5 sq. mi.).

   Example DARF calculation using **Equation 5-4d** (3-hour):

$$\alpha = \ln[(A+1.01)^{0.5}+5] = \ln[(5 \text{ sq. mi.}+1.01)^{0.5}+5] = 2.008$$

$$\alpha = 2.008$$

$$D = -0.10629(2.008)^5 + 1.92912(2.008)^4 - 12.09185(2.008)^3 + 34.6926(2.008)^2$$
$$- 46.9964(2.008) + 25.4646$$

$$D = 0.971$$

   The DARF, D, is 0.971. Multiply the 180-minute point precipitation data (Column 2) by the reduction factor (D = 0.971) to adjust the precipitation value for the watershed area (Column 3).

$$\text{180-minute duration point precipitation (inches)} = 1.630 \text{ inches} \times 0.971 = 1.583 \text{ inches}$$

Table 5-3. Example Effective Design Storm Precipitation Data

Duration (minutes)	**100-year Precipitation Depths** (inches)	
	NOAA[1]	**DARF**
1	2	3
5	0.393	0.346
10	0.564	0.497
15	0.682	0.603
30	0.932	0.830
60	1.100	0.986
120	1.410	1.317
180	1.630	1.583

[1]Data obtained from the NOAA Atlas 14 database for an arbitrary location in San Bernardino County

3. **Using the adjusted precipitation (if applicable) from Step 2, or the unadjusted precipitation from Step 1, develop the precipitation time series distribution following guidance in Section 5.3.2.1.**

   To develop the time series distribution for the effective design storm hyetograph, determine the number of time ordinates that comprise the storm duration using a unit time period of 5 minutes (e.g., 180 minutes / 5 minutes = 36 ordinates). Multiply the UTPO value (Column 1 of **Table 5-5**; note that column references in Step 4 through Step 6 apply to **Table 5-5**) by the unit time period (5 minutes) to create the time series (Column 2). Use log-log interpolation or HEC-HMS to interpolate the precipitation values between the adjusted precipitation values. Perform these calculations to determine the unknown values along the time distribution (Column 3).

4. **Estimate the watershed (or subarea) low loss and maximum loss rates following the guidance presented in Section 3.6.**

   From the delineated study watershed area, determine the curve number (CN), impervious area fraction (a_i), pervious area fraction (a_p), infiltration rate for the pervious area (F_p), and maximum loss rate (F_m) for each land use within the watershed; **Table 5-4** summarizes these data.

Table 5-4. Example Loss Parameters

Development Type K	**Area Fraction**	**Area** (acres)	**HSG**	**CN**	a_i	a_p	**Pervious Infiltration Rate** F_p (inches/hour)	**Maximum Loss Rate** F_m (inches/hour)
Annual Grass (Fair Cover)	0.1	320	D	84	0.00	1.00	0.31	0.310
Residential SF $\frac{1}{4}$ - acre	0.6	1,920	B	75	0.38	0.62	0.47	0.291
Commercial	0.3	960	B	92	0.85	0.15	0.15	0.023

The area-averaged maximum loss rate (F_m) for the watershed is 0.213 (Column 7). For further guidance on how to calculate F_m and area-averaged F_m refer to Step 5 and Step 10 of the **Chapter 4** Rational Method example (**Section 4.4**).

Low loss rates are calculated at each UTPO using **Equation 3-1** through **Equation 3-6**; the calculations shown for **Equation 3-1** through **Equation 3-3** are for the single family residential, 1/4-acre development type.

Equation 3-2 (Total Soil Capacity):

$$S = \frac{1000}{CN} - 10 = \frac{1000}{75} - 10 = 3.333$$

Equation 3-1 (Initial Abstraction):

$$I_a = 0.2S = 0.2(3.33) = 0.667$$

Equation 3-3 (X-hour Storm Runoff Yield Fraction):

note: $P_x = P_3$

$$Y_j = \frac{(P_3 - I_a)^2}{(P_3 - I_a + S)P_3} = \frac{(1.583\text{ inches} - 0.667)^2}{(1.583\text{ inches} - 0.667 + 3.333)1.583\text{ inches}} = 0.125$$

Compute the 3-hour storm runoff yield fraction (Y_j) for the remaining development types and use **Equation 3-4** to determine the watershed runoff yield fraction (Y).

$$Y = (Y_1A_1 + Y_2A_2 + Y_3A_3)/(A_1 + A_2 + A_3)$$

$$Y = \frac{(0.294\text{ x }320\text{ acres}) + (0.125\text{ x }1920\text{ acres}) + (0.550\text{ x }960\text{ acres})}{(320\text{ acres} + 1920\text{ acres} + 960\text{ acres})} = 0.269$$

$$Y = 0.269$$

Equation 3-5 (Runoff Losses):

$$\overline{Y} = 1\text{-}Y = 1 - 0.269 = 0.731$$

$$\overline{Y} = 0.731$$

Use **Equation 3-6** to determine the low loss rate at each time ordinate (Column 6). An example of this calculation is shown for time ordinate 20:

$$F^{*} = \overline{Y} \text{ x } I = 0.731 \text{ x } 0.311 \frac{\text{inches}}{\text{hour}} = 0.227 \frac{\text{inches}}{\text{hour}}$$

$$F^{*} = 0.227 \frac{\text{inches}}{\text{hour}}$$

5. **Apply the governing loss rates from Step 4 to the precipitation hyetograph developed in Step 3 to compute the effective rainfall hyetograph.**

   Compare the low loss rate to the maximum loss rate to determine the governing loss rate (Column 8) at each time ordinate. If the low loss rate is greater than the maximum loss rate, then the maximum loss rate governs. If the low loss rate is less than the maximum loss rate, then the low loss rate governs.

   The effective rainfall depth (Column 10) at each time ordinate is then determined by subtracting the governing loss depth (Column 9) from the incremental rainfall depth (Column 4).

   Lastly, the effective design storm hyetograph is created using the left, left, right placement scheme for the effective rainfall depth values (Column 10) as illustrated in **Figure 5-12**.

Table 5-5. Example Effective Design Storm Hyetograph Results

Unit Time Period Ordinate	Cumulative Time (minutes)	Cumulative Rainfall Depth (inches)	Incremental Rainfall Depth (inches)	Incremental Rainfall Intensity I (inches/hour)	Low Loss Rate F* (inches/hour)	Maximum Loss Rate F_m (inches/hour)	Governing Loss Rate F* or F_m (inches/hour)	Governing Loss Depth (inches)	Effective Rainfall Depth (inches)
1	2	3	4	5	6	7	8	9	10
1	5	0.346	0.346	4.150	3.034	0.213	0.213	0.018	0.328
2	10	0.497	0.152	1.819	1.330	0.213	0.213	0.018	0.134
3	15	0.603	0.105	1.265	0.925	0.213	0.213	0.018	0.087
4	20	0.689	0.086	1.028	0.752	0.213	0.213	0.018	0.068
5	25	0.763	0.075	0.897	0.656	0.213	0.213	0.018	0.057
6	30	0.830	0.067	0.805	0.588	0.213	0.213	0.018	0.049
7	35	0.863	0.032	0.387	0.283	0.213	0.213	0.018	0.014
8	40	0.892	0.029	0.347	0.254	0.213	0.213	0.018	0.011
9	45	0.918	0.026	0.316	0.231	0.213	0.213	0.018	0.008
10	50	0.942	0.024	0.291	0.212	0.213	0.212	0.018	0.006
11	55	0.965	0.022	0.270	0.197	0.213	0.197	0.016	0.006
12	60	0.986	0.021	0.252	0.184	0.213	0.184	0.015	0.006
13	65	1.019	0.034	0.403	0.294	0.213	0.213	0.018	0.016
14	70	1.051	0.032	0.385	0.281	0.213	0.213	0.018	0.014
15	75	1.082	0.031	0.369	0.270	0.213	0.213	0.018	0.013
16	80	1.112	0.030	0.355	0.260	0.213	0.213	0.018	0.012
17	85	1.140	0.029	0.342	0.250	0.213	0.213	0.018	0.011
18	90	1.168	0.028	0.331	0.242	0.213	0.213	0.018	0.010
19	95	1.194	0.027	0.320	0.234	0.213	0.213	0.018	0.009
20	100	1.220	0.026	0.311	0.227	0.213	0.213	0.018	0.008
21	105	1.245	0.025	0.302	0.221	0.213	0.213	0.018	0.007

Unit Time Period Ordinate	**Cumulative Time** (minutes)	**Cumulative Rainfall Depth** (inches)	**Incremental Rainfall Depth** (inches)	**Incremental Rainfall Intensity I** (inches/hour)	**Low Loss Rate** F* (inches/hour)	**Maximum Loss Rate** F_m (inches/hour)	**Governing Loss Rate** F* or F_m (inches/hour)	**Governing Loss** Depth (inches)	**Effective Rainfall** Depth (inches)
1	2	3	4	5	6	7	8	9	10
22	110	1.270	0.024	0.294	0.215	0.213	0.213	0.018	0.006
23	115	1.294	0.024	0.286	0.209	0.213	0.209	0.017	0.007
24	120	1.317	0.023	0.279	0.204	0.213	0.204	0.017	0.006
25	125	1.342	0.025	0.295	0.216	0.213	0.213	0.018	0.007
26	130	1.366	0.024	0.289	0.211	0.213	0.211	0.018	0.006
27	135	1.389	0.024	0.283	0.207	0.213	0.207	0.017	0.007
28	140	1.412	0.023	0.277	0.203	0.213	0.203	0.017	0.006
29	145	1.435	0.023	0.272	0.199	0.213	0.199	0.017	0.006
30	150	1.457	0.022	0.267	0.195	0.213	0.195	0.016	0.006
31	155	1.479	0.022	0.262	0.191	0.213	0.191	0.016	0.006
32	160	1.500	0.021	0.257	0.188	0.213	0.188	0.016	0.005
33	165	1.522	0.021	0.253	0.185	0.213	0.185	0.015	0.006
34	170	1.542	0.021	0.249	0.182	0.213	0.182	0.015	0.006
35	175	1.563	0.020	0.245	0.179	0.213	0.179	0.015	0.005
36	180	1.583	0.020	0.241	0.176	0.213	0.176	0.015	0.005

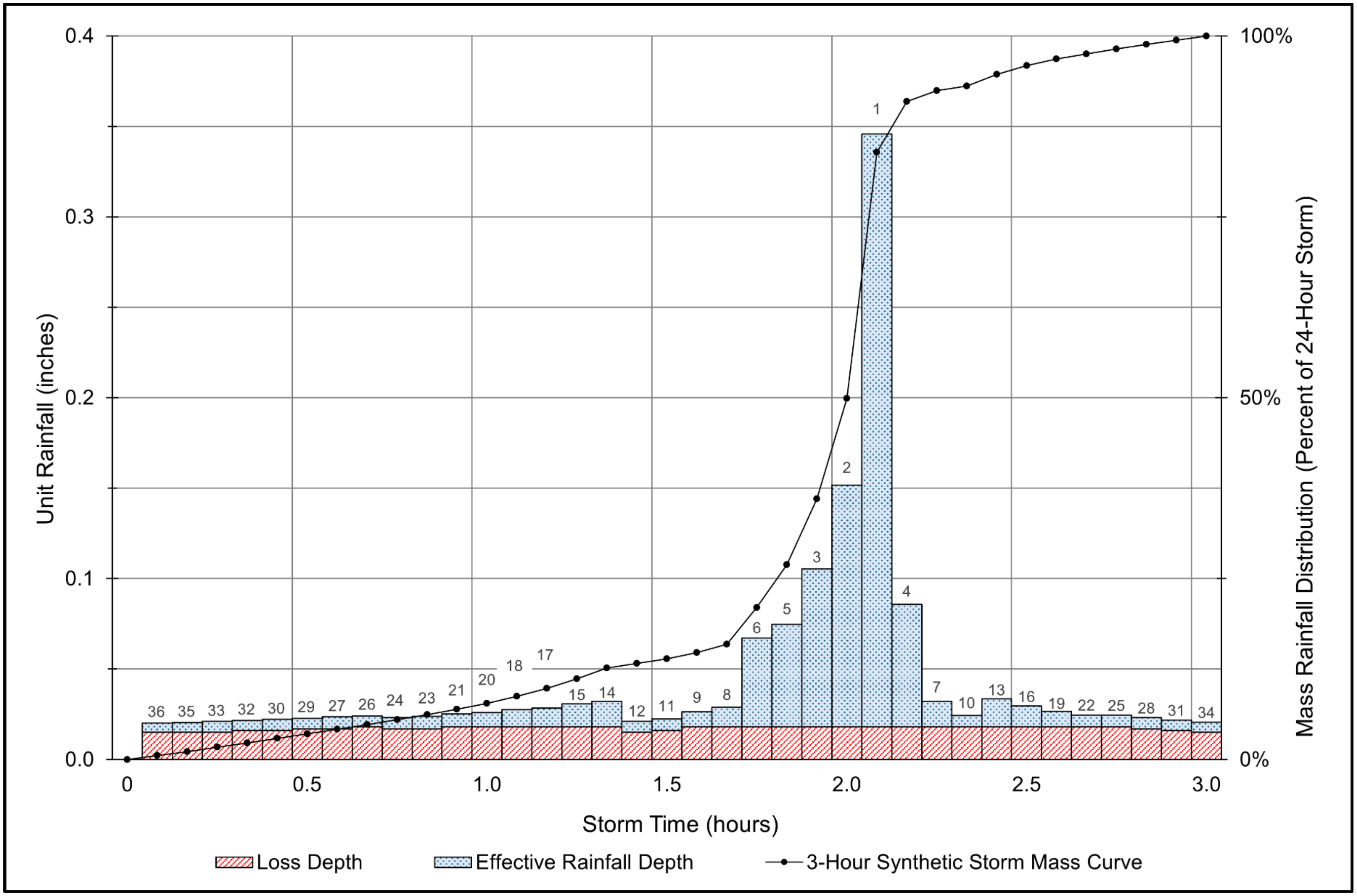


Figure 5-12. Example Effective Design Storm Hyetograph

5.3.3. Runoff Hydrograph Calculations

The runoff hydrograph is computed by convolving a synthetic UH (see **Section 5.3.1**) with an effective rainfall hyetograph (see **Section 5.3.2.4**). The following steps describe the standard procedure for calculating the resulting flow hydrograph:

1. Multiply the effective unit rainfall value at the first UTPO by each ordinate of the synthetic UH to determine the flood hydrograph that would result from that rainfall increment.
2. Repeat Step 1 for each successive effective rainfall value, shifting each resulting hydrograph forward by 1 unit time period to align with the timing of the corresponding rainfall increment.
3. Sum the ordinates of all the shifted hydrographs flow values from Step 1 and Step 2 to obtain the total design storm runoff hydrograph at the point of concentration.
4. If applicable, baseflow (**Section 5.5**) can be added directly to the runoff hydrograph from Step 3.

5.3.3.1. Runoff Hydrograph Example

Use a runoff hydrograph worksheet, synthetic UH data, and effective design storm hyetograph to develop the runoff hydrograph. The runoff hydrograph worksheet (**Figure 5-14** and **Figure 5-15**) contains callouts to the steps outlined in the previous section.

SAN BERNARDINO COUNTY HYDROLOGY MANUAL

STUDY NAME: EXAMPLE PROBLEM

100-YEAR STORM DURATION = ____ HOUR(S)

UNIT TIME PERIOD = ____ MINUTE(S)

Calculated by ____________ Date __________

Checked by ____________ Date __________

Page __ of __

Effective Rainfall Depth (inches)																																						Hydrograph (cfs)
UH (cfs)	UTPO	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	
	1																																					
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Figure 5-13. Blank Unit Hydrograph Worksheet

SAN BERNARDINO COUNTY
HYDROLOGY MANUAL

STUDY NAME: EXAMPLE PROBLEM

100-YEAR STORM DURATION = 3 HOUR(S)

UNIT TIME PERIOD = 5 MINUTE(S)

Calculated by ______________ Date __________

Checked by ______________ Date __________

Page 1 of 2

Effective Rainfall Depth (inches)		0.005	0.005	0.006	0.005	0.006	0.006	0.007	0.006	0.006	0.007	0.007	0.008	0.010	0.011	0.013	0.014	0.006	0.006	0.008	0.011	0.049	0.057	0.087	0.134	0.328	0.068	0.014	0.006	0.016	0.012	0.009	0.006	0.007	0.006	0.006	0.006	Hydrograph (cfs)
UH (cfs)	UTPO	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	
193.5	1	1.0																																				1.0
348.3	2	1.8	1.0																																			2.8
541.8	3	2.8	1.9	1.2																																		5.8
967.5	4	4.9	2.9	2.1	1.1																																	11.0
1,625.4	5	8.3	5.2	3.3	1.9	1.2																																19.9
2,322.0	6	11.8	8.8	5.9	2.9	2.2	1.1																															32.7
2,709.0	7	13.8	12.5	9.9	5.3	3.4	2.0	1.3																														48.1
2,709.0	8	13.8	14.6	14.1	8.8	6.0	3.1	2.3	1.2																													63.9
3,096.0	9	15.8	14.6	16.5	12.6	10.1	5.5	3.6	2.1	1.2																												82.0
3,289.5	10	16.7	16.7	16.5	14.7	14.5	9.2	6.4	3.3	2.2	1.3																											101.5
3,289.5	11	16.7	17.8	18.8	14.7	16.9	13.1	10.7	5.9	3.4	2.4	1.4																										121.8
3,250.8	12	16.5	17.8	20.0	16.9	16.9	15.3	15.3	9.9	6.0	3.7	2.5	1.5																									142.2
2,709.0	13	13.8	17.6	20.0	17.9	19.3	15.3	17.8	14.1	10.1	6.6	3.9	2.7	1.9																								160.9
2,360.7	14	12.0	14.6	19.8	17.9	20.5	17.5	17.8	16.4	14.5	11.1	6.9	4.3	3.3	2.0																							178.7
1,857.6	15	9.5	12.8	16.5	17.7	20.5	18.6	20.3	16.4	16.9	15.8	11.6	7.6	5.2	3.7	2.5																						195.5
1,625.4	16	8.3	10.0	14.4	14.7	20.2	18.6	21.6	18.8	16.9	18.5	16.6	12.8	9.3	5.7	4.4	2.7																					213.6
1,161.0	17	5.9	8.8	11.3	12.9	16.9	18.4	21.6	20.0	19.3	18.5	19.4	18.3	15.6	10.2	6.9	4.9	1.2																				229.8
928.8	18	4.7	6.3	9.9	10.1	14.7	15.3	21.4	20.0	20.5	21.1	19.4	21.4	22.2	17.1	12.4	7.6	2.1	1.2																			247.3
812.7	19	4.1	5.0	7.1	8.8	11.6	13.3	17.8	19.7	20.5	22.4	22.1	21.4	25.9	24.5	20.7	13.6	3.2	2.2	1.6																		265.8
657.9	20	3.3	4.4	5.6	6.3	10.1	10.5	15.5	16.4	20.2	22.4	23.5	24.4	25.9	28.5	29.6	22.9	5.8	3.5	2.9	2.1																	284.2
503.1	21	2.6	3.6	4.9	5.1	7.2	9.2	12.2	14.3	16.9	22.2	23.5	26.0	29.6	28.5	34.6	32.7	9.7	6.2	4.5	3.8	9.5																306.8
425.7	22	2.2	2.7	4.0	4.4	5.8	6.6	10.7	11.3	14.7	18.5	23.2	26.0	31.5	32.6	34.6	38.1	13.9	10.5	8.1	5.9	17.1	11.0															333.3
348.3	23	1.8	2.3	3.1	3.6	5.1	5.2	7.6	9.9	11.6	16.1	19.4	25.7	31.5	34.7	39.5	38.1	16.2	15.0	13.6	10.6	26.6	19.8	16.9														373.6
270.9	24	1.4	1.9	2.6	2.7	4.1	4.6	6.1	7.0	10.1	12.7	16.9	21.4	31.1	34.7	42.0	43.6	16.2	17.5	19.4	17.8	47.5	30.8	30.5	25.9													448.2
193.5	25	1.0	1.5	2.1	2.3	3.1	3.7	5.3	5.6	7.2	11.1	13.3	18.6	25.9	34.3	42.0	46.3	18.5	17.5	22.6	25.4	79.8	54.9	47.4	46.5	63.4												599.4
116.1	26	0.6	1.0	1.6	1.9	2.7	2.8	4.3	4.9	5.8	7.9	11.6	14.7	22.6	28.5	41.5	46.3	19.6	20.0	22.6	29.7	113.9	92.3	84.6	72.4	114.2	13.1											781.2
116.1	27	0.6	0.6	1.2	1.5	2.2	2.4	3.3	4.0	5.1	6.3	8.3	12.8	17.8	24.9	34.6	45.7	19.6	21.2	25.8	29.7	132.9	131.8	142.1	129.3	177.6	23.6	2.8										1007.7
38.7	28	0.2	0.6	0.7	1.1	1.7	2.0	2.8	3.1	4.1	5.5	6.6	9.2	15.6	19.6	30.1	38.1	19.4	21.2	27.4	33.9	132.9	153.8	203.0	217.2	317.2	36.7	5.0	1.2									1309.9
38.7	29	0.2	0.2	0.7	0.6	1.2	1.5	2.3	2.6	3.1	4.5	5.8	7.3	11.1	17.1	23.7	33.2	16.2	21.0	27.4	36.0	151.9	153.8	236.9	310.2	532.9	65.5	7.7	2.2	3.0								1680.0
38.7	30	0.2	0.2	0.2	0.6	0.7	1.1	1.8	2.1	2.7	3.4	4.7	6.4	8.9	12.2	20.7	26.1	14.1	17.5	27.1	36.0	161.4	175.7	236.9	361.9	761.2	110.0	13.8	3.4	5.4	2.2							2019.0
38.7	31	0.2	0.2	0.2	0.2	0.7	0.7	1.3	1.6	2.2	2.9	3.6	5.2	7.8	9.8	14.8	22.9	11.1	15.2	22.6	35.6	161.4	186.7	270.7	361.9	888	157.2	23.2	6.0	8.4	4.0	1.7						2228.2
38.7	32	0.2	0.2	0.2	0.2	0.2	0.7	0.8	1.2	1.7	2.4	3.0	4.0	6.3	8.6	11.9	16.3	9.7	12.0	19.7	29.7	159.5	186.7	287.6	413.7	888	183.4	33.1	10.1	15.0	6.3	3.0	1.3					2316.7
38.7	33	0.2	0.2	0.2	0.2	0.2	0.2	0.8	0.7	1.2	1.8	2.5	3.4	4.8	6.9	10.4	13.1	6.9	10.5	15.5	25.8	132.9	184.5	287.6	439.5	1015	183.4	38.6	14.4	25.3	11.2	4.7	2.3	1.3				2446.4
38.7	34	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.7	0.7	1.3	1.9	2.7	4.1	5.3	8.4	11.4	5.5	7.5	13.6	20.3	115.8	153.8	284.2	439.5	1078	209.6	38.6	16.8	36.1	18.8	8.4	3.5	2.3	1.2			2492.3
0	35	0.0	0.2	0.2	0.2	0.2	0.2	0.3	0.2	0.7	0.8	1.4	2.1	3.3	4.5	6.4	9.3	4.9	6.0	9.7	17.8	91.2	134.0	236.9	434.3	1078	222.7	44.1	16.8	42.1	26.9	14.1	6.3	3.6	2.1	1.1		2423.1
0	36	0.0	0.0	0.2	0.2	0.2	0.2	0.3	0.2	0.2	0.8	0.8	1.5	2.6	3.7	5.4	7.1	3.9	5.2	7.7	12.7	79.8	105.4	206.4	361.9	1066	222.7	46.9	19.3	42.1	31.4	20.2	10.5	6.4	3.3	2.0	1.1	2278.3
	37	[illegible]	0.0	0.0	0.2	0.2	0.2	0.3	0.2	0.2	0.3	0.8	0.9	1.9	2.9	4.4	6.0	3.0	4.2	6.8	10.2	57.0	92.3	162.4	315.4	888	220.0	46.9	20.5	48.1	31.4	23.6	15.0	10.7	5.9	3.2	2.0	1985.2
																						45.6	65.9	142.1	248.2	773.9	183.4	46.3	20.5	51.1	35.9	23.6	17.5	15.3	9.9	5.6	3.1	1722.3
																						39.9	52.7	101.5	217.2	609.0	159.8	38.6	20.2	51.1	38.1	26.9	17.5	17.9	14.2	9.5	5.5	1446.3
																						32.3	46.1	81.2	155.1	532.9	125.7	33.6	16.8	50.5	38.1	28.6	20.0	17.9	16.5	13.5	9.3	1238.7

Step 1: Multiply the effective unit rainfall value at the first unit time period ordinate (UTPO) by each ordinate of the synthetic UH to determine the flood hydrograph that would result from that rainfall increment.

Example Calculation: UT Ordinate 22: 0.005 x 425.7 = 2.1 cfs (2.2 cfs, rounding)

Step 2: 2. Repeat Step 1 for each successive effective rainfall value, shifting each resulting hydrograph forward by one unit time period to align with the timing of the corresponding rainfall increment.

Example Calculation: 0.006 x 193.5 = 1.2 cfs
(1.1 cfs, rounding)

Step 3: 3. Sum the ordinates of all the shifted hydrographs flow values from Step 1 and Step 2 to obtain the total design storm runoff hydrograph at the point of concentration.

Example Calculation:
sum row at UT Ordinate 19 = 265.5 cfs
(265.8 cfs, rounding)

Figure 5-14. Runoff Hydrograph Example Worksheet (Page 1 of 2)

SAN BERNARDINO COUNTY
HYDROLOGY MANUAL

STUDY NAME: EXAMPLE PROBLEM

100-YEAR STORM DURATION = 3 HOUR(S)

UNIT TIME PERIOD = 5 MINUTES(S)

Calculated by ______________ Date ___________

Checked by ______________ Date ___________

Page 2 of 2

Effective Rainfall Depth (inches)		0.005	0.005	0.006	0.005	0.006	0.006	0.007	0.006	0.006	0.007	0.007	0.008	0.010	0.011	0.013	0.014	0.006	0.006	0.008	0.011	0.049	0.057	0.087	0.134	0.328	0.068	0.014	0.006	0.016	0.012	0.009	0.006	0.007	0.006	0.006	0.006	Hydrograph (cfs)
UH (cfs)	UTPO	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	
	41						0.0	0.0	0.2	0.2	0.3	0.3	0.3	0.4	0.4	1.5	1.6	1.2	1.7	2.9	4.7	24.7	37.3	71.1	124.1	380.6	110.0	26.5	14.7	42.1	37.7	28.6	21.3	20.4	16.5	15.8	13.3	1000.4
	42							0.0	0.0	0.2	0.3	0.3	0.3	0.4	0.4	0.5	1.6	0.7	1.2	2.3	3.8	20.9	28.6	57.5	108.6	304.5	78.6	23.2	11.6	36.7	31.4	28.3	21.3	21.7	18.9	15.8	15.5	834.9
	43								0.0	0.0	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.7	0.7	1.6	3.0	17.1	24.2	44.0	87.9	266.4	62.9	16.5	10.1	28.9	27.4	23.6	21.0	21.7	20.1	18.0	15.5	713.9
	44									0.0	0.0	0.3	0.3	0.4	0.4	0.5	0.5	0.2	0.7	1.0	2.1	13.3	19.8	37.2	67.2	215.7	55.0	13.2	7.2	25.3	21.5	20.5	17.5	21.5	20.1	19.2	17.8	598.4
	45										0.0	0.0	0.3	0.4	0.4	0.5	0.5	0.2	0.2	1.0	1.3	9.5	15.4	30.5	56.9	164.9	44.5	11.6	5.8	18.0	18.8	16.2	15.3	17.9	19.8	19.2	18.9	487.9
	46											0.0	0.0	0.4	0.4	0.5	0.5	0.2	0.2	0.3	1.3	5.7	11.0	23.7	46.5	139.6	34.1	9.4	5.1	14.4	13.5	14.1	12.0	15.6	16.5	18.9	18.9	402.8
	47												0.0	0.0	0.4	0.5	0.5	0.2	0.2	0.3	0.4	5.7	6.6	16.9	36.2	114.2	28.8	7.2	4.1	12.6	10.8	10.1	10.5	12.3	14.4	15.8	18.6	327.4
	48													0.0	0.0	0.5	0.5	0.2	0.2	0.3	0.4	1.9	6.6	10.2	25.9	88.8	23.6	6.1	3.1	10.2	9.4	8.1	7.5	10.7	11.3	13.8	15.5	254.9
	49														0.0	0.0	0.5	0.2	0.2	0.3	0.4	1.9	2.2	10.2	15.5	63.4	18.3	5.0	2.6	7.8	7.6	7.1	6.0	7.7	9.9	10.8	13.5	191.4
	50															0.0	0.0	0.2	0.2	0.3	0.4	1.9	2.2	3.4	15.5	38.1	13.1	3.9	2.2	6.6	5.8	5.7	5.3	6.1	7.1	9.5	10.7	138.2
	51																0.0	0.0	0.2	0.3	0.4	1.9	2.2	3.4	5.2	38.1	7.9	2.8	1.7	5.4	4.9	4.4	4.3	5.4	5.7	6.8	9.3	110.1
	52																	0.0	0.0	0.3	0.4	1.9	2.2	3.4	5.2	12.7	7.9	1.7	1.2	4.2	4.0	3.7	3.3	4.3	5.0	5.4	6.7	73.4
	53																		0.0	0.0	0.4	1.9	2.2	3.4	5.2	12.7	2.6	1.7	0.7	3.0	3.1	3.0	2.8	3.3	4.0	4.7	5.3	60.1
	54																			0.0	0.0	1.9	2.2	3.4	5.2	12.7	2.6	0.6	0.7	1.8	2.2	2.4	2.3	2.8	3.1	3.8	4.7	52.3
	55																				0.0	0.0	2.2	3.4	5.2	12.7	2.6	0.6	0.2	1.8	1.3	1.7	1.8	2.3	2.6	2.9	3.8	45.0
	56																					0.0	0.0	3.4	5.2	12.7	2.6	0.6	0.2	0.6	1.3	1.0	1.3	1.8	2.1	2.5	2.9	38.1
	57																						0.0	0.0	5.2	12.7	2.6	0.6	0.2	0.6	0.4	1.0	0.8	1.3	1.7	2.0	2.4	31.5
	58																							0.0	0.0	12.7	2.6	0.6	0.2	0.6	0.4	0.3	0.8	0.8	1.2	1.6	2.0	23.8
	59																								0.0	0.0	2.6	0.6	0.2	0.6	0.4	0.3	0.3	0.8	0.7	1.1	1.6	9.2
	60																									0.0	0.0	0.6	0.2	0.6	0.4	0.3	0.3	0.3	0.7	0.7	1.1	5.2
	61																										0.0	0.0	0.2	0.6	0.4	0.3	0.3	0.3	0.2	0.7	0.7	3.7
	62																											0.0	0.0	0.6	0.4	0.3	0.3	0.3	0.2	0.2	0.7	3.0
	63																												0.0	0.0	0.4	0.3	0.3	0.3	0.2	0.2	0.2	2.0
	64																													0.0	0.0	0.3	0.3	0.3	0.2	0.2	0.2	1.5
	65																														0.0	0.0	0.3	0.3	0.2	0.2	0.2	1.2
	66																															0.0	0.0	0.3	0.2	0.2	0.2	0.9
	67																																0.0	0.0	0.2	0.2	0.2	0.7
	68																																	0.0	0.0	0.2	0.2	0.4
	69																																		0.0	0.0	0.2	0.2
	70																																			0.0	0.0	0.0
	71																																				0.0	0.0

Figure 5-15. Runoff Hydrograph Example Worksheet (Page 2 of 2)

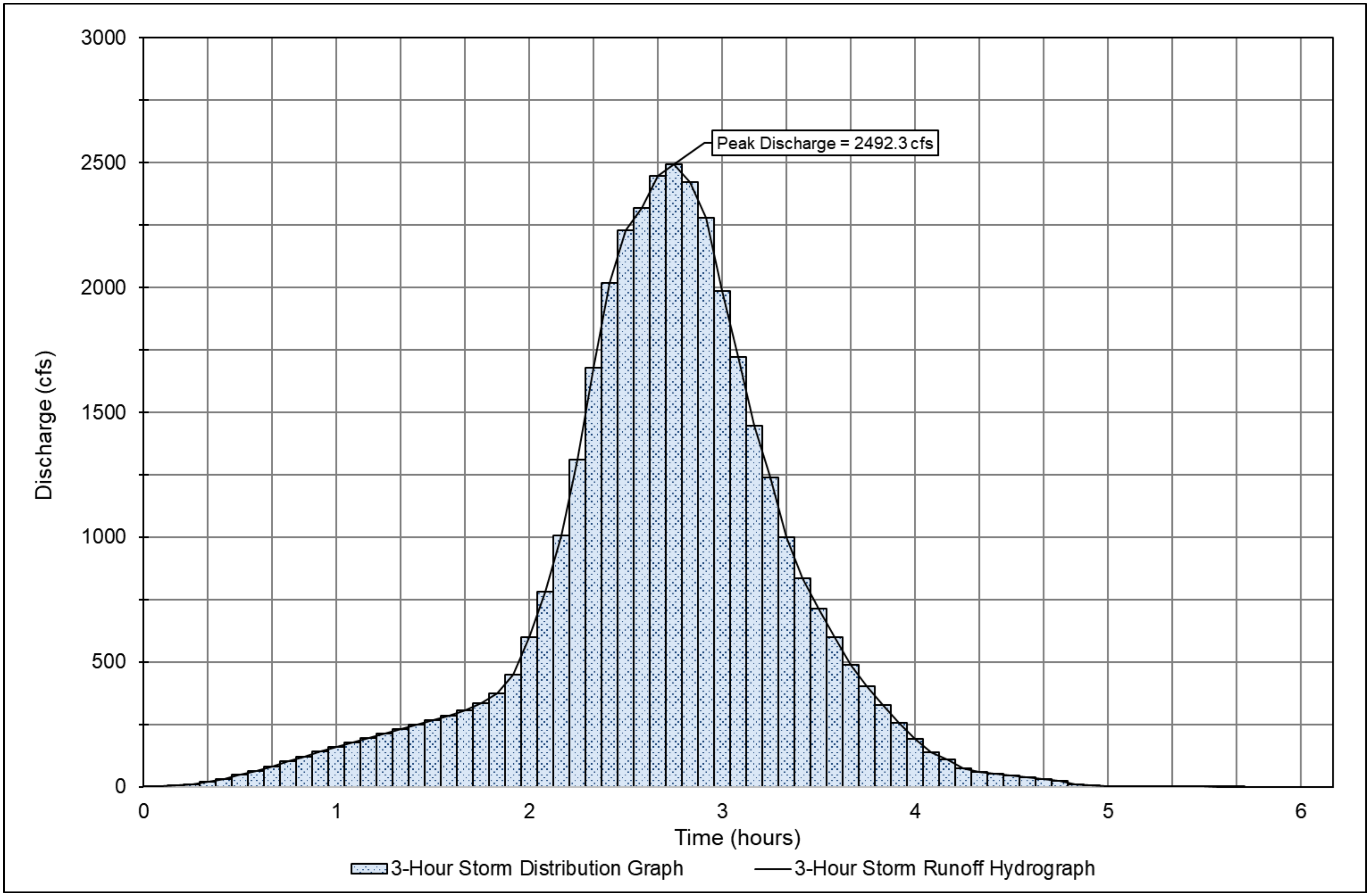

Figure 5-16. Runoff Hydrograph Example Curve

5.4. Baseflow

Baseflow is typically a negligible factor for developing design storm hydrographs in San Bernardino County. However, in certain cases—such as unlined channels draining mountainous areas with permeable geologic formations—a nominal baseflow (e.g., 10 cfs per sq. mi.) may be appropriate. If supporting data are available, baseflow may be incorporated by adding it directly to the ordinates of the computed runoff hydrograph. In fully urbanized watersheds, baseflow is generally considered negligible and may be omitted.

5.5. Hydrologic Routing

In complex or heterogeneous watersheds, it is often necessary to subdivide the area into hydrologic subareas. These subareas are connected by hydraulic reaches between points of concentration, allowing runoff to be routed downstream to the watershed outlet. This routing process is similar to the travel time component used in time of concentration calculations (see **Section 4.1.2**); however, in natural watersheds, terrain variability—such as cross-sectional geometry, longitudinal slope, and surface roughness—often necessitates more sophisticated hydrologic routing procedures when using synthetic UH methods.

Several hydrologic routing techniques are available. Commonly used methods include Modified Puls, Kinematic Wave, and Muskingum-Cunge, each briefly described below:

> **Modified Puls:** This method is a form of level pool or storage routing, similar to the approach used for detention basin analysis. It can account for backwater effects through a defined stage-storage relationship. For example, a stage-storage rating curve developed in HEC-RAS can be entered into HEC-HMS to implement this routing scheme.
>
> **Kinematic Wave**: This method approximates the momentum equations under steady-state assumptions, where the energy grade line is assumed to equal the channel slope. It is best suited for relatively steep reaches (e.g., slopes of 10 feet per mile or greater) and does not account for backwater effects or storage attenuation.
>
> **Muskingum-Cunge:** This method approximates the momentum equations by including the pressure gradient (hydraulic grade line) term while neglecting inertial (acceleration) terms. It is generally better suited for flatter channels (e.g., slopes of 2 feet per mile or less) and can simulate attenuation and limited backwater effects.

Both the Kinematic Wave and Muskingum-Cunge methods are suitable for ungauged watersheds because they are physically based. The primary parameter requiring estimation is the Manning's roughness coefficient. Additionally, both methods support circular conduit geometry, making them suitable for pipe flow routing applications.

Each hydrologic routing method has strengths and limitations, depending on the watershed characteristics and modeling objectives. For detailed guidance on model setup and method selection, users should refer to the most current HEC-HMS User's Manual and Technical Reference Manual. Regardless of the method used, the selected routing approach must be technically justified and approved by the County as part of any hydrologic study.

CHAPTER 6
POST-FIRE HYDROLOGIC PARAMETER ADJUSTMENTS

6.1. Introduction

Wildfires frequently consume a watershed's vegetative canopy, leaf litter, and duff, thereby reducing interception, storage, and evapotranspiration losses during subsequent rainstorms. Burned soils often become hydrophobic, limiting infiltration (DeBano 1981, Cannon et al. 2004, U.S. Department of Agriculture (USDA) 2016). These processes temporarily produce an increased clear-water runoff volume relative to the unburned watershed condition for a given storm (USDA 2016).

The loss of vegetation also reduces the surface roughness that impedes flow, resulting in decreased infiltration, higher runoff velocities, and reduced times of concentration. Together, these common post-fire conditions often produce larger clear-water design flows at watershed outlets; for example, a 10-year storm falling on a recently burned watershed might produce the 100- or even 200-year flood event.

Fire combusts soil-binding organic matter, and this, coupled with the larger, faster overland flows, frequently results in accelerated stripping of sediment from hillslopes. Wildfire also increases the supply of woody debris to streams. While this chapter addresses the effects of fire on clear-water hydrology, **Chapter 7** provides guidance for assessing increases in sediment/debris loading for post-fire conditions.

After a wildfire, watersheds begin a process of hydrologic recovery that can last from a period of a few years to many decades (Livingston et al. 2005, Wagenbrenner et al. 2021). Typically, the greatest impacts on hydrologic functions are observed in the first years after a fire, then decline at varying rates (Wagenbrenner et al. 2021). The period required for a watershed to reach a new, stable condition varies as a function of many independent variables (e.g., the fraction of the watershed that burned, whether the fire is followed by a relatively wet or dry period, etc.), with burn severity typically considered to be a key control (Livingston et al. 2005, Wagenbrenner et al. 2021). Areas with low burn severity often recover within just a few years and do not develop sufficient hydrophobicity to see substantial changes in their infiltration characteristics, while many severely burned areas require 15 to 20-plus years to recover.

This chapter discusses methodologies for adjusting hydrologic parameters—specifically the curve number (CN), time of concentration, and lag time—following a wildfire, in accordance with County-prescribed guidelines. These adjustments are essential for estimating post-fire clear-water runoff in undeveloped watersheds.

6.1.1. Curve Number (CN) Adjustment

A systematic study by Livingston et al. (2005), "Los Alamos Post-Fire Watershed Recovery: A Curve-Number-Based Evaluation," provides guidance for CN adjustments during post-fire hydrologic recovery periods and is commonly used in California by post-fire experts. This paper is the source of the methods presented in this chapter.

The proposed method relates burn severity and pre-fire CN to post-fire CN, as well as the theoretical change in the CN ratio over an “ideal” hydrologic recovery period. For most subbasins experiencing moderate to high burn severity, the authors estimated recovery periods of 6 to 10 years, whereas the most severely burned subbasins were projected to require 10 to 20 years to return to pre-fire hydrologic conditions.

The study began by characterizing pre-fire and anticipated post-recovery conditions of the forested subbasins using the framework provided in SCS Technical Release 55 (USDA 1986), the predecessor to USDA (2004). Notably, footnotes for Table 2-2e in TR-55 (see Table 9-1 in USDA, 2004) differentiate between “poor” and “good” hydrologic conditions for the “woods” land cover type, based in part on whether the area is subject to “regular burning” or is “protected”. The authors suggest that pre-fire and recovered CN values should correspond to the associated fire and forest management patterns, such that the pre-fire CN for an overgrown forested watershed should reflect the “good” hydrologic condition, and the future recovered CN should reflect the “poor” hydrologic condition if current forest practices include regular burning or “good” if pre-fire watershed management practices will continue.

The authors assumed antecedent moisture condition (AMC) II for both pre- and post-fire CN estimation.

Additionally, Livingston et al. introduced a classification factor termed the “Wildfire Hydrologic Impact” (WHI), defined as a function of the percentage of a subbasin that burned at moderate or high severity. Users are instructed to begin the post-fire CN estimation process by determining the WHI for their watershed, based on **Figure 6-1**.

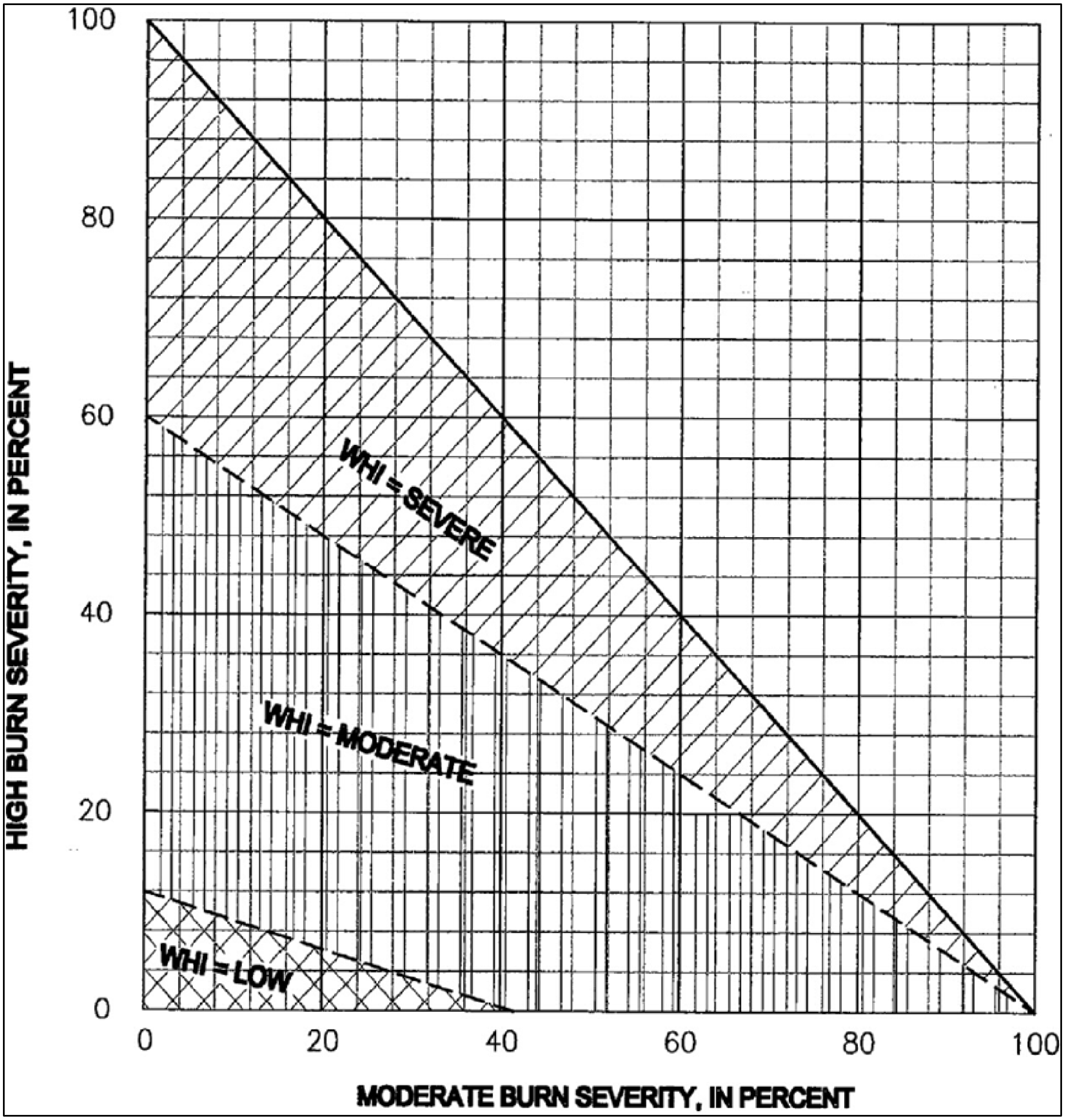

Figure 6-1. WHI for Small Burned Subbasins as a Function of Burn Severity (Livingston et al. 2005)

Livingston et al. acknowledge that the abrupt changes from one classification to the next are not realistic, but note there is insufficient information with which to develop a transition region. As such, they suggest that in cases where a subbasin's burn severity data place it near a dashed line, users might complete the CN adjustment analysis for both WHI classes and either average the results or select one based on engineering or management considerations.

The authors then propose a set of curves (one for each WHI classification) that relate a subbasin's pre-fire CN to an initial (immediately following the fire) ratio of the pre-fire CN to the post-fire CN, developed based on the study watersheds. The curves are shown in **Figure 6-2**. The authors note that the generalized relationships are applicable to the Los Alamos area and other areas in the American Southwest with similar pre-fire CN values and pre-fire rainfall-runoff characteristics.

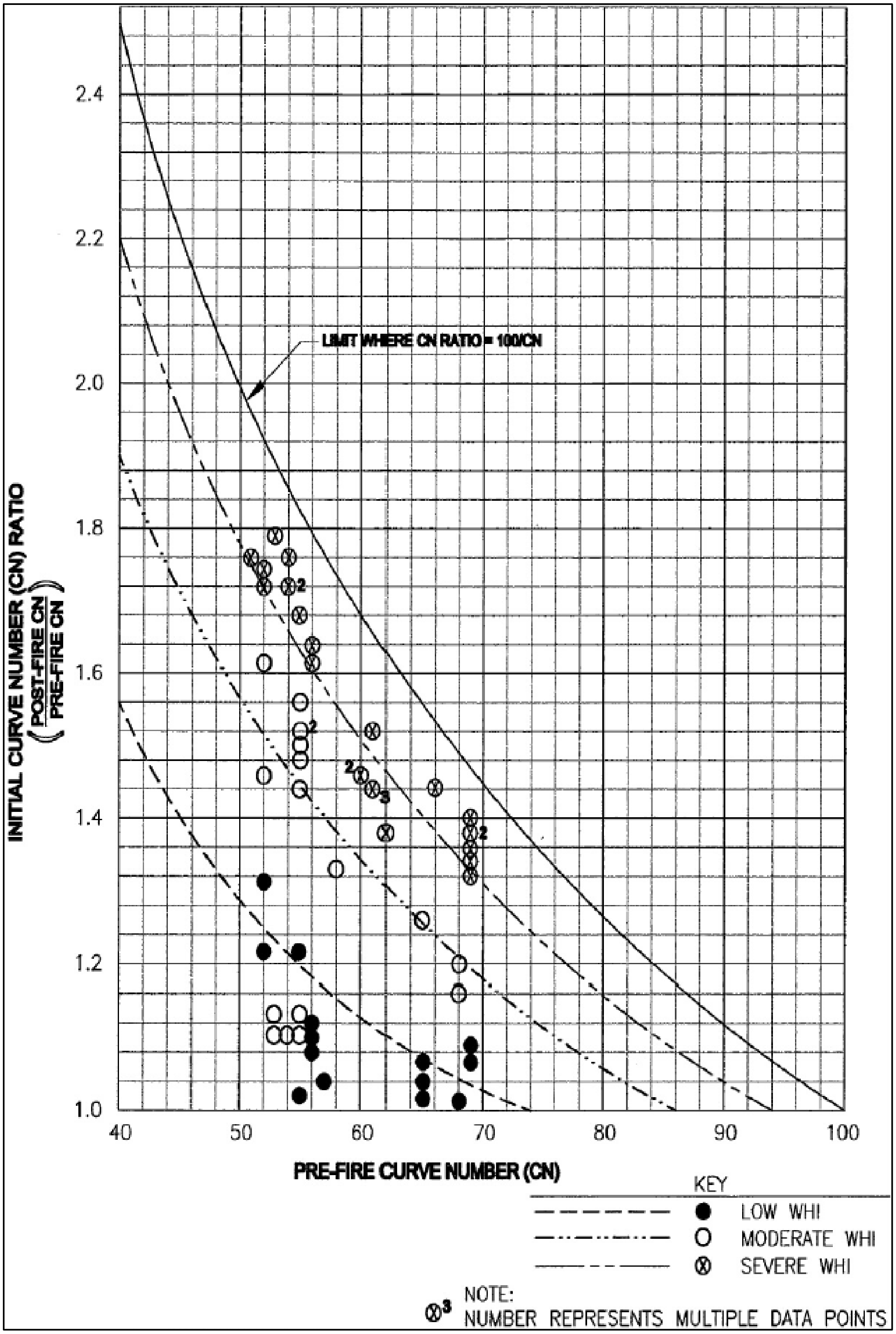

Figure 6-2. General Relation Between Pre- and Initial Post-Fire CN Ratio for Indicated WHI (Livingston et al. 2005)

Finally, the authors propose a theoretical model for CN ratio changes throughout the hydrologic recovery period, based on **Figure 6-3**.

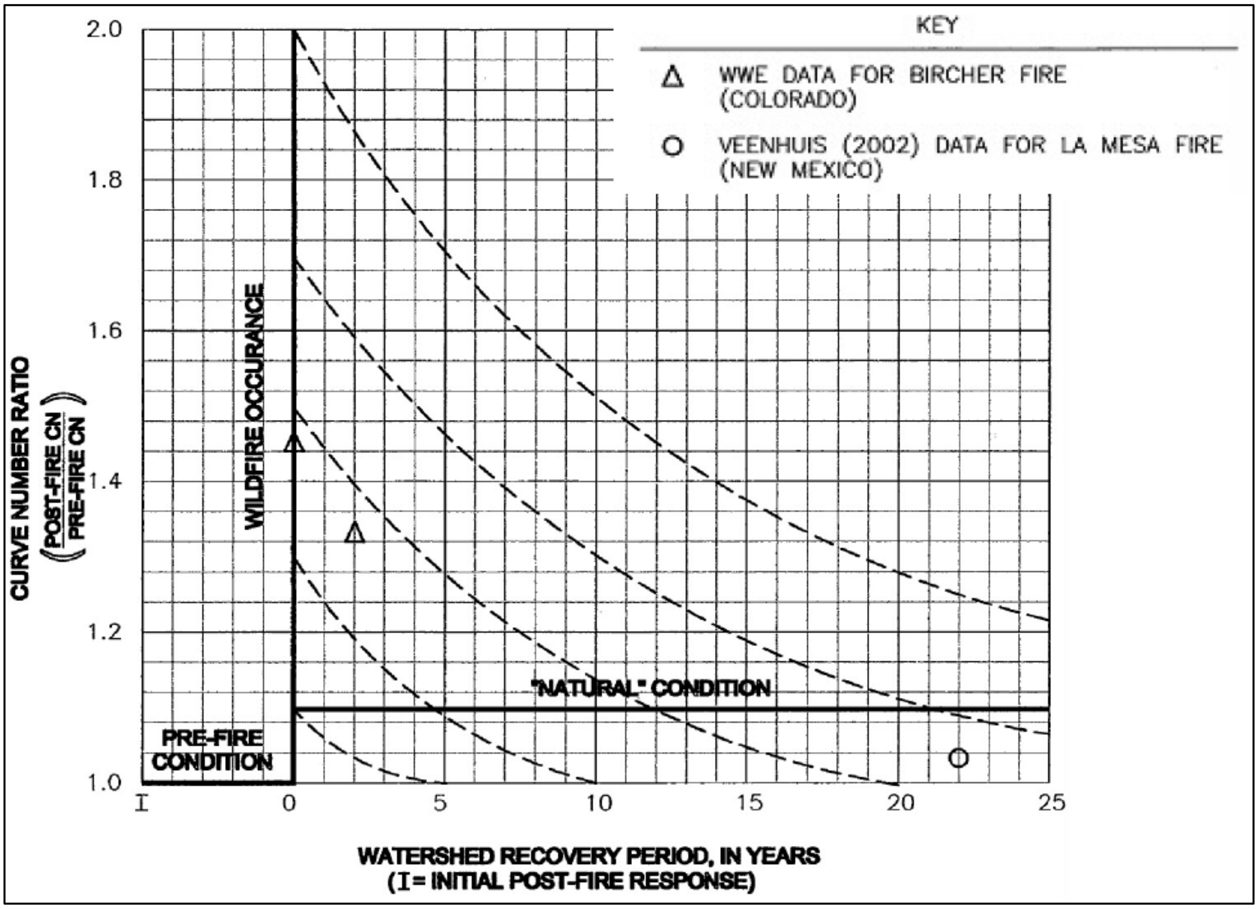

Figure 6-3. Conceptual Change in CN Ratios during the Post-Fire Watershed Recovery Period (Livingston et al. 2005)

The plot illustrates a conceptual “ideal” recovery period, and the authors note that the rate of hydrologic recovery for any particular subbasin depends on many factors, including the extent of post-fire rehabilitation (e.g., reseeding), weather conditions, and land use (e.g., logging or road building) during the recovery period.

The Livingston et al. (2005) model is a post-fire CN adjustment method with historical precedence in California that can be applied using simple methods.

This chapter provides CN adjustment methods for two scenarios:

Representative Fire-prone Conditions: Some watersheds in the southwest portion of San Bernardino County burn frequently enough, and with sufficient severity, such that their average, representative condition is one of a watershed partially recovered from a fire. A hydrologic assessment based only on the current, observed conditions could underestimate the impact of fire on the basin's typical hydrology. **Section 6.2** identifies areas requiring special consideration and prescribes methods for adjusting the CN when some or all of a watershed is located within these areas.

20 Years or Less Post-Fire: While the greatest impacts of fire on watershed hydrology usually diminish within a few years following a fire, post-fire runoff may remain elevated for decades, relative to pre-fire conditions. **Section 6.3** prescribes methods for adjusting the CN in watersheds which partially or fully burned within the past 20 years.

When performing a hydrologic assessment, users must determine whether some or all the study watershed burned during the past 20 years, and, if so, the date of the most recent fire in each project subarea. Additionally, if a hydrologic analysis is performed for a watershed that includes subareas requiring special analysis under representative conditions and subareas which burned in the past 20 years, the hydrologic analysis should be performed twice: once for the representative conditions, described in **Section 6.2**, and once for the current, post-fire conditions, following the guidance in **Section 6.3**. Then, the larger of the resulting peak flows should be used for design.

6.1.2. Time of Concentration and Lag Adjustment

As described in **Chapter 5**, the USACE Lag method computes lag time for large, undeveloped watersheds based on the average basin roughness factor ($\bar{n}$), which reflects the roughness of all collection streams and watershed channels. When CN adjustments are made to represent a hypothetical or unobservable burn condition, corresponding adjustments to the basin roughness factor should also be considered.

To evaluate potential changes in basin roughness, users should begin with a visual estimate of the current $\bar{n}$ as a baseline. They should then compare the actual time since the last fire to the subarea's average fire-return interval and estimated recovery period. Based on this comparison, users may determine if and how roughness values would change under the theoretical burn condition. In general, roughness adjustments many years after a fire are expected to be minor relative to those made immediately following a burn event.

When CN is adjusted to reflect current post-fire conditions, a potential revision of the basin roughness factor ($\bar{n}$) is also warranted. Post-fire roughness conditions vary across a watershed. Burned vegetation typically results in reduced overland flow roughness, often approximated using bare soil values. In contrast, stream channels, especially smaller collectors, may experience increased roughness due to sediment and debris accumulation. While larger channels may recover relatively quickly, smaller ones can take years to return to pre-fire conditions (USDA 2016).

Given this spatial and temporal variability, the standard basin factor tables in **Chapter 5** may not be directly applicable to post-fire conditions. Instead, visual assessment and engineering judgment should be used to estimate the average Manning's n value for all channels in the affected basin. This process should rely on established references, and all assumptions must be clearly documented. Due to the complexity of post-fire hydrologic response, roughness estimation should, whenever possible, be performed by personnel with experience in post-fire hydrology.

6.1.3. Emergency Post-Fire Hydrologic Assessments

A third scenario that may require adjustment to the hydrologic assessments, the emergency post-fire period, is not included in this Manual. These assessments are often required immediately following wildfire events. As discussed previously, wildfires can dramatically alter flood-risk conditions. A watershed with a relatively low annual probability of burning—one that may not require hydrologic adjustments under representative conditions based on the methods in **Section 6.2**—can still pose temporarily elevated risks to life, property, and critical infrastructure following a fire.

The most significant hydrologic impacts typically occur immediately after the fire. During this period, it may be necessary to rapidly assess extreme short-term flood hazards or support the design of temporary infrastructure and mitigation measures. These assessments often rely on simplified methods that can be implemented quickly with limited input data and depend heavily on engineering judgment.

Due to the wide variability in post-fire conditions, data availability, and the rapidly evolving nature of research and best practices, users are advised to coordinate with the County and/or the California Geological Survey's Burned Watershed Geohazards Program when conducting emergency hydrologic evaluations during an identified emergency period.

6.1.4. Consideration of Post-Fire Scenarios in Regional Facility Design

Post-fire scenarios will not be incorporated into the design of the regional ultimate facility. Regional facilities are developed in alignment with the County's aim to manage and convey flows from a 100-year storm event (a storm with a 1% annual probability of occurrence), in accordance with the County's objective to protect public safety, property, and infrastructure. The design flow for the regional facility will be based on the precipitation outlined in **Chapter 2**, as well as sediment bulking factors (if applicable) described in **Chapter 7**.

Based on past wildfire events, the County has found that the existing post-fire response and mitigation methods have been effective in reducing risks to life and property. Furthermore, the hydrologic impacts of wildfire tend to diminish over time and are generally short in duration relative to the expected service life of the regional facility. Including post-fire hydrological conditions in the ultimate design would significantly increase project costs and reduce the overall benefit-cost ratio, making such an approach economically impractical.

Therefore, while these scenarios are not considered applicable for the design of the regional ultimate facility, they may be evaluated on a case-by-case basis for site-specific or smaller-scale projects where appropriate.

6.2. Adjusting the Curve Number (CN) for Representative Fire-Prone Conditions

6.2.1. Areas Affected by Fire under Representative Conditions

Figure 6-4 identifies areas requiring special consideration of the effects of fire under representative conditions. When a hydrologic analysis includes one or more undeveloped subareas falling within these target area boundaries, an additional analysis must be performed for each such subarea to identify whether loss and basin roughness parameters should be adjusted to reflect the hypothetical representative burn conditions.

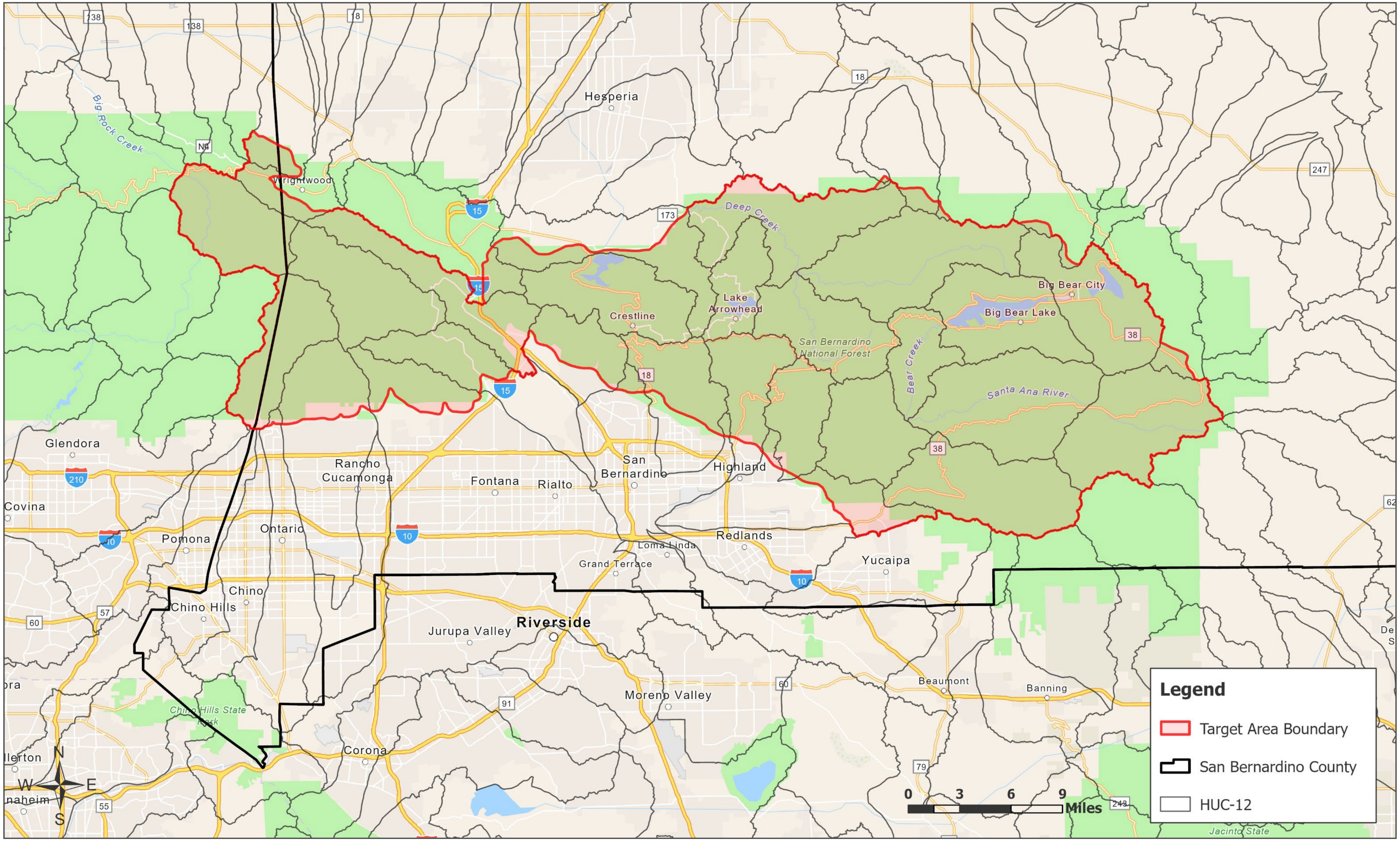

Figure 6-4. Watershed Areas Requiring Special Analysis under Representative Burn Conditions (Overall Map)

6.2.2. Identification of Characteristic Watershed Burn Condition Parameters

When one or more undeveloped subareas fall within the areas requiring special analysis for the effects of fire, users must evaluate three representative parameter values for each such subarea using GIS analysis. Note that subareas should be selected to coincide with areas of similar hydrologic characteristics, including burn characteristics. The three parameters are:

Percentage of Area Burned with Moderate Severity: This is the percentage of the subarea's total area characterized by the moderate burn severity class, based on a 50th percentile synthetic burn severity layer developed by Kean and Staley (2021). The clipped raster dataset ("*SBC_SimSeverity_P50_Clipped.tif*") is available at the San Bernardino County Flood Control webpage.

Percentage of Area Burned with High Severity: This is the percentage of the subarea's total area characterized by the high burn severity class, based on the same 50th percentile synthetic burn severity layer developed by Kean and Staley (2021).

Average Annual Burn Probability: The average annual burn probability is used to determine the fire interval and the theoretical average time since the most recent fire, where the latter is taken as half the fire interval. The average annual burn probability should be computed as an area-weighted average for each subarea using the raster dataset developed by Pyrologix LLC (2023), which is available for download at https://rrk.sdsc.edu/socal.html. The webpage includes a list of data products available for download; the relevant raw data layer, titled "Annual Burn Probability," should be used, and can be found in the *Severity* subsection of the *Fire Dynamics* section.

6.2.3. CN Adjustment for Representative Burned Watershed Conditions

A standard spreadsheet tool must be used to identify an appropriate adjusted CN for a given subarea if an adjustment is warranted (Livingston et al. 2005). The worksheet containing the tool, titled "*CN Adjust – Representative Cond*," is included in an Excel workbook titled "*CN Adjustment Tool*," which is available for download at the San Bernardino County Flood Control webpage.

Using the Excel worksheet, after entering basic identifying project reference information, the following steps should be followed:

Step 1: Determine WHI classification by inputting the computed moderate and high burn severity percentages for the given subarea in the orange-shaded cells. A red point is added to the plot, and the corresponding WHI classification is identified in the red text. When the point is located near or on a dashed line separating two WHI classes, users should complete the CN adjustment analysis for both WHI classes and either average the results or select one based on engineering or management considerations.

Step 2: Determine the Initial CN Ratio by inputting the pre-fire (unburned) CN value, selected based on guidance in Chapter 3. The pre-fire CN value is plotted on the appropriate WHI curve, and a red callout label notes the computed ratio between the CN before and immediately following a hypothetical fire.

Step 3: Review the Subarea's Estimated Recovery Period, a yellow-dashed recovery curve is plotted starting at the initial CN ratio at Year 0, then decreases following a characteristic trajectory until it crosses the x-axis. The subarea's representative watershed recovery period is rounded to the nearest year and labeled in red text.

Step 4: Estimate the Adjusted CN by inputting the average annual burn probability for the subarea in the orange-shaded cell. Two blue lines and an orange curve are then plotted:

- The solid blue lines plot the years since the most recent hypothetical fire, starting at 0 years and increasing with a slope of 1:1. After reaching the full number of years in the average fire interval (which is computed as the inverse of the annual burn probability), a second line drops back to 0 years, representing the next hypothetical fire. The subarea's "representative" condition is taken as half its average fire interval. This point is plotted on the curve, and its value is labeled.

- The dashed orange curve plots the adjusted CN value over the course of the recovery period. An orange point is plotted and labeled at the adjusted CN value corresponding to the average number of years since the most recent hypothetical fire.

The average fire interval represents the typical time between fires in the subarea, and the average time since the last hypothetical fire is assumed to be half of this interval. This average time since the last hypothetical fire is then compared to the estimated recovery period. For example, if the average fire interval is 30 years, the representative condition corresponds to approximately 15 years since the last hypothetical fire. If the recovery period is shorter—say, 12 years—the subarea is considered fully recovered at the representative condition because more time has passed than the recovery period. In such cases, the pre-fire CN value should be used for analysis, indicating that no CN adjustment is necessary.

The spreadsheet tool should be used independently for each subarea located in an area potentially affected by fire under representative conditions, and the results attached to the study report.

6.3. Adjusting the Curve Number (CN) 20 Years or Less Post-Fire

6.3.1. Identification of Characteristic Watershed Burn Condition Parameters

When one or more subareas in a hydrologic watershed model burned in the past 20 years, users should account for the current burn conditions by evaluating three parameter values for each burned subarea using GIS analysis. Unlike the methods presented in **Section 6.2**, these parameters characterize the known burn severity and the elapsed time since the most recent fire, rather than a hypothetical, representative burn condition. Subareas should be selected to coincide with areas of similar hydrologic characteristics, including burn characteristics.

The three parameters include the following:

Percentage of Area Burned with Moderate Severity: This is the percentage of the subarea's total area characterized by the moderate burn severity class for the most recent fire, based on a burn severity map developed for that fire. Appropriate datasets may either be a four-class, burned-area reflectance classification (BARC4) layer—derived from remotely sensed post-fire data—or a refined soil burn severity (SBS) layer developed by a federal Burned Area Emergency Response (BAER) team or a California State

Watershed Emergency Response Team (WERT) using ground-based survey data. SBS maps should be given precedence over BARC4 maps whenever they exist.

In many cases, burn severity maps for historical fires are available in the multi-agency Burn Severity Viewer interface (https://burnseverity.cr.usgs.gov/viewer/), where users can select and view available products developed for a defined date range and location, then download burn severity raster layers for individual fires.

Percentage of Area Burned with High Severity: This is the percentage of the subarea's total area characterized by the high burn severity class for the most recent fire, based on a burn severity map developed for the fire of interest. The same raster layer used to identify the percentage burned with moderate severity should be used to identify the percentage burned with high severity.

Time Since Last Fire: Users must determine whether some or all the study watershed burned during the past 20 years, and, if so, the date of the most recent fire in each project subarea. CAL FIRE's Fire and Resource Assessment Program maintains a database of historic fire perimeters that can be downloaded as a geodatabase (https://www.fire.ca.gov/what-we-do/fire-resource-assessment-program/fire-perimeters) for use with GIS applications or viewed online in an interactive map.

6.3.2. CN Adjustment for Post-Fire Watershed Burn Conditions

After evaluating the three parameter values for each burned subarea, users must apply a standard spreadsheet tool to identify an appropriate adjusted CN in cases where an adjustment is warranted, based on the methods proposed by Livingston et al. (2005). The worksheet containing the tool, titled "*CN Adjust – Post-fire Cond*," is included in an Excel workbook titled "*CN Adjustment Tool*," which is available for download at the San Bernardino County Flood Control webpage.

Using the Excel worksheet, after entering basic identifying project reference information, the following procedures should be followed:

Step 1: Determine the WHI classification by inputting the computed moderate and high burn severity percentages for the subarea in the orange-shaded cells. A red point is added to the plot and the corresponding WHI classification is identified in the red text. When the point is located near or on a dashed line separating two WHI classes, users should complete the CN adjustment analysis for both WHI classes and either average the results or select one based on engineering or management considerations.

Step 2: Determine the Initial CN Ratio by inputting the pre-fire (unburned) CN value, selected based on guidance in Chapter 3. The pre-fire CN value is plotted on the appropriate WHI curve, and a red callout label notes the computed ratio between the CN before and immediately following the most recent fire.

Step 3: Review the Subarea's Estimated Recovery Period; a yellow-dashed recovery curve is plotted starting at the initial CN ratio at Year 0, then decreases following a characteristic trajectory until it crosses the x-axis. The subarea's theoretical recovery period is rounded to the nearest year and labeled in red text.

Step 4: Estimate the Adjusted CN by inputting the time since the most recent fire (in years) in the orange-shaded cell. A blue line and an orange curve are plotted:

- The solid blue line plots the years since the most recent fire, starting at 0 years and increasing with a slope of 1:1 until reaching the time since the most recent fire. This point is plotted on the curve, and its value is labeled.

- The dashed orange curve plots the adjusted CN over the course of the estimated recovery period. An orange point is plotted and labeled at the adjusted CN value corresponding to the number of years since the most recent fire.

If the number of years since the most recent fire is greater than the estimated duration of the subarea's recovery period, the subarea is considered fully recovered from the most recent fire at the time of the analysis and the pre-fire CN value should be used in the analysis (i.e., no adjustment is required).

The computed adjusted CN reflects the "ideal" recovery model proposed by Livingston et al. (2005) and may be more appropriate in some cases than others, given the variability in watershed recovery processes. Because of this, the adjusted CN should be validated whenever possible using observations of current conditions and users should consider the post-fire weather that preceded the analysis. In some cases, such as a prolonged drought at the start of a recovery period that delayed vegetation regrowth, the CN should be further adjusted based on engineering judgment. However, any adjustments must be justified, particularly when they result in a smaller CN than the value predicted using the Livingston et al. (2005) methods.

The spreadsheet tool should be used independently for each subarea that burned partially or fully in the past 20 years, and the results should be attached to the study report.

6.4. Post-Fire Example

The following example illustrates the methods prescribed for hydrology studies of watersheds under representative burn conditions and during the post-fire period. This example considers the case of a hypothetical hydrologic watershed study performed in 2024.

Based on **Figure 6-5**, undeveloped portions of this watershed fall within the target area requiring special analysis of the effects of fire under representative conditions (**Section 6.2**). Based on available fire perimeter data shown in **Figure 6-6**, portions of the example watershed also burned during the past 20 years. As such, the watershed must be modeled under both representative conditions and current post-fire burn conditions, and the larger resulting flow taken as the design flow.

The following subsections describe both types of analyses for a given subarea in the project watershed, shown in **Figure 6-7**; note that similar analyses must be performed for all affected subareas in the watershed model.

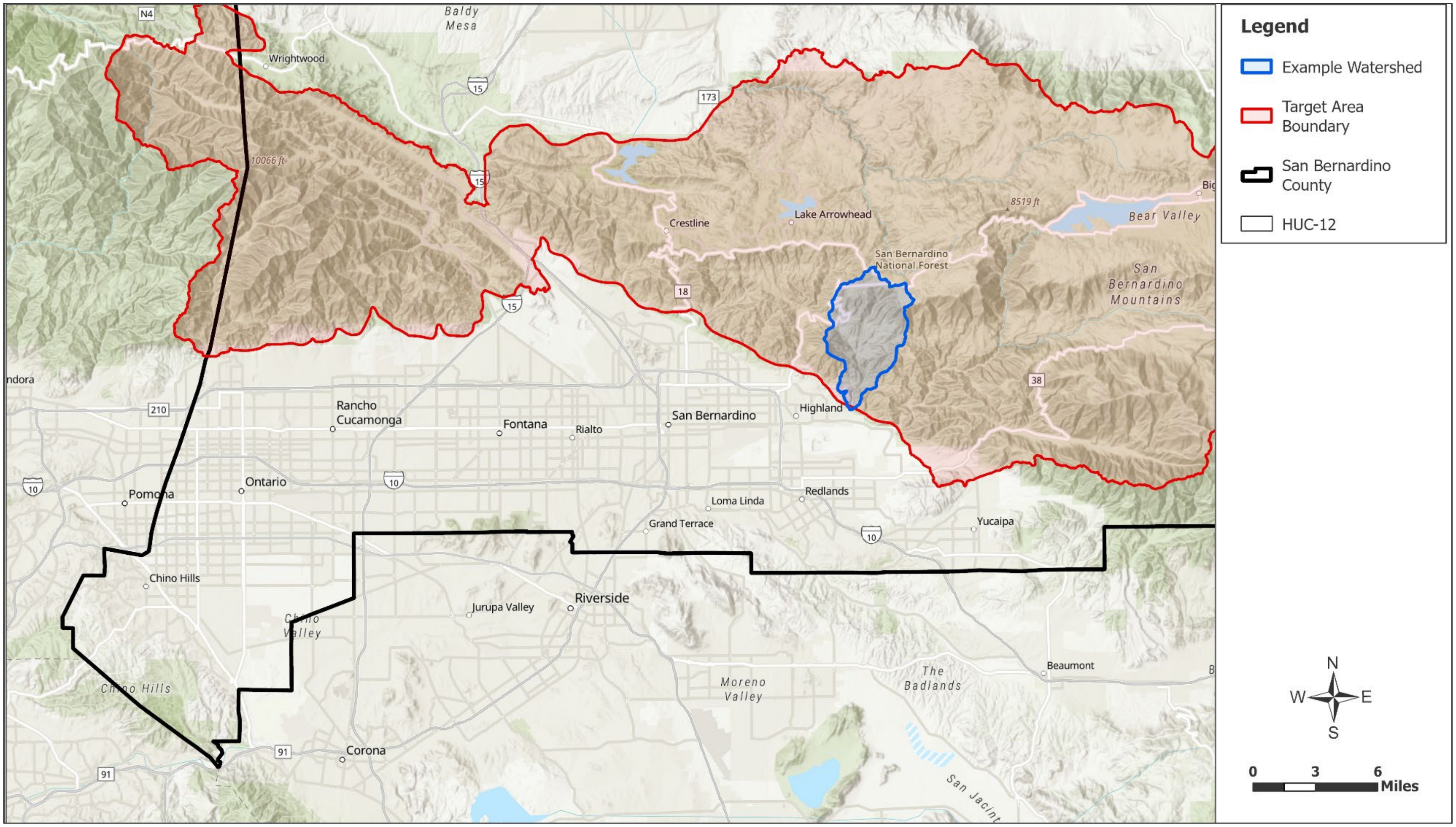

Figure 6-5. Example Watershed located in the Target Area Boundary

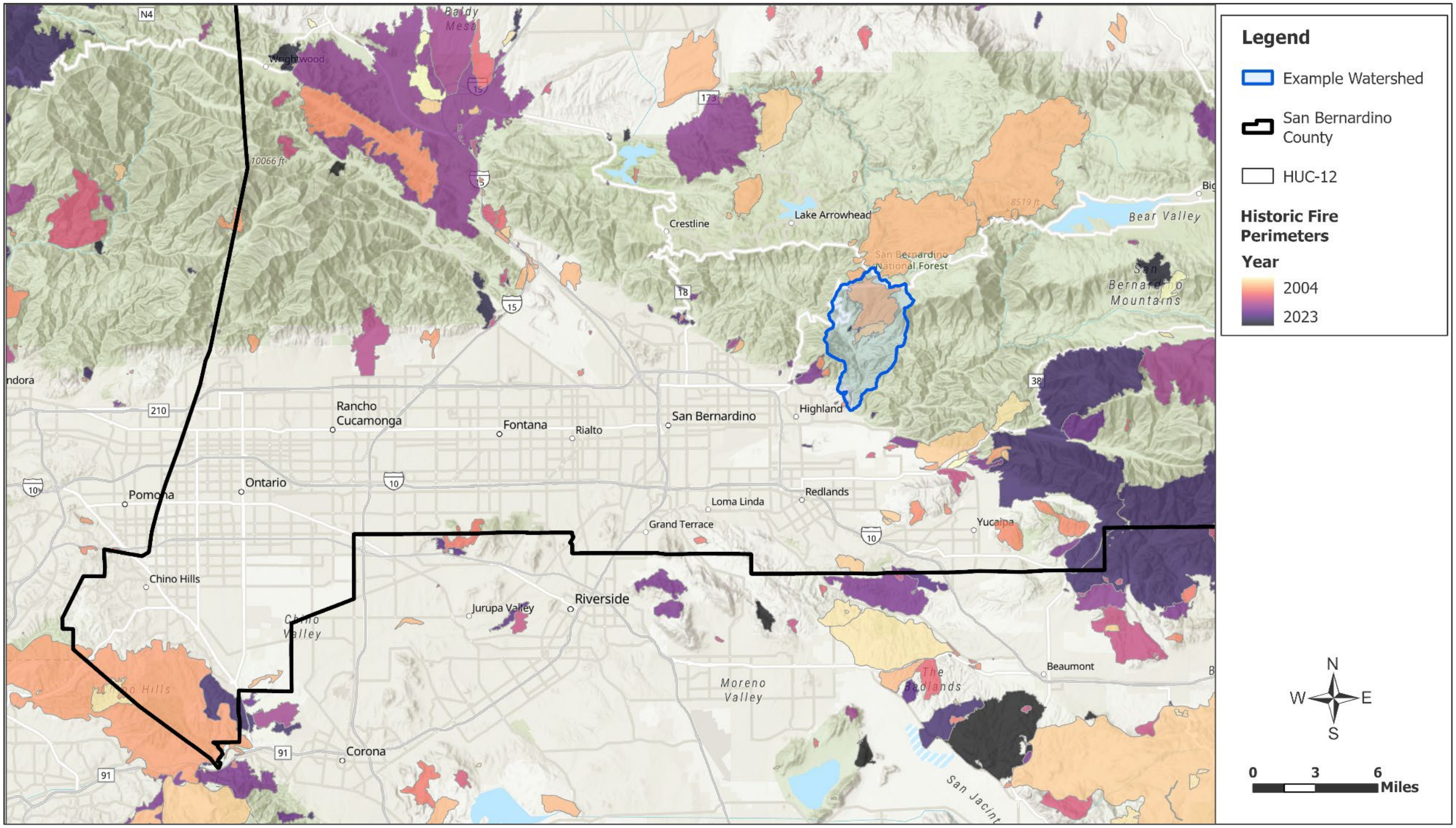

Figure 6-6. Example Watershed with Historical Burn Perimeters (2004–2023)

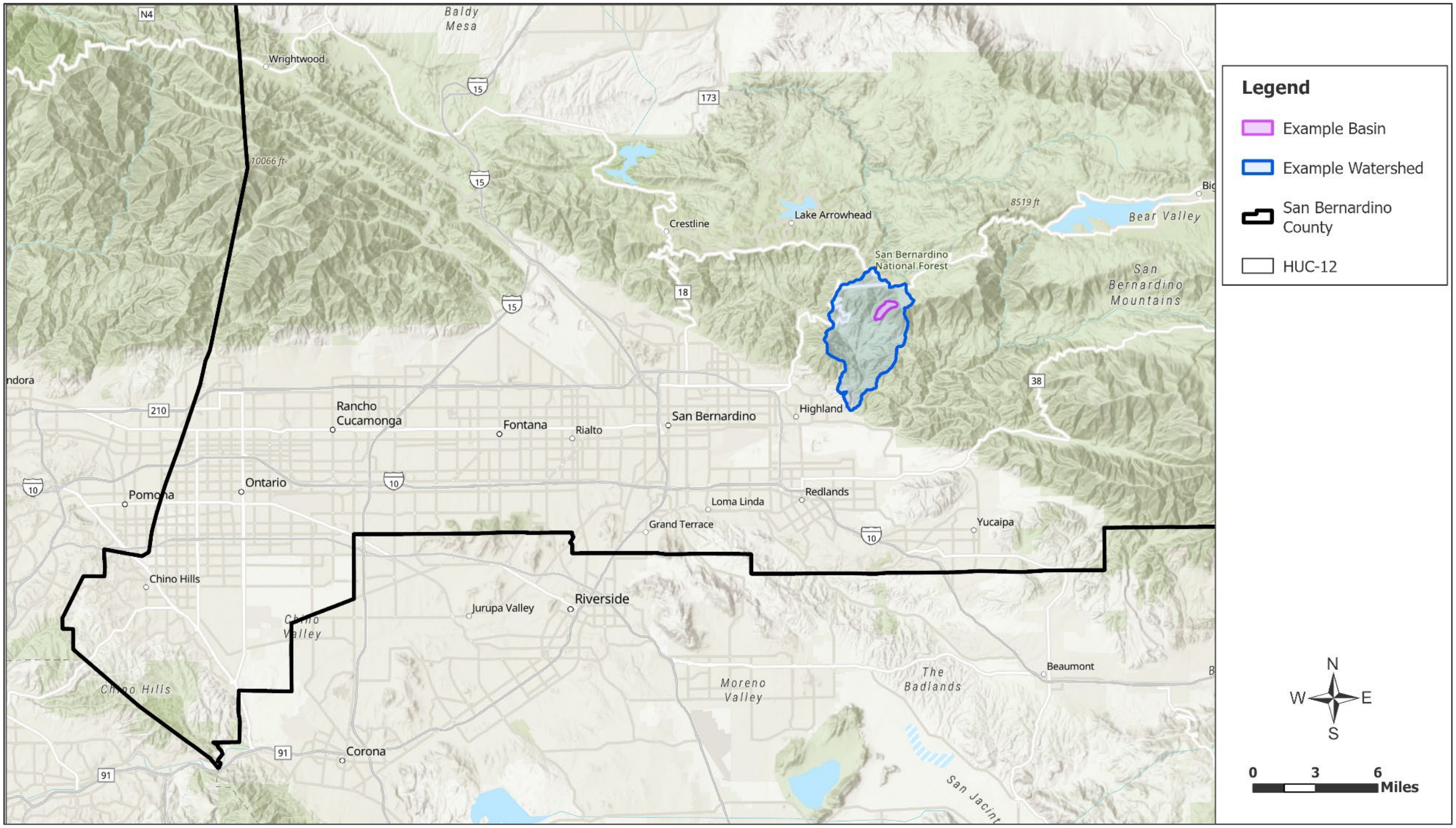

Figure 6-7. Example Subarea located within Example Watershed

6.4.1. Example Assessment for Representative Burned Watershed Conditions

1. Determine the WHI classification for the example subarea (see Section 6.2.3).

Based on GIS analysis of the 50th percentile synthetic burn severity dataset ("*SBC_SimSeverity_P50_Clipped.tif*"), 0% of the subarea burns with high severity and 100% with moderate severity. Inputting these values in Step 1 of the *CN Adjust – Representative Cond* sheet results in a WHI classification of Moderate, as shown in **Figure 6-8.**

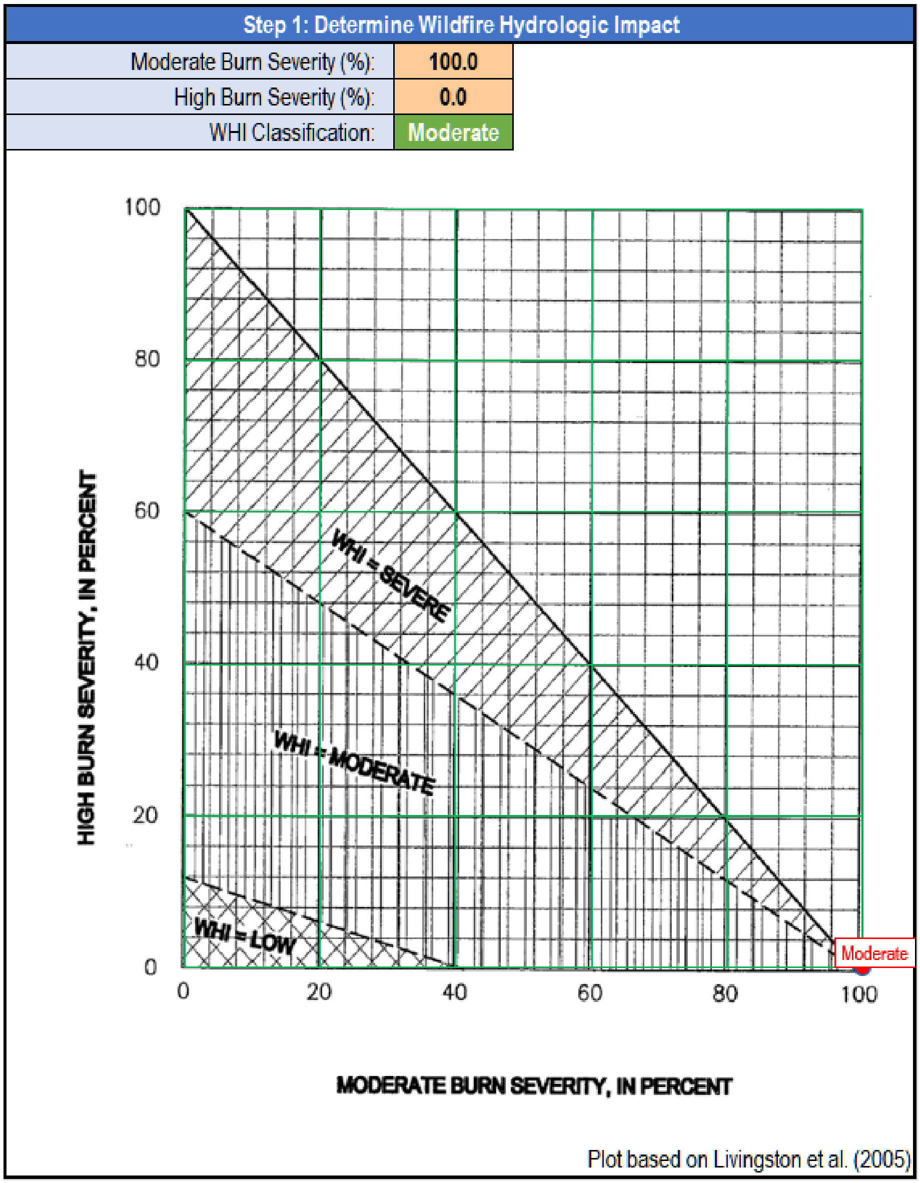

Figure 6-8. Step 1: Determine WHI Classification – Representative Conditions

The plotted point falls on the boundary between the Moderate and Severe regions; therefore, one of the two should be selected based on engineering judgment or management considerations. Alternatively, evaluate the CN adjustment for both WHI classes and adopt the average as the final adjustment. For brevity, this case assumes the Moderate classification. Alternatively, users may select the Severe WHI classification by instead entering, for example, 0% moderate burn severity and 100% high burn severity.

2. **Identify and input the subarea's pre-fire CN value to compute the initial CN ratio (see Section 6.2.3).**

   The subarea consists mainly of evergreen forest with Type A soils, and some Type D. Assuming fair cover quality and AMC II, the area-weighted pre-fire CN is taken as 50, and this value is entered in Step 2 of the spreadsheet. The initial CN ratio (1.57) is plotted on the Moderate WHI curve, as shown in **Figure 6-9**.

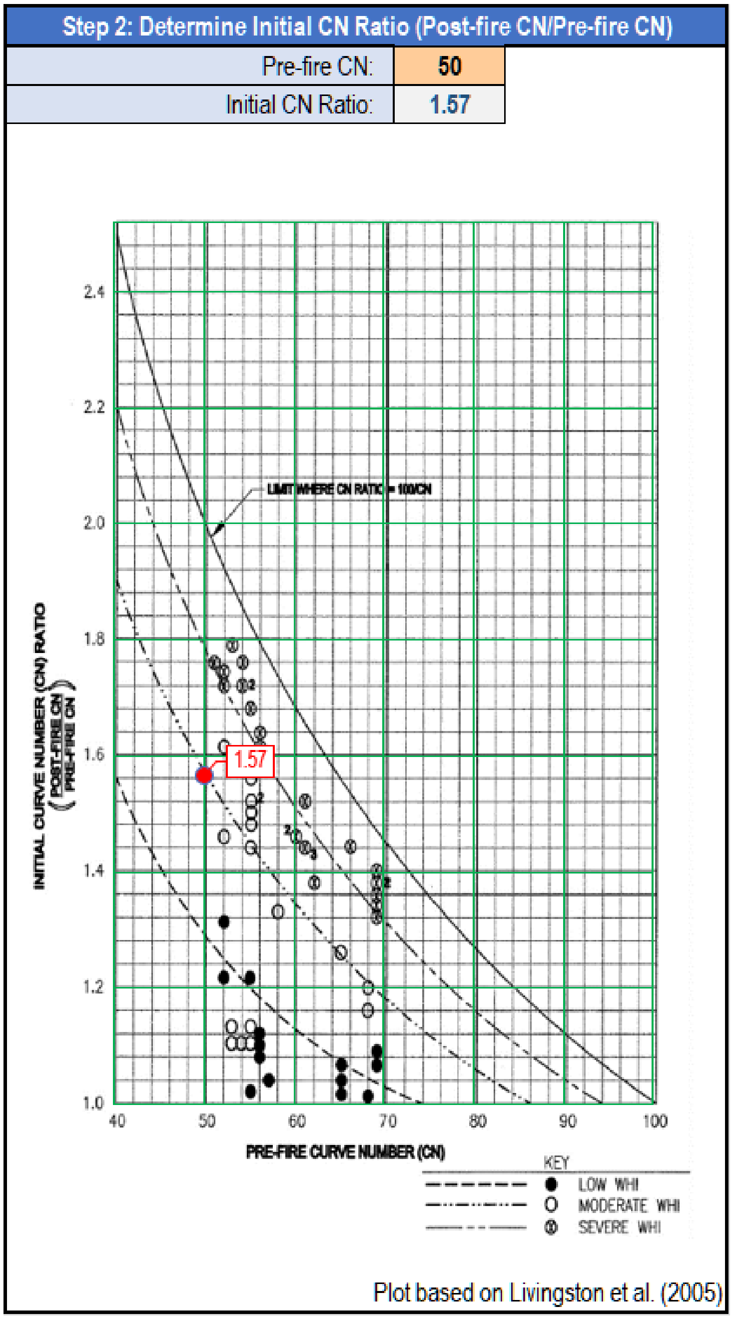

Figure 6-9. Step 2: Determine Initial CN Ratio – Representative Conditions

3. **Review the Subarea's Estimated Recovery Period (see Section 6.2.3).**

   No user input is required in Step 3, but the dashed yellow line showing the ideal recovery curve looks reasonable, with a theoretical recovery period of approximately 23 years. The plot is shown in **Figure 6-10**.

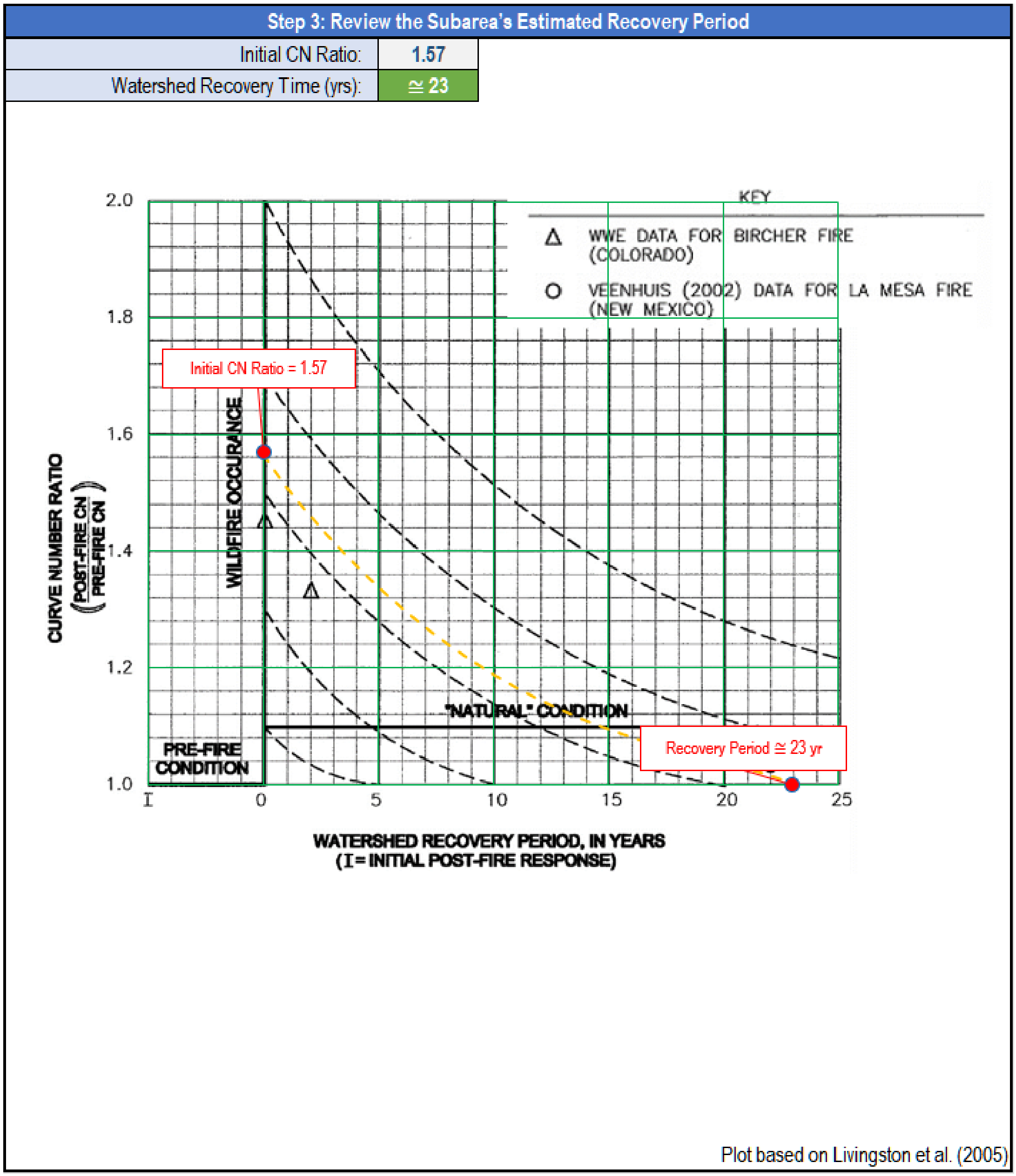

Figure 6-10. Step 3: Estimate Recovery Period – Representative Conditions

4. **Compute and input the subarea's area-averaged annual burn probability to estimate the adjusted CN value (see Section 6.2.3).**

   The subarea's area-averaged annual burn probability is computed as 0.047, based on the Pyrologix LLC (2023) annual burn probability raster dataset, and this value is entered in Step 4. As shown in Figure 6-11, this annual probability is equivalent to an average fire interval of 21 years, with half of that interval equal to 10.5 years. At the corresponding point in the watershed recovery curve, the adjusted CN is 59, which is assumed for the subarea in the representative conditions assessment.

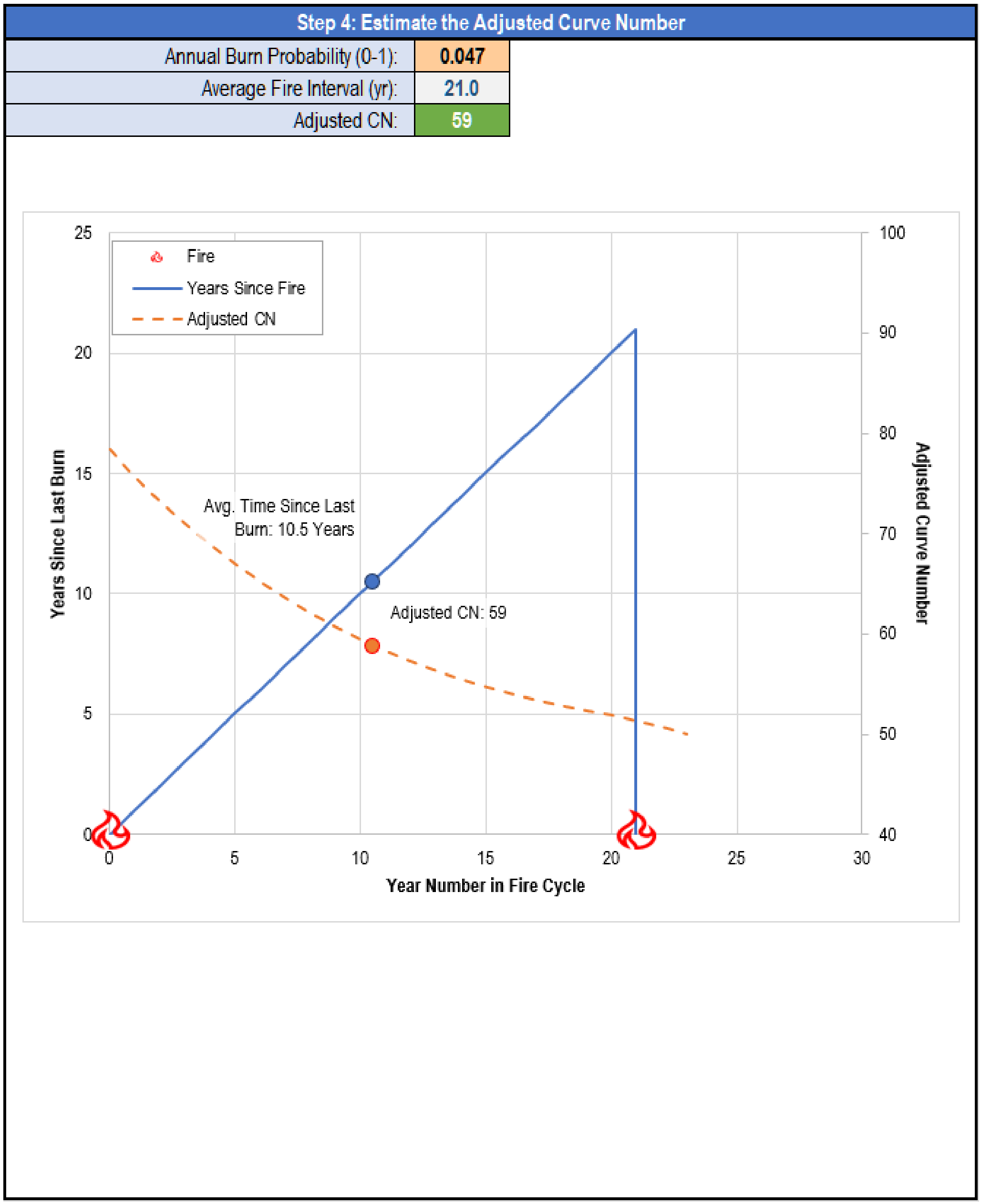

Figure 6-11. Step 4: Estimate the Adjusted Curve Number – Representative Conditions

5. **Repeat Steps 1–4 for all applicable subareas.**

6. **Evaluate the basin roughness factor for the representative watershed condition (see Section 6.2).**

   Based on the above analysis, the hypothetical average conditions reflect a watershed that is largely, but not yet fully, recovered from a hypothetical fire 10.5 years prior. In most cases, the majority of the post-fire debris will have been flushed from channels a decade after a fire, so any initial increases in channel roughness values immediately following a fire will likely have decreased to approach the pre-fire, recovered value. Other factors affecting collector roughness, such as vegetation regrowth, will depend on the vegetation type, and a small adjustment to the roughness value relative to the current observed condition may be warranted.

6.4.2. Example Assessment 17 Years Post-Fire

Next, the watershed is evaluated for current burned conditions.

1. **Identify and input characteristic burn severity values for the most recent fire in the example subarea (see Sections 6.3.1 and 6.3.2).**

   Based on the CAL FIRE historical fire perimeter database, the example subarea burned most recently in the Slide Fire of October 2007. A BARC4 burn severity raster for this fire is downloaded from the multi-agency Burn Severity Viewer interface, and based on GIS analysis, 83.9% of the subarea burned with high severity and 14.4% burned with moderate severity.

   Inputting these values in Step 1 of the *CN Adjust – Post-fire Cond* sheet results in a WHI classification of Severe, as shown in **Figure 6-12**.

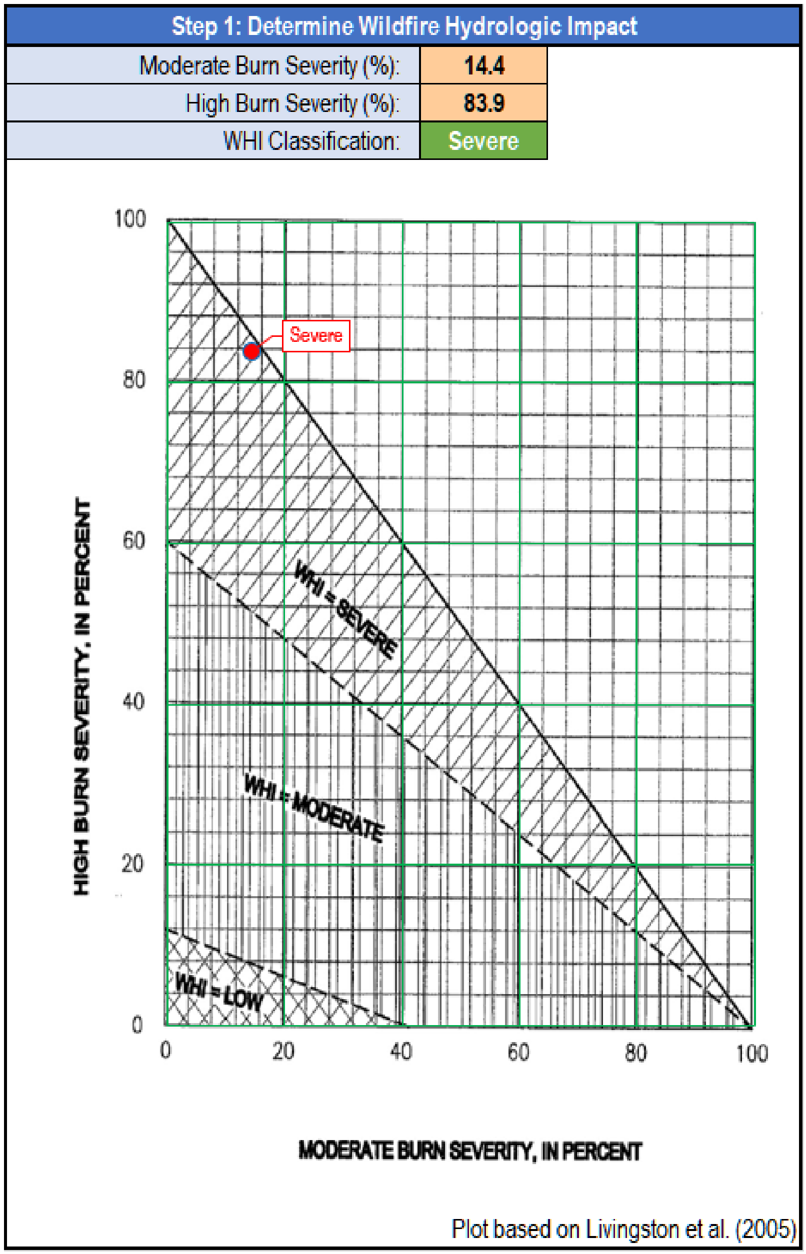

Figure 6-12. Step 1: Determine WHI Classification – Post-Fire Conditions

2. **Identify and input the subarea's pre-fire CN value to compute the initial CN ratio (see Section 6.3.2).**

   As in Step 2 (see **Section 6.4.1**), the area-weighted pre-fire CN is taken as 50, and this value is entered in Step 2 of the spreadsheet. The initial CN ratio (1.79) is plotted on the Severe WHI curve, as shown in **Figure 6-13**.

Step 2: Determine Initial CN Ratio (Post-fire CN/Pre-fire CN)	
Pre-fire CN:	50
Initial CN Ratio:	1.79

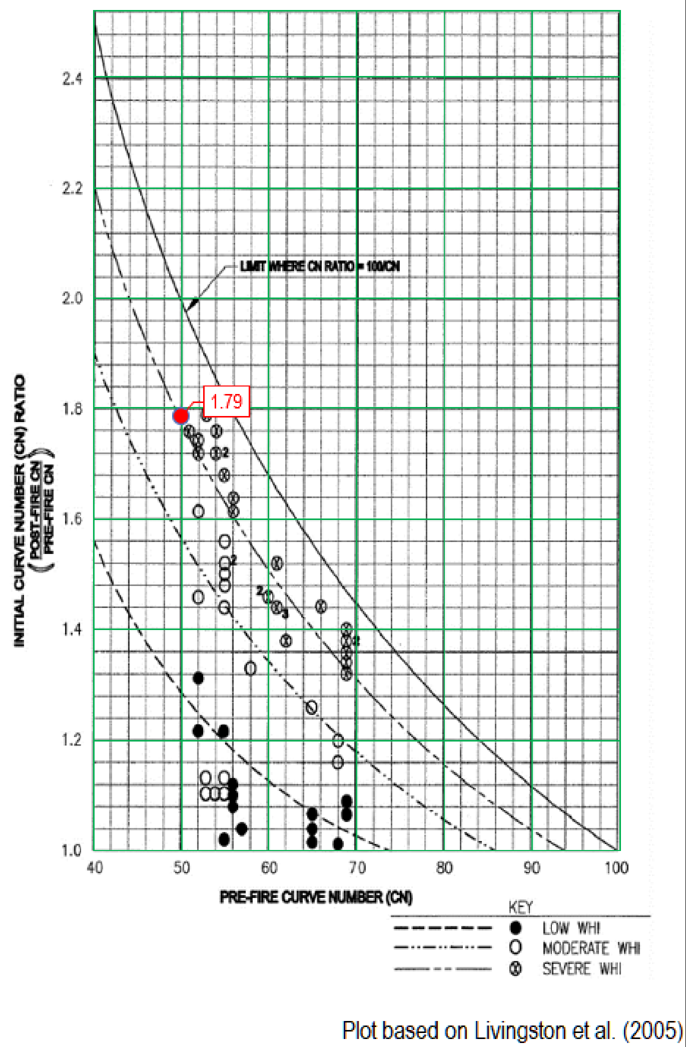

Figure 6-13. Step 2: Determine Initial CN Ratio – Post-Fire Conditions

3. **Review the Subarea's Estimated Recovery Period (see Section 6.3.2).**

   Due to the Severe WHI classification, the theoretical recovery period exceeds 25 years—longer than the estimate for the representative conditions case—as shown by the dashed yellow line in **Figure 6-14**.

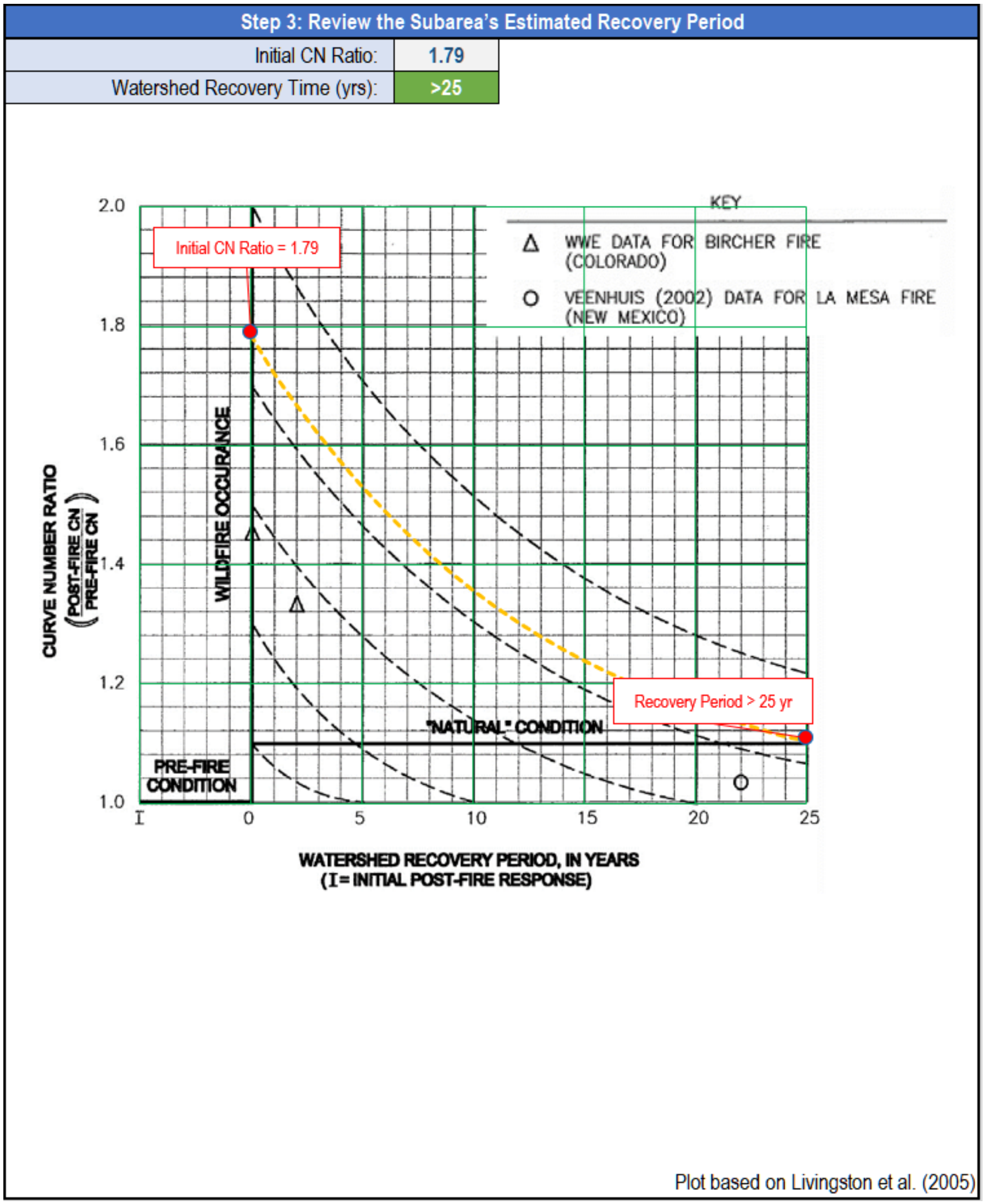

Figure 6-14. Step 3: Estimate Recovery Period – Post-Fire Conditions

4. **Input the time since the most recent fire to estimate the adjusted CN value (see Section 6.3.2).**

   This example assumes that 17 years have passed since the October 2007 Slide Fire. This value is entered in Step 4, as shown in **Figure 6-15**. At this (current) point in the recovery curve, the adjusted CN is 60, and this value is assumed for the post-fire conditions assessment.

Step 4: Estimate the Adjusted Curve Number	
Time Since Fire (yr):	17
Adjusted CN:	60

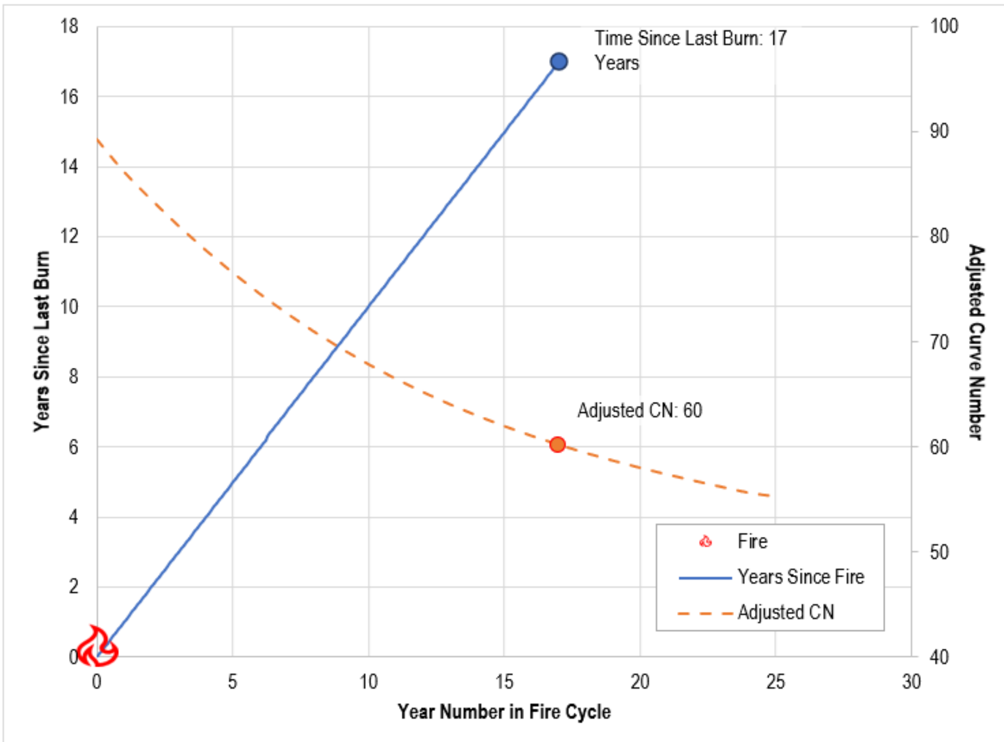

Figure 6-15. Step 4: Estimate the Adjusted Curve Number – Post-Fire Conditions

5. **Repeat Steps 1–4 for all applicable subareas.**

6. **Evaluate the basin roughness factor for the post-fire condition (see Section 6.3).**

   Ideally, personnel with post-fire hydrology experience would make a visual assessment of the watershed's current condition and use engineering judgment and standard references to estimate the mean Manning's n roughness value ($\bar{n}$) of the channels within the basin; this value is then used to compute USACE Lag.

6.4.3. Evaluation of Design Flows

Evaluate and compare clear-water peak flows and hydrographs at the project location using methodological guidance provided in **Chapter 5**—one set for the representative conditions, using adjusted CN and USACE Lag values evaluated for that scenario, and one for the post-fire (current) conditions, using adjusted CN and USACE Lag values evaluated for that scenario.

The larger (i.e., more conservative) of the resulting flow peaks is taken as the clear-water design flow.

CHAPTER 7
SEDIMENT BULKING

7.1. Introduction

In general, erosion is the loosening or dissolution and removal of material (i.e., sediment) from the earth's surface. Materials may be eroded, or mobilized, by a variety of forces, including rain-splash, flowing water, wind, and human activity, and are then typically transported and deposited in another location. While surface movement due to gravity can be the primary erosion process in steep watersheds dominated by bedrock (DiBiase and Lamb 2020) or non-cohesive soils (Lamb et al. 2011), water is the primary agent responsible for eroding and transporting sediment downslope in the landscape; therefore, the movement of sediment is an important consideration for flood control engineering projects (Vanoni 2006).

In addition to sediment, vegetative debris sometimes comprises an appreciable portion of material transported from watershed hillslopes into streams. In this chapter, the term "sediment/debris" is used generally to include both categories.

As discussed in **Chapter 6**, wildfires commonly result in larger clear-water peak flows for a given precipitation event. This response results from decreased losses and surface roughness, which increase runoff volumes, accelerate flow velocities, and shorten times of concentration.

Wildfire also combusts soil-binding organic matter, and this, coupled with the larger, faster overland flows, frequently results in accelerated sediment stripping from hillslopes. Wildfire can also dramatically increase the supply of woody debris to streams (Moore et al. 2016). Total sediment/debris production (and yield, assuming sufficiently large flows) may increase by a factor of more than 30 during the first year following a fire, before decreasing substantially in the second year and returning to normal in about 10 years (Rowe et al. 1949, 1954; Scott and Williams 1978).

The introduction of large quantities of sediment and debris into post-fire runoff can dramatically affect both the magnitude and fluid dynamics of flows and produce highly destructive debris flows, particularly in mountainous areas. Debris flow rates may take 1 to 3 years to return to pre-fire rates, while elevated sediment transport rates may persist for a longer period (Gartner et al. 2014, Moore et al. 2016, Santi and Morandi 2013).

7.1.1. Sediment/Debris Hazards and Flood Control Infrastructure

Flood control infrastructure (such as dams, sediment/debris basins, and channels) is designed to protect property and lives by safely conveying and/or storing flood flows. Because sediment/debris displaces water and contributes to an increase in the volumetric flow rate and flood depth, engineers must consider the role of sediment/debris in the evaluation of design flows. When increased loading is expected, engineers typically have two options: (1) capture and temporarily store excess sediment/debris in a sediment/debris basin and/or (2) increase the capacity of flood control facilities beyond the clear-water flow rate to account for the volume of the additional sediment/debris. Consequently, sediment/debris yield and sediment/debris bulking are important considerations for engineering design.

Sediment/debris yield is the quantity of eroded sediment and vegetative debris delivered to a watershed outlet for a specified flood event or period. Sediment/debris yields for design storm events are often computed at canyon mouths to inform the sizing of sediment/debris basins, particularly in areas prone to wildfire and debris flows.

Sediment/debris bulking is the process used to increase design flows beyond the clear-water flow rate to account for the volumetric contribution of transported sediment/debris. When designing structures downstream of highly erodible areas, and particularly those areas subject to wildfires, engineers use different approaches to account for the additional flow volume and changes in flow behavior associated with high sediment/debris concentrations—including the potential for short-lived amplification of flows due to the formation of dilated surge fronts during debris flow events (Iverson 1997, Johnson et al. 2012, Kean et al. 2016). Frequently, the clear-water discharge is computed based on a hydrologic analysis, then the peak clear-water flow rate is multiplied by a factor greater than 1 to compute a design flow that includes the volumetric contribution of the entrained sediment/debris. This is known as a bulking factor, and it is equal to the ratio of the bulked discharge to the clear-water discharge. Bulking factors typically vary based on watershed size and condition, with the largest bulking factors applied to small, recently burned watersheds.

As an alternative to applying a standard bulking factor, engineers can compute the sediment/debris yield at a project location and apply it to the clear-water hydrograph to develop a bulked design hydrograph.

7.1.2. Addressing Sediment/Debris Hazards

This chapter provides methods for addressing sediment/debris hazards under two scenarios:

> **Addressing Sediment/Debris Hazards for Representative Conditions:** Some watersheds in San Bernardino County are very erodible under typical conditions and sediment/debris hazards must be addressed as part of routine design, regardless of the burn condition at the time of the analysis. **Section 7.2** prescribes methods for watersheds falling within one of these areas.
>
> **Addressing Sediment/Debris Hazards ≤ 10 Years Post-fire:** While the greatest impacts of fire on sediment/debris loading typically diminish within a few years following a fire, elevated loading often persists for about 10 years, relative to pre-fire conditions. **Section 7.3** prescribes methods for sediment bulking assessments in watersheds which partially or fully burned within the past 10 years.

When performing a sediment bulking assessment, users must determine whether some or all of the study watershed burned during the past 10 years, and, if so, the date and coverage area(s) of the fire(s). CAL FIRE's Fire and Resource Assessment Program maintains a database of historic fire perimeters that can be downloaded as a geodatabase (https://www.fire.ca.gov/what-we-do/fire-resource-assessment-program/fire-perimeters) for use with GIS applications. The historic fire perimeters can also be viewed online in an interactive map https://experience.arcgis.com/experience/b72eede32897423683a94c61bf9d3027. When the latter is used, the filter tool can be applied to select the 10 most recent fire years.

When a routine sediment bulking analysis is performed for a watershed requiring analysis under typical conditions (as indicated in **Section 7.2**) and that has burned within the past 10 years, the sediment bulking analysis should be performed twice: once for the typical conditions, described in **Section 7.2**, and once for the current, post-fire conditions, following the guidance in **Section 7.3**. Then, the larger of the sediment/debris yield and/or bulked peak flows should be used for design.

7.1.3. Emergency Post-Fire Sediment Bulking Assessments

The effects of a fire on sediment/debris yield are generally greatest during the first 2 years following the fire. During this period, rapid assessment of extreme sediment/debris hazards to support the design of temporary infrastructure and mitigation measures may be needed. These assessments often rely on simplified methods that can be implemented quickly with limited input data and depend heavily on engineering judgment. Due to the wide variability in post-fire conditions, data availability, and the rapidly evolving nature of research and best practices, users are advised to coordinate with the County and/or the California Geological Survey's Burned Watershed Geohazards Program when conducting emergency sediment/debris evaluations during an identified emergency period. Specific methods for evaluating and mitigating sediment/debris hazards during the first 2 years following a wildfire are not included in this Manual.

7.2. Addressing Sediment/Debris Hazards for Representative Conditions

Users must complete up to three steps to address potential sediment/debris hazards, described in the following sections.

7.2.1. Step 1: Determine Whether Watershed is Potentially Affected by Sediment/Debris Under Representative Conditions

Sediment/debris is typically not explicitly considered in design hydrology when normal streamflows are expected; rather, bulking factors are usually applied when hyperconcentrated or debris flows are anticipated during the design life of a facility. In San Bernardino County, these conditions are expected in mountainous areas generally subject to fire and subsequent erosion and in arid regions where the facility is near an alluvial fan. This means that many drainage basins are not expected to yield enough sediment during design events to warrant the application of sediment bulking factors under normal, pre-fire conditions. In Step 1, users must identify whether the project is located within or immediately downstream of selected areas with elevated risks.

Figure 7-1 identifies areas requiring consideration of the effects of sediment/debris under typical conditions. These include the following areas:

- **Sediment/debris hazard regions associated with areas having at least a 1% annual chance of any debris flows or at least 0.5% annual chance of major debris flows:** The classification of major debris flows follows the definition proposed by Kean and Staley (2021), which is based on a 15-minute rainfall event more than three times larger than the triggering threshold. Areas meeting these criteria include watersheds in the San Gabriel and San Bernardino Mountains, as well as the Chino Hills near the southwest corner of the county.

- **Potential debris flow runout reaches downstream of debris flow hazard areas:** A debris flow runout reach is the depositional flow path of the debris flow. Potential debris flow runout reaches originate in debris flow hazard areas with average slopes of 2% or greater. Affected areas include the floodplains of the reaches shown.

- **Active desert alluvial fans:** The California Geological Survey has performed detailed mapping of alluvial fans within only some quadrangles in the county. Where detailed mapping data are available, hazard areas include alluvial fans active in the Late Holocene. In the unmapped quadrangles (i.e., those areas with more shading), hazard areas include all areas where the California Geological Survey has indicated alluvial fans may be present (Bedrossian et al. 2012). Note that the hazard areas in the latter quadrangles are conservative and many indicated hazard areas are not expected to contain active alluvial fans.

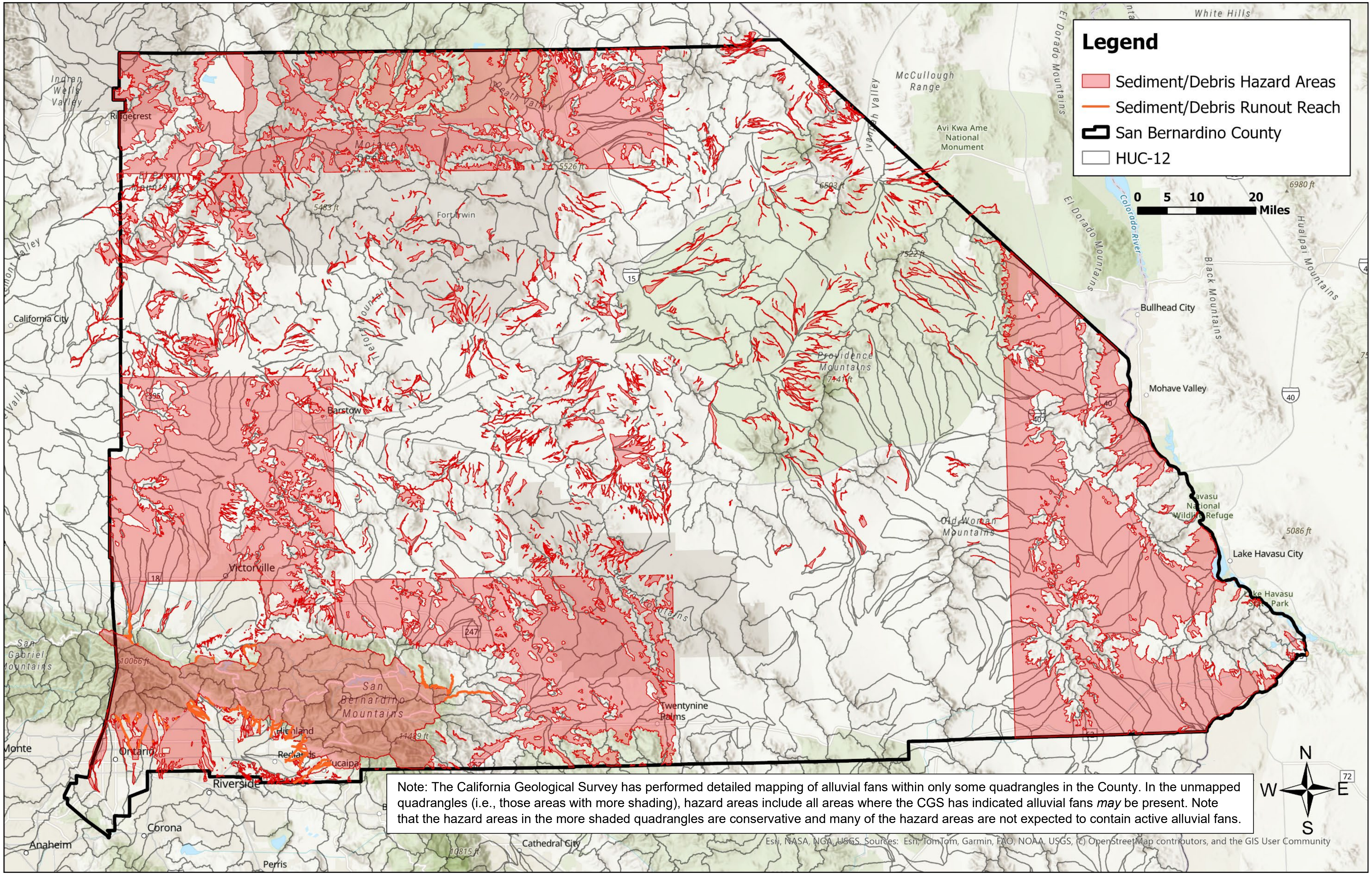

Figure 7-1. Watershed Areas Requiring Sediment/Debris Analysis under Typical Conditions

When a hydrologic analysis is performed for a watershed falling within one of the above areas, users should proceed to Step 2 to determine whether planned infrastructure is at risk. Otherwise, sediment/debris does not need to be explicitly considered in hydrologic design.

7.2.2. Step 2: Identify Whether Planned Infrastructure is at Risk

If a new project is planned in an identified sediment/debris hazard region, the next step is to evaluate whether facilities are at risk. For example, a small drainage project may be located within a sediment/debris hazard area but outside the floodplain of the stream with the debris flow potential and may not require consideration for sediment/debris. Likewise, detailed analysis (e.g., modeling) by a qualified geologist or engineer may indicate likely debris flow runout location(s) well upstream of a project planned near or in a channel with a slope greater than 2%.

When a project watershed is located within an identified sediment/debris hazard area but is assessed not to be at risk due to its location away from a debris-producing stream, the rationale for this assessment must be documented in the study report. Otherwise, when planned infrastructure is at risk, users should proceed to Step 3 to select an appropriate mitigation option.

7.2.3. Step 3: Select Appropriate Mitigation Option

Sediment/debris hazards should generally be mitigated using one or both of the following options:

Design sediment/debris detention basins to remove part or all the design sediment/debris yield from the design flow: To avoid the need for downstream flow bulking or to reduce the bulking factor required, sediment/debris basins may be designed at or near canyon outlets (i.e., not on an alluvial fan or valley fill area) and sized with sufficient capacity and appropriate hydraulics to capture and store all or part of the design sediment/debris yield. Sediment/debris basin design is addressed in detail in **Chapter 8**; however, guidance for evaluating the design sediment/debris yield is described here given the interrelationship between the mitigation options and methods.

Bulk the clear-water design flow to reflect the volumetric contribution of sediment/debris: In some cases, it may be infeasible or undesirable to construct a sediment/debris basin at the mouth of a canyon upstream of a project location, or a planned sediment/debris basin may have insufficient capacity to capture and store the design sediment/debris yield. In these cases, downstream flood conveyance infrastructure (e.g., engineered channels, culverts, etc.) must be sized to convey the bulked peak flow. Users can either develop a bulked hydrograph based on the clear-water hydrograph and the design sediment/debris yield or apply a standard bulking factor.

7.2.3.1. Evaluate Sediment/Debris Yield

When sediment/debris hazards will be addressed using a sediment/debris detention basin, or when flood control facilities will be designed to accommodate a bulked hydrograph, users must compute the watershed's design sediment/debris yield. This will be the sediment/debris yield at the project location resulting from the 100-year storm falling on representative watershed conditions, computed using the U.S. Army Corps of Engineers' Los Angeles District Debris Method (Gatwood et al. 2000). If a sediment/debris basin will be used to mitigate the sediment/debris yield, refer to **Chapter 8** for basin design guidance.

The Los Angeles District Debris Method enables users to estimate unit debris yield values for flood events larger than those with a 5-year recurrence and was developed using measured debris volumes deposited in sediment basins and dams at the mouths of canyons in Southern California. The method was principally intended for use in sizing structures to capture and store sediment/debris delivered from singular storm events that would then be emptied between storms, thus removing or reducing the need to bulk downstream design flows.

One of five multiple regression equations is selected based on the size of the watershed area, as shown in **Equation 7-1**. The most important distinction is between that of the equation for the smallest watersheds (0.1 to 3 sq. mi, corresponding to **Equation 7-1a**) and larger watersheds (**Equation 7-1b** through **Equation 7-1e**). Because regional storms tend to have variable intensity over large areas, short-term precipitation (specifically, the maximum 1-hour precipitation) is a more-accurate predictor of debris yield for small watersheds where runoff data are not typically available, whereas runoff is a better indicator variable (and more likely available) for larger watersheds.

Equation 7-1 Unit Debris Yield Equation Set (Gatwood et al. 2000)

a 0.1 to 3 sq. mi.

$$Log(D_y) = 0.65(Log(P))+0.62(Log(RR))+0.18(Log(A))+0.12(FF)$$

b 3 to 10 sq. mi.

$$Log(D_y) = 0.85(Log(Q))+0.53(Log(RR))+0.04(Log(A))+0.22(FF)$$

c 10 to 25 sq. mi.

$$Log(D_y) = 0.88(Log(Q))+0.48(Log(RR))+0.06(Log(A))+0.2(FF)$$

d 25 to 50 sq. mi.

$$Log(D_y) = 0.94(Log(Q))+0.32(Log(RR))+0.14(Log(A))+0.17(FF)$$

e 50 to 200 sq. mi.

$$Log(D_y) = 1.02(Log(Q))+0.23(Log(RR))+0.16(Log(A))+0.13(FF)$$

where:

D_y = Unit debris yield (cubic yards/sq. mi.)
P = Maximum 1-hour precipitation (inches to two places after the decimal, times 100)
Q = Unit peak runoff (cfs/sq. mi)
RR = Relief ratio (drainage slope, feet/mile)
A = Drainage area (acres)
FF = Non-dimensional fire factor

The Fire Factor requires the following information:

1. The Fire Factor is taken from one of two sets of curves selected based on the watershed area.

2. **Figure 7-2** presents the curve used for watersheds 0.1 to 3.0 sq. mi, and **Figure 7-3** presents the curves used for watersheds 3.0 to 200 sq. mi.

3. Additional details describing computation of fire factors for partially burned watersheds is described in Appendix A of Gatwood et al. (2000), which is available at: https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/files/156797234/156797311/1/1694874917182/Gatwood_2000_Los+Angeles+District+Debris+Method.pdf.

Gatwood et al. (2000) note that because the equations were developed based on readily available data from the San Gabriel Mountains, the equations cannot be used directly in other areas with different characteristics. To account for other important variables that differ in other parts of the region, such as surficial geology, soils, hillslope, channel geomorphology, etc., the authors describe the use of an appropriate Adjustment-Transposition (A-T) Factor, which is multiplied by the unit debris yield. Appendix B of Gatwood et al. (2000) presents four different techniques for calculating the A-T Factor, selected based on the available data.

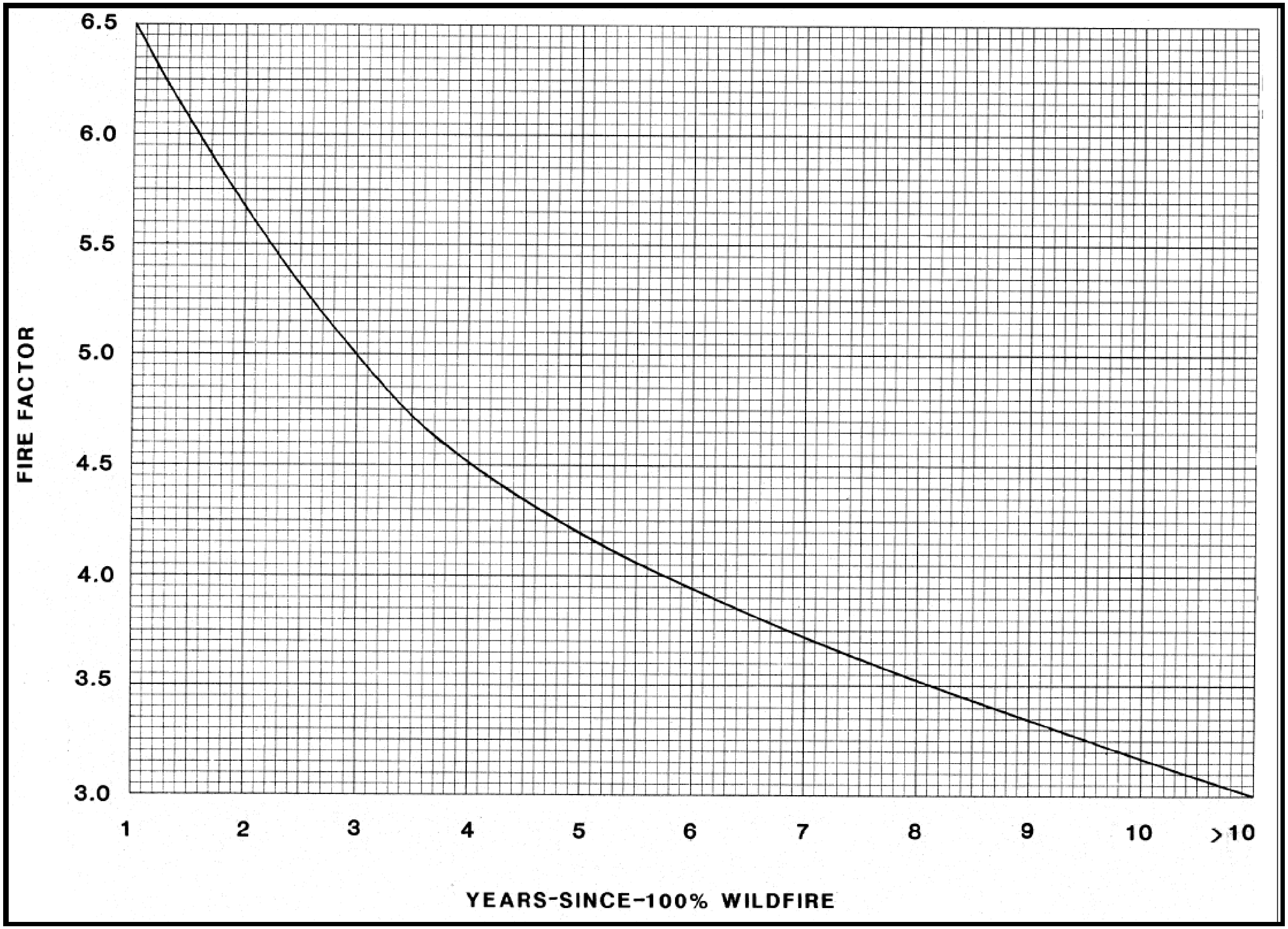

Figure 7-2. Fire Factor Curve for Watersheds 0.1 to 3.0 sq. mi (Figure A-1 from Gatwood et al. (2000))

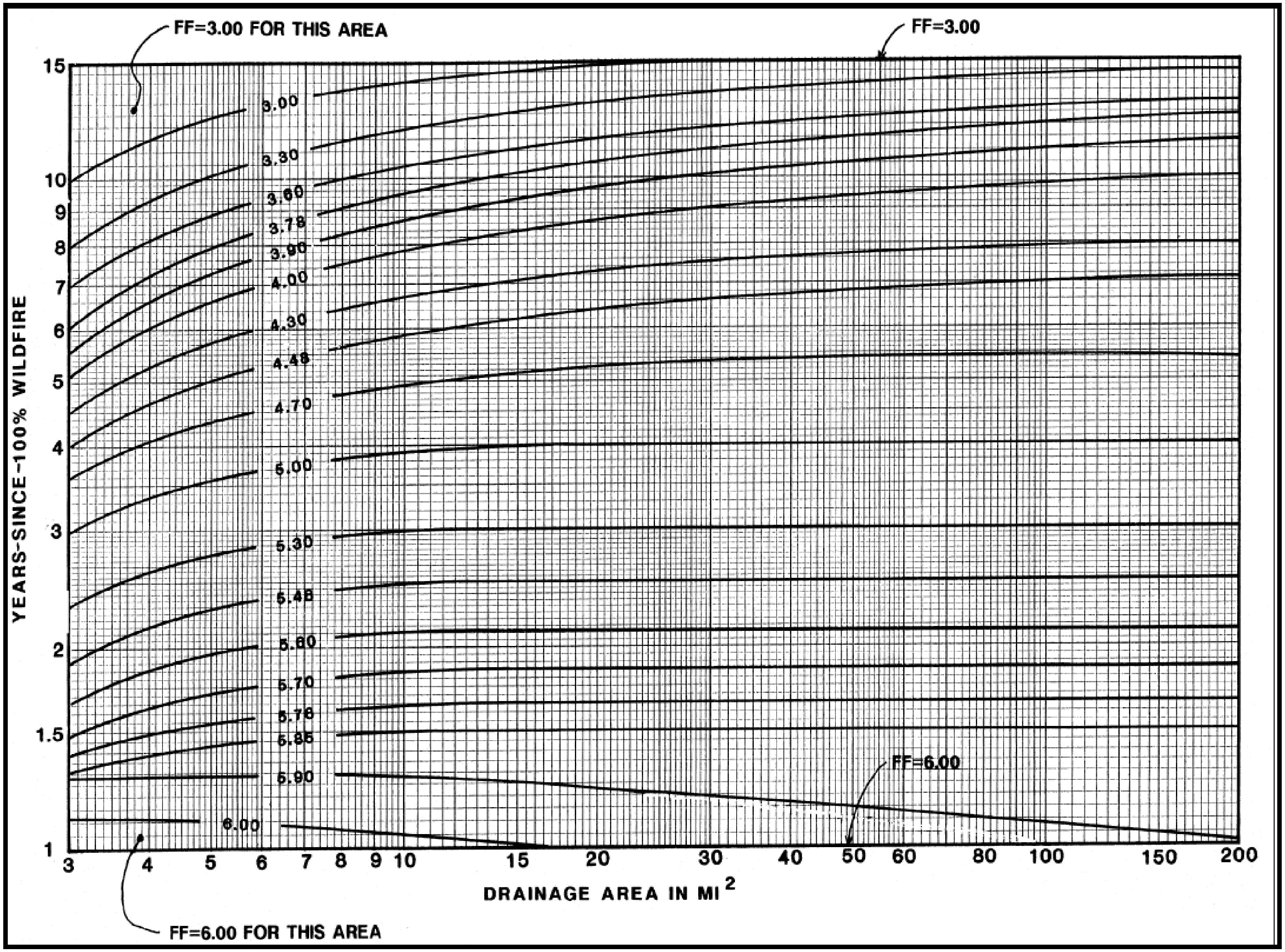

Figure 7-3. Fire Factor Curve for Watersheds 3.0 to 200 sq. mi (Figure A-2 from Gatwood et al. (2000))

Because the dimensionless Fire Factor is selected in part based on the time since the most recent fire, a value representative of the typical watershed conditions must be identified. This value—the hypothetical average time since the last fire—is taken as half of the average time between fires (i.e., the fire interval) in the watershed. The fire interval is computed based on the watershed's area-averaged annual burn probability, where the fire interval is the inverse of the burn probability.

A raster dataset of annual burn probabilities was developed by Pyrologix LLC (2023) and is available for download at https://rrk.sdsc.edu/socal.html. (Note that this is the same dataset used in **Section 6.2.2**.) The webpage includes a list of data products available for download; the relevant raw data layer, titled "Annual Burn Probability," should be used, and can be found in the *Severity* subsection of the *Fire Dynamics* section.

A standard spreadsheet tool developed by the County can be used to evaluate the Fire Factor. The worksheet containing the tool, titled "*Fire Factor – Representative*," is included in an Excel workbook titled "*Sediment-Debris Yield and Bulking Tool*," which is available for download at the San Bernardino County Flood Control webpage.

After entering the project reference information under the worksheet titled "*Fire Factor – Representative,*" input the watershed area (in acres) and the annual burn probability (as a percentage) in the orange-shaded cells. A note will appear in Row 15 indicating whether Figure A-1 or A-2 should be used to assess the representative Fire Factor. For watersheds 0.1 to 3 sq. mi., a labeled point will be added to Figure A-1 suggesting a Fire Factor value, while for watersheds 3 to 200 sq. mi., intersecting lines and an unlabeled point will be added to Figure A-2. Review the indicated plot, identify a representative Fire Factor for the watershed, and input this value in the corresponding orange-shaded cell.

Next, compute the design sediment/debris yield using the worksheet titled "*Sediment-Debris Yield*" in the same Excel workbook. Input values in all orange-shaded cells, including the following:

Analysis Type: Select "Representative conditions" to indicate the type of analysis.

Relief Ratio, RR (feet per mile): The relief ratio should be determined by calculating the difference in elevation between the highest point in the watershed (measured at the end of the longest stream) and the lowest point (at the sediment/debris collection site) and dividing the difference between these two (in feet) by the maximum stream length (in miles) measured along the longest stream.

Maximum 1-hour Precipitation, P (inches) – If Indicated: For watersheds less than or equal to 3 sq. mi, the area-averaged 100-year, 1-hour precipitation should be identified using Atlas 14 precipitation frequency estimates from the NOAA Precipitation Frequency Data Server (PFDS):
https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=ca.

Peak Runoff (cfs) – If Indicated: For watersheds ranging from 3 to 200 sq. mi, the peak runoff should be evaluated using methods described in **Chapter 5**. In most cases, this will be a clear-water flow, unless the 100-year flow is determined based on a frequency analysis of gauged streamflow data.

Adjustment-Transposition (A-T) Factor: As described above, users must apply one of the four techniques described in Appendix B of Gatwood et al. (2000) to evaluate the A-T

Factor for the watershed and document the rationale and any computations in the study report.

Based on the input values, the spreadsheet tool will compute the total sediment/debris yield using the indicated equation from Gatwood et al. (2000) and the watershed area.

7.2.3.2. Bulking the Clear-Water Design Flow

A bulked design flow may be evaluated using either of two methods:

1. Select a standard, conservative bulking factor based on the watershed area and multiply by the clear-water hydrograph or computed peak flow
2. Apply the design sediment/debris yield to the clear-water hydrograph, then evaluate the resulting bulking factor at the flow peak

For watersheds where only a peak design flow is computed, or when preferred, a conservative bulking factor may be adopted for representative conditions without additional analysis:

1.2 for watershed areas > 3 sq. mi.

1.6 for watershed areas ≤ 3 sq. mi.

If a standard bulking factor is applied to a watershed with a design clear-water hydrograph for representative conditions, a standard spreadsheet (titled "*Clear-water Hydrograph*," in the same Excel workbook previously used to compute the design sediment/debris yield) can be used to input the basic identifying project information and the time-series hydrograph data. The timestep used should adequately capture the flow peak.

Next, use the worksheet titled "*Design Hydrograph*" to compute the bulked hydrograph. Input the selected bulking factor (i.e., 1.2 or 1.6) in the orange-shaded cell, then review the plotted bulked hydrograph and peak flow.

To apply the design sediment/yield to a clear-water hydrograph to develop a bulked hydrograph, first input the basic identifying project information and hydrograph time series data for the representative conditions in the standard spreadsheet titled "Clear-water Hydrograph."

Next, use the worksheet titled "*Sediment-Debris Distribution*" to apply the sediment/debris yield to the input clear-water hydrograph. Review the resulting bulking factor computed at the peak flow in Row 20. The bulking factor for representative conditions should be at minimum 1.2 and maximum 1.6. If the computed value is outside this range, the closest value (1.2 or 1.6) must be used to compute the peak design flow. In this case, use the worksheet titled "*Design Hydrograph*" to compute the bulked hydrograph. Input the selected bulking factor in the orange-shaded cell, then review the plotted bulked hydrograph and peak flow.

Figure 7-4 summarizes the recommended methods for analyses under representative watershed conditions.

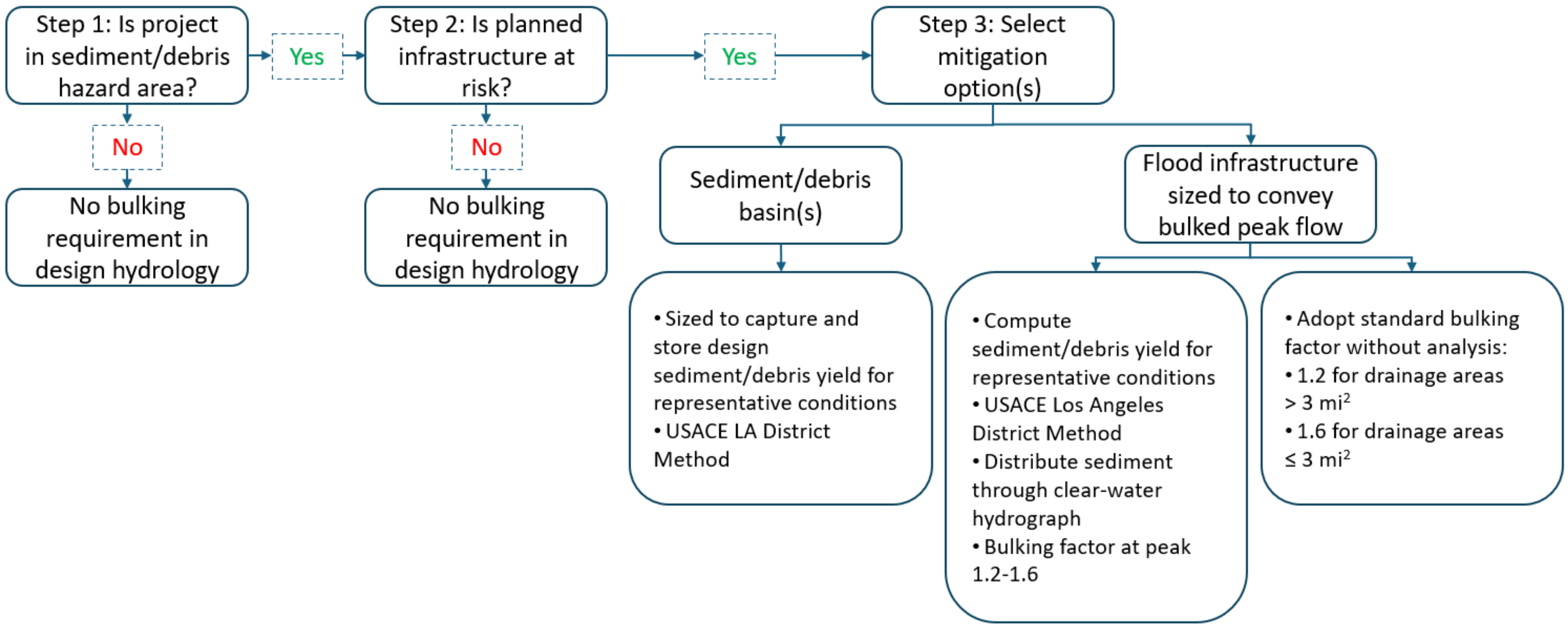

Figure 7-4. Flowchart Summarizing Methods for Representative Watershed Conditions

7.3. Addressing Sediment/Debris Hazards ≤ 10 Years Post-Fire

As discussed previously, wildfires can dramatically increase sediment/debris-risk conditions, and a watershed with a relatively low annual probability of debris flows—one that may not require consideration under representative conditions based on the methods in **Section 7.2**—can pose elevated risks to life, property, and critical infrastructure for approximately 10 years following a fire, relative to pre-fire conditions. When some or all of the study watershed burned during the past 10 years, sediment/debris hazards must be assessed for the current watershed condition, regardless of whether the watershed is located within or immediately downstream of an area of identified elevated risks under representative conditions.

Methods for 10 years or less post-fire are the same as those recommended for the pre-fire, representative conditions case, with the following exceptions:

- Rather than computing the Fire Factor (FF) based on the hypothetical average time since the last fire, the FF should be computed based on the actual time since the last fire(s) and the percentage of the watershed that burned.

  A standard spreadsheet (titled "*Fire Factor – Post-fire*," located in the same Excel workbook previously described) can be used to input information about the number, year, and size of the fire(s) that occurred in the past 10 years. Then, using the relevant Fire Factor curve (Figure A-1 or Figure A-2, also available as worksheets by the same name in the workbook), users can evaluate the FF for the year of the fire and each year since the fire(s).

  Note that the "Years-Since-100% Wildfire" labels on Figure A-1 and Figure A-2 are slightly misleading; a value of 1 "Year-Since-100% Wildfire" should be interpreted as the first year, which is the year of the fire. (Users may prefer to think of this as Year 0.) So, in Figure A-1, a 100% burned watershed returns to an unburned FF of 3.0 in the 10th year following the fire, but a total of 11 Fire Factors would be recorded for this period.

- For planning design in a burned area up to 10 years following a fire, when runoff and debris flow hazards have decreased substantially, bulking factors should be determined using the same methods described in **Section 7.2.3**, but with a lower limit of 1.25 (instead of 1.2) and an upper limit of 1.7 (instead of 1.6). Bulking factors determined by applying the sediment/debris yield to the 100-year clear-water, post-fire design flow should be constrained to values of 1.25 to 1.7.

- Alternatively, the following standard bulking factors may be used:

  1.25 for drainage areas > 3 sq. mi

  1.7 for drainage areas ≤ 3 sq. mi

Figure 7-5 summarizes the recommended methods for non-emergency post-fire analyses.

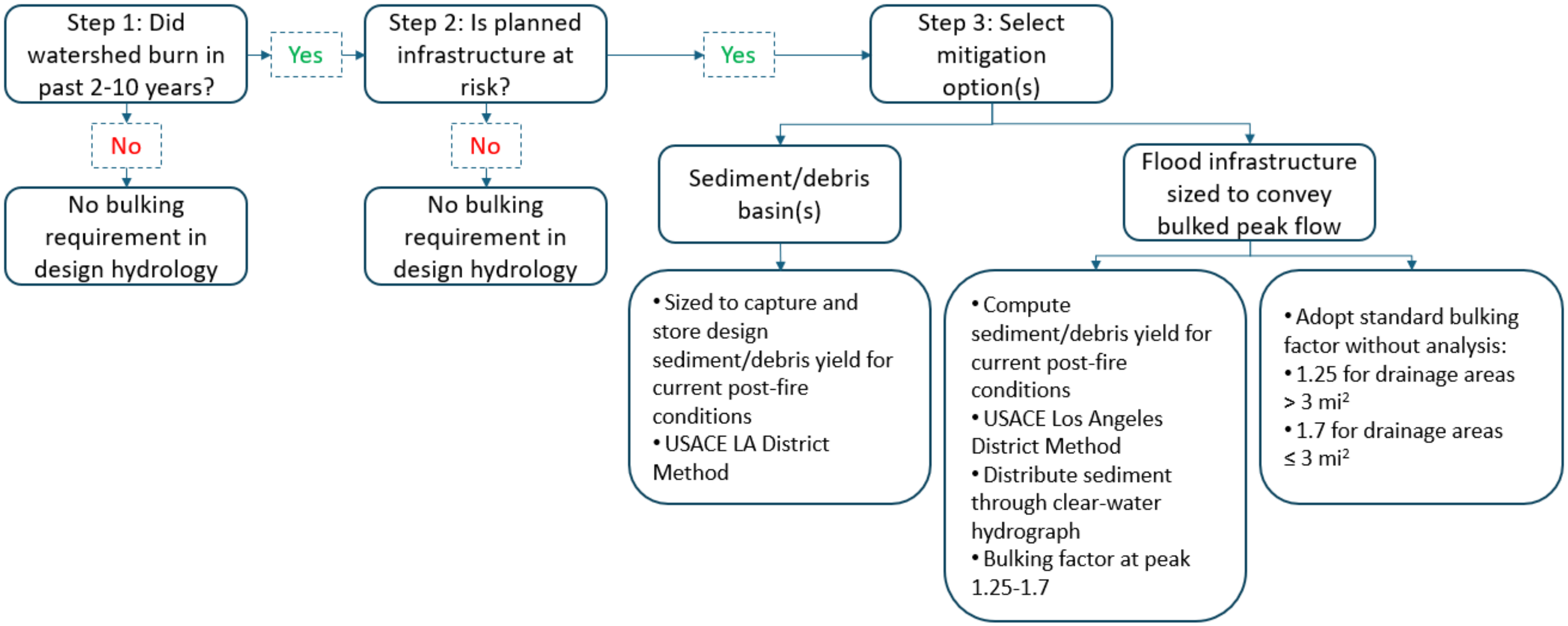

Figure 7-5. Flowchart Summarizing Methods for Non-Emergency Post-Fire Analyses

7.4. Sediment Bulking Example

The following example illustrates the methods prescribed for sediment bulking/debris hazard studies of watersheds under representative burn conditions and during the post-fire period. For this case, a hypothetical hydrologic watershed study performed in 2025 for an approximately 380-acre watershed located in the San Bernardino Mountains is considered; the step-by-step processes outlined in **Sections 7.2** and **7.3** are then followed.

7.4.1. Addressing Sediment/Debris Hazards for Representative Conditions

First, the potential sediment/debris hazards for representative conditions are assessed, following the process in **Figure 7-4**.

Step 1: Determine Whether Watershed is Potentially Affected by Sediment/Debris under Representative Conditions

Based on **Figure 7-6**, the watershed falls within the sediment/debris hazard area requiring analysis under representative conditions, so potential hazards must be assessed.

Step 2: Identify Whether Planned Infrastructure is at Risk

For this example, assume that the project is a culvert replacement on California State Route 38. Because the watershed drains directly to the planned culvert, the project is considered a value at risk of sediment/debris hazards.

Step 3: Select Appropriate Mitigation Option

Sediment/debris hazards can be mitigated using either of two methods: designing a sediment/debris detention basin to remove part or all of the design sediment/debris yield for the design flow or bulking the clear-water design flow. For the example, assume that a debris basin cannot be constructed and the clear-water design flow must be bulked. The bulking factor is assessed using both acceptable methods, for comparison purposes.

First, a standard bulking factor can be assessed. Because this watershed is smaller than 3 sq. mi, a conservative bulking factor of 1.6 can be assumed for representative conditions without any further analysis.

Alternatively, a sediment bulking factor can be evaluated by computing a design sediment/debris yield and applying it to the design clear-water hydrograph.

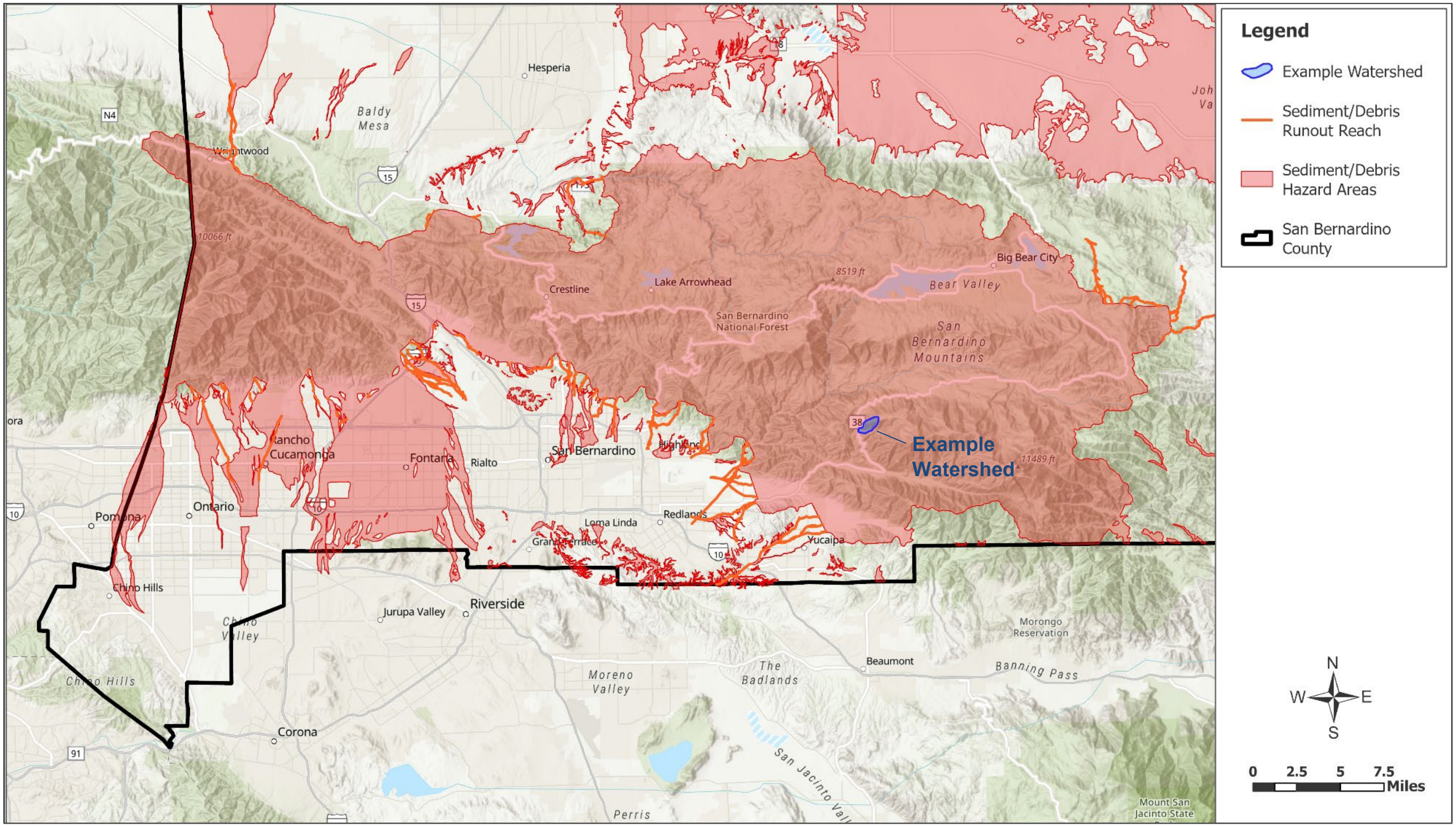

Figure 7-6. Example Watershed located in the Sediment/Debris Hazard Areas

Determine the Representative Fire Factor (FF) Using the Area-Averaged Burn Probability:

Based on GIS analysis using the annual burn probability raster dataset developed by Pyrologix, LLC (2023) and available at https://rrk.sdsc.edu/socal.html, the area-averaged annual burn probability for the project watershed is 0.016, or 1.6%. This corresponds to a representative fire interval of 1/0.016 = 62.5 years and an average of 31.25 years since the last fire—in other words, a recovered watershed.

Inputting the watershed area and annual burn probability values in the "*Sediment-Debris Yield and Bulking Tool," "Fire Factor – Representative"* sheet results in a plotted FF of 3.0, which is then input as shown in **Figure 7-7**.

Compute Fire Factor for Representative Conditions	
Watershed Area, A (Acres) :	380.0
Watershed Area, A (Sq. Mi.):	0.59
Annual Burn Probability (%):	1.6%
Representative Years-Since-100% Wildfire:	32.25
Representative Fire Factor, FF (-):	3.00

Use Figure A-1 to identify Fire Factor

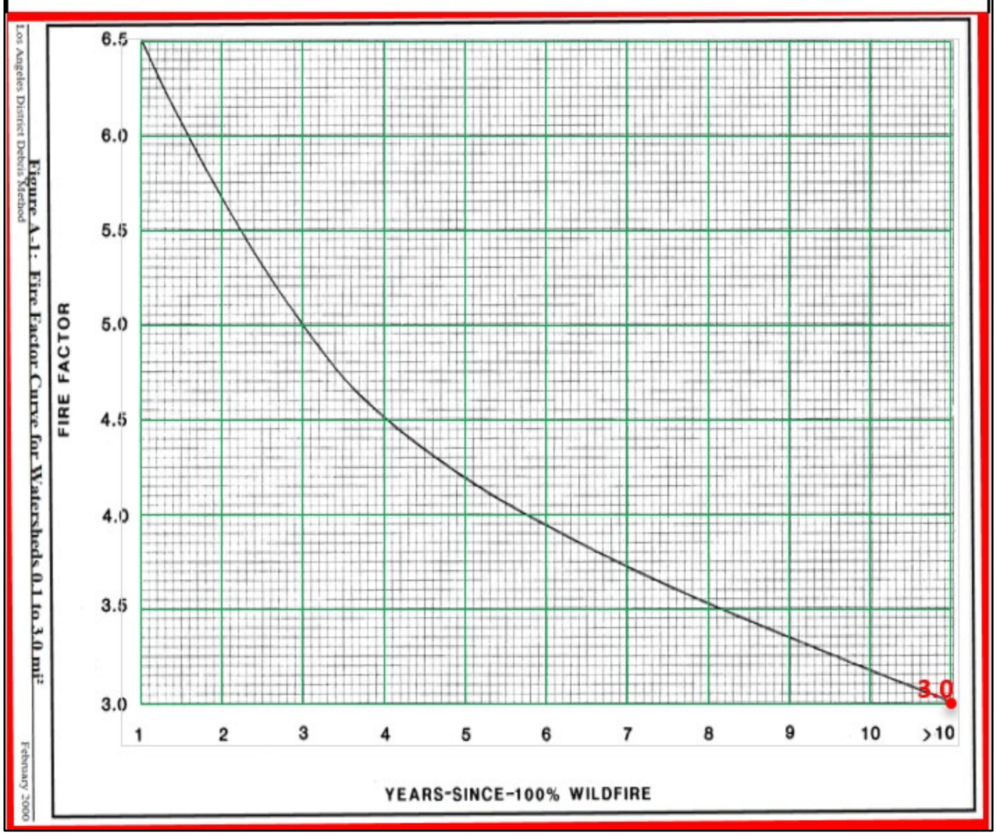

Figure 7-7. Determine Fire Factor – Representative Conditions

Calculate the Representative Sediment/Debris Yield:

Next, the sediment/debris yield for representative conditions is calculated using the "*Sediment-Debris Yield and Bulking Tool," "Sediment-Debris Yield"* tab. Values are input in all orange-shaded cells, including the following:

Analysis Type: "Representative conditions" selected

Relief Ratio, RR (feet per mile): The relief ratio is calculated using GIS. The highest point of the watershed is 7,798.8 feet, the lowest point of the watershed is 5,285.6 feet, and the longest streamline connecting these two points is 1.51 miles. Thus, RR = (7,798.8 feet − 5,285.6 feet) / 1.51 mi = 1,664 feet/mile.

Maximum 1-hour Precipitation, P (inches): Because the watershed is smaller than 3 sq. mi, the maximum 1-hour precipitation is used. An area-averaged 100-year, 1-hour precipitation depth of 2.12 inches was obtained using Atlas 14 precipitation frequency estimates.

Adjustment-Transposition (A-T) Factor: For this example, an A-T value of 1 is assumed. Typically, methods described in Appendix B of Gatwood et al. (2000) would be used to evaluate the A-T Factor for the watershed.

Note that the **Peak Runoff (cfs)** cell is shaded gray because the peak runoff is not needed for watersheds smaller than 3 sq. mi.

Inputting these values in the spreadsheet results in a total sediment/debris yield of 12,800 cubic yards, as shown in **Figure 7-8**. If a debris basin were to be used to mitigate the hazard, this volume would be used together with the methods described in **Chapter 8**.

Input Parameters and Compute Design Sediment/Debris Yield	
Analysis Type:	Representative conditions
Watershed Area, A (Acres):	380.0
Relief Ratio, RR (Feet/Mile):	1664.0
Fire Factor, FF:	3.00
Maximum 1-hr Precipitation, P (Inches):	2.12
Maximum 1-hr Precipitation, P (Inches x 100):	212
Peak Runoff (cfs):	459.0
Unit Peak Runoff, Q (cfs/Sq. Mi.):	773.1
Adjustment-Transposition Factor, A-T:	1.00
Use the Total Sediment/Debris Yield calculated below	
Eq. number from Gatwood et al. (2000):	1
Log(Dy)=0.65(Log(P))+0.62(Log(RR))+0.18(Log(A))+0.12(FF)	
Unit Sediment/Debris Yield, Dy (yd^3/Sq. Mi.):	21,557
Total Sediment/Debris Yield (yd^3):	12,800

Figure 7-8. Calculate Debris Yield – Representative Conditions

Use Sediment/Debris Yield to Develop Bulked Hydrograph:

To apply the design sediment/debris yield to the design clear-water hydrograph, the first step is to input the time series data for the clear-water hydrograph using the "*Sediment-Debris Yield and Bulking Tool," "Clear-water Hydrograph"* sheet. For this example, a hypothetical clear-water hydrograph with a 1-minute timestep is assumed, as shown in **Figure 7-9**. (Note: Only a portion of the input is shown.)

Summary of Input Clear-Water Hydrograph	
Peak Flow (cfs):	459.0
Time of Peak (Hour):	0.900
Total Volume (Cubic-Feet):	1,620,045
Total Volume (Acre-Feet):	37

Input Clear-Water Hydrograph Time Series Data	
Time (Hour)	**Clear-Water Flow Rate, Q_W (cfs)**
0.000	0.0
0.017	0.0
0.033	0.1
0.050	0.1
0.067	0.2
0.083	0.2
0.100	0.3
0.117	0.6
0.133	0.9
0.150	1.2
0.167	1.5
0.183	1.8
0.200	2.2
0.217	2.8
0.233	3.4
0.250	4.2

Figure 7-9. Clear-Water Hydrograph

Next, use the "*Sediment-Debris Yield and Bulking Tool," "Sediment-Debris Distribution"* sheet to apply the design sediment/debris yield to the clear-water hydrograph. Select a typical bulking exponent of 3.0, then review the computed bulking factor at the peak in Row 20. In this case, a peak bulking factor of 1.38 is computed, as shown in **Figure 7-10**, which falls within the standard range and reflects a bulked peak flow rate of approximately 635 cfs.

Sediment/Debris Data Inputs and Bulked Hydrograph Summary	
Watershed Area (Sq. Mi.):	0.59
Unit Sediment/Debris Yield, Dy (yd^3/Sq. Mi.):	21,557
Selected Bulking Exponent, *n* :	3.0
Peak Clear-Water Flow Rate (cfs):	459.0
Time of Peak (Hour):	0.900
Total Sediment/Debris Volume (yd^3):	12,800
Bulking Constant, *a* :	1.82E-06
Computed Sediment/Debris Volume (yd^3):	12,800
Peak Sediment/Debris Flow Rate (cfs):	176.1
Bulked Peak Flow Rate (cfs):	635.1
Computed Bulking Factor at Peak:	1.38

Use 1.38 as the bulking factor

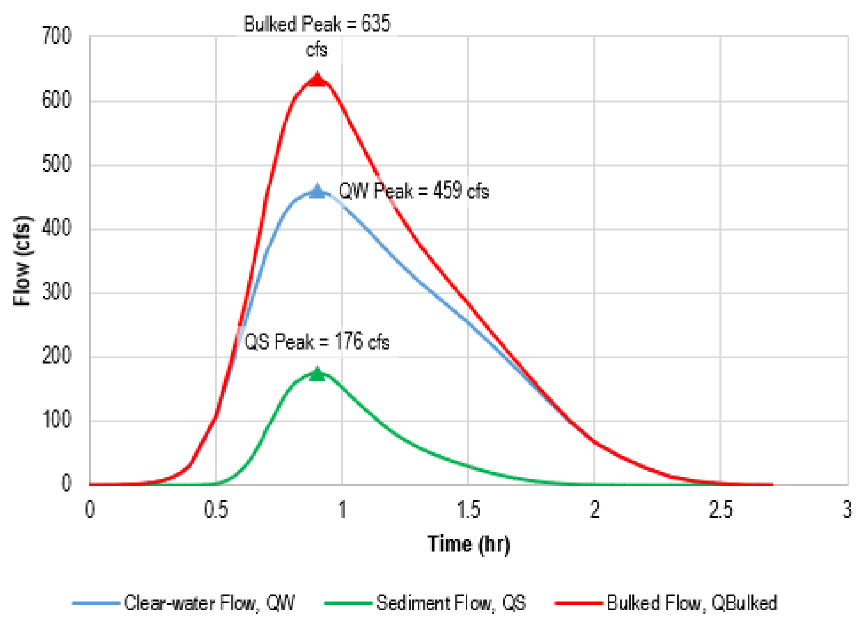

Figure 7-10. Sediment/Debris Distribution Calculation for Representative Conditions

If, for instance, a peak bulking factor less than 1.2 had been computed, a message in Row 21 would say, "Use 1.2 as the bulking factor." Using the "*Sediment-Debris Yield and Bulking Tool,*" *"Design Hydrograph"* sheet, a selected bulking factor of 1.2 would be entered, resulting in the bulked hydrograph shown in **Figure 7-11** with a peak bulked flow of approximately 551 cfs.

Input Bulking Factor and Review	
Selected Bulking Factor:	1.20
Peak Clear-Water Flow Rate (cfs):	459.0
Peak Bulked Flow Rate (cfs):	550.8

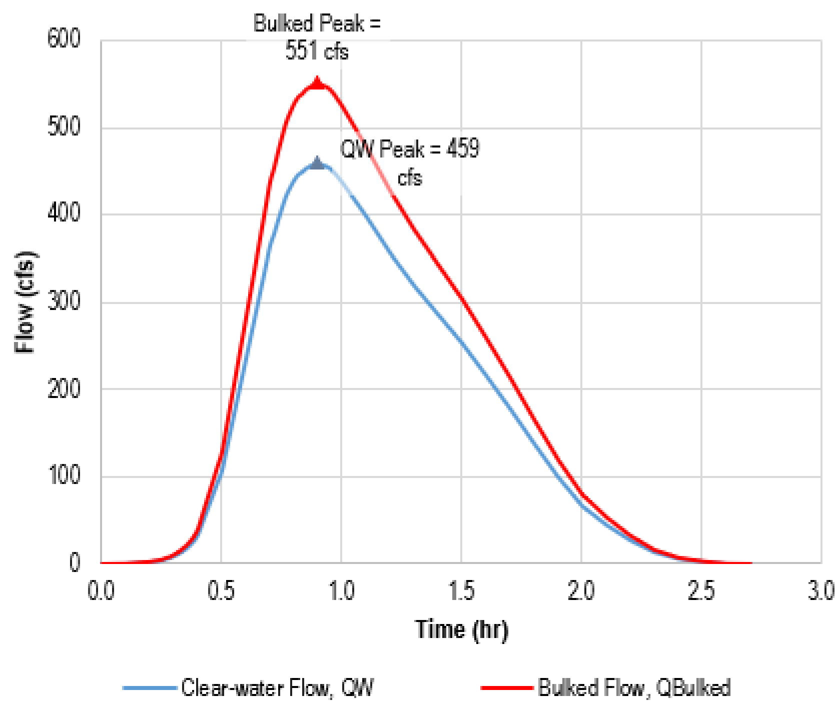

Figure 7-11. Bulked Hydrograph based on Selected Bulking Factor

7.4.2. Addressing Sediment/Debris Hazards ≤ 10 Years Post-Fire

Next, assess potential sediment/debris hazards for post-fire conditions, following the process in **Figure 7-5**.

Step 1: Determine Whether Watershed is Potentially Affected by Sediment/Debris under Post-fire Conditions

Based on **Figure 7-12**, which shows recent fire perimeters downloaded from CAL FIRE through January 2025, the last fire in the watershed was the 2020 El Dorado Fire and 100% of the watershed burned in the fire.

Step 2: Identify Whether Planned Infrastructure is at Risk

As described previously, the project is considered a value at risk of sediment/debris hazards.

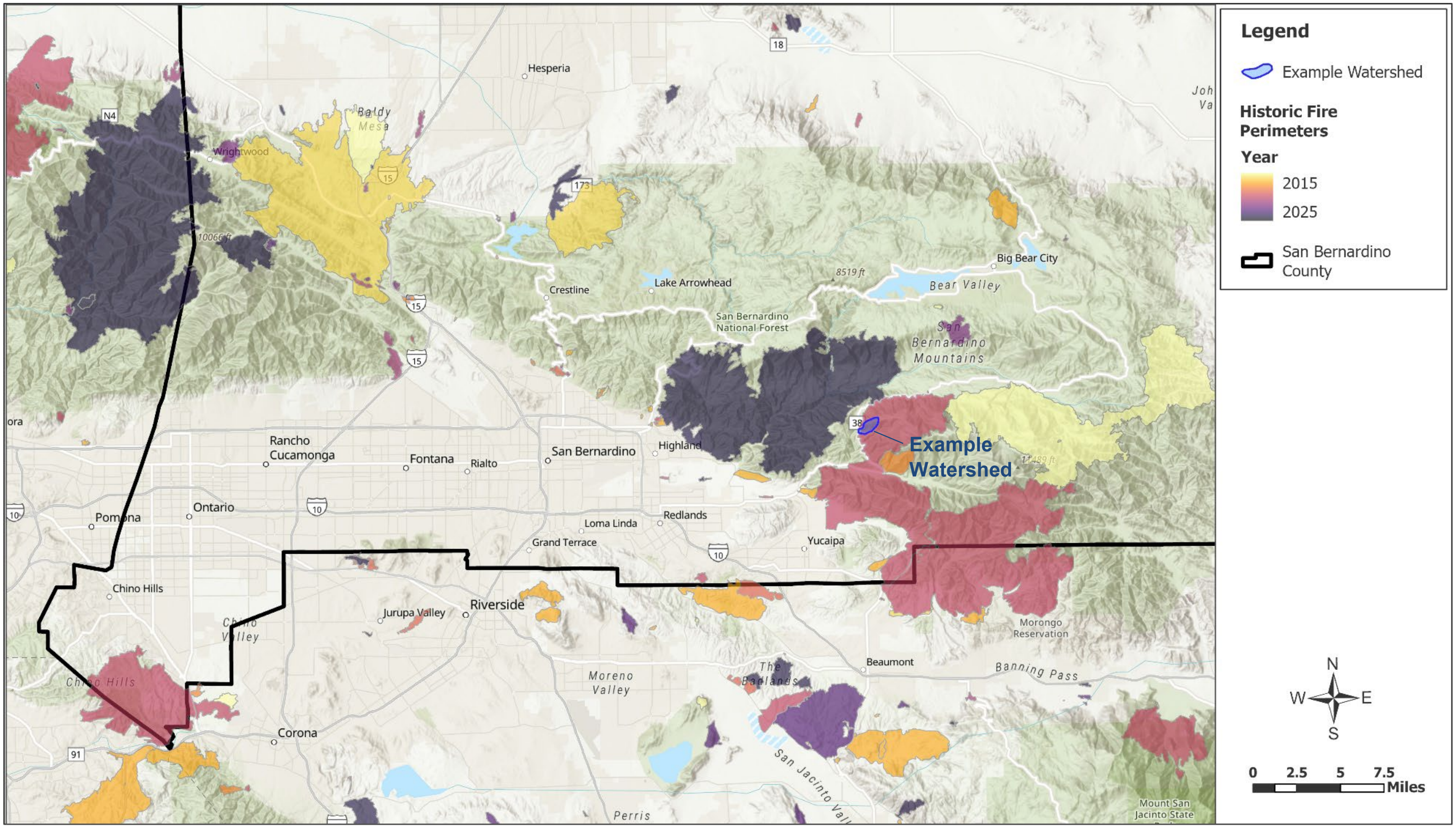

Figure 7-12. Example Watershed with Historical Burn Perimeters (2015–2025)

Step 3: Select Appropriate Mitigation Option

Next, evaluate the bulking factor under the current (post-fire) conditions.

First, a standard bulking factor can be assessed. Because this watershed is smaller than 3 sq. mi, a conservative bulking factor of 1.7 can be assumed for post-fire conditions without any further analysis.

Alternatively, a sediment bulking factor will be evaluated by computing a design sediment/debris yield and applying it to the design clear-water hydrograph.

Determine the Post-fire Fire Factor (FF) Based on the Most Recent Fire:

The "*Sediment-Debris Yield and Bulking Tool," "Fire Factor – Post-fire"* sheet helps users evaluate the watershed's FF based on the past 10 years of fire history. The example watershed burned only once in the past 10 years—and burned completely—during the 2020 El Dorado Fire. In Step 1, input the watershed area, then select the number of fires in the past 10 years from the dropdown list.

Next, input information about the El Dorado Fire in the remaining, orange-shaded cells, as shown in **Figure 7-13**. (If a smaller fire had occurred after the 100% El Dorado burn, additional cells would be shaded orange to indicate required information about that fire.)

Step 1: Input Watershed Burn History		
	Watershed Area (Acres):	**380.0**
	Watershed Area (Sq. Mi.):	0.59
	Number of Fires in the Past 10 Years?:	**1**
	Enter information about fire in last 10 years	
Fire Year	Fire Name	Burned Area (Acres)
2020	**El Dorado**	**380.0**

Figure 7-13. Determine Fire Factor – Post-Fire Step 1

In Step 2, note the figure that must be used (Figure A-1) and, reading values from the "*Sediment-Debris Yield and Bulking Tool," "Figure A-1"* sheet, input the FFs for the El Dorado burn area in the orange-shaded cells. Starting in 2020 (the year of the fire, considered Year 1), identify and input the FF for Years 2020 through 2025, as shown in **Figure 7-14**. If the watershed had burned multiple times in the past 10 years, cells in other columns would also be shaded orange and require separate FFs for those burn areas.

The spreadsheet tool computes the composite, weighted FF for the watershed. In this case, because the entire watershed burned in the El Dorado Fire and did not burn again, the Fire Factor for the El Dorado burn area is the same as that for the watershed: 3.94.

Step 2: Evaluate Fire Factor Over Past 10 Years										
	Computed FF for 2025:		3.94							
				Using Figure A-1, enter FF for the burned area for each year/fire in orange-shaded cells.						
Year	**Weighted FF**	**% Unburned**	**FF Unburned Area**	**% El Dorado Burn Area**	**El Dorado - FF**	**Leave blank; no 2nd fire**	**Leave blank; no 2nd fire**	**Leave blank; no 3rd fire**	**Leave blank; no 3rd fire**	
2015	3.00	100.0%	3.00	0.0%		0.0%		0.0%		
2016	3.00	100.0%	3.00	0.0%		0.0%		0.0%		
2017	3.00	100.0%	3.00	0.0%		0.0%		0.0%		
2018	3.00	100.0%	3.00	0.0%		0.0%		0.0%		
2019	3.00	100.0%	3.00	0.0%		0.0%		0.0%		
2020	6.50	0.0%	3.00	100.0%	6.50	0.0%		0.0%		
2021	5.66	0.0%	3.00	100.0%	5.66	0.0%		0.0%		
2022	4.99	0.0%	3.00	100.0%	4.99	0.0%		0.0%		
2023	4.50	0.0%	3.00	100.0%	4.50	0.0%		0.0%		
2024	4.18	0.0%	3.00	100.0%	4.18	0.0%		0.0%		
2025	3.94	0.0%	3.00	100.0%	3.94	0.0%		0.0%		

Figure 7-14. Determine Fire Factor – Post-Fire Step 2

Calculate the Post-fire Sediment/Debris Yield:

Next, calculate the post-fire design sediment/debris yield using the "*Sediment-Debris Yield and Bulking Tool," "Sediment-Debris Yield"* sheet*.* This is the same sheet used for the representative conditions case and most of the values input in the orange-shaded cells are the same:

> **Analysis Type:** "Post-fire conditions" selected.
>
> **Relief Ratio, RR (feet per mile):** As before, the RR is 1,664 feet/mile.
>
> **Maximum 1-hour Precipitation, P (inches):** As before, the maximum 1-hour precipitation of 2.12 inches is used.
>
> **Adjustment-Transposition (A-T) Factor:** As before, an A-T Factor 1 is assumed.

As before, the **Peak Runoff (cfs)** value is not needed for watersheds smaller than 3 sq. mi.

Together with the Fire Factor of 3.94, the total computed sediment/debris yield is 16,596 cubic yards, as shown in **Figure 7-15**. (Compare this to the 12,800 cubic yards computed for the representative conditions case.) If a debris basin were to be used to mitigate the hazard, this volume would be used together with the methods described in **Chapter 8**.

Input Parameters and Compute Design Sediment/Debris Yield	
Analysis Type:	**Post-fire conditions**
Watershed Area, A (Acres):	380.0
Relief Ratio, RR (Feet/Mile):	1664.0
Fire Factor, FF:	3.94
Maximum 1-hr Precipitation, P (Inches):	2.12
Maximum 1-hr Precipitation, P (Inches x 100):	212
Peak Runoff (cfs):	459.0
Unit Peak Runoff, Q (cfs/Sq. Mi.):	773.1
Adjustment-Transposition Factor, A-T:	1.00
Use the Total Sediment/Debris Yield calculated below	
Eq. number from Gatwood et al. (2000):	1
Log(Dy)=0.65(Log(P))+0.62(Log(RR))+0.18(Log(A))+0.12(FF)	
Unit Sediment/Debris Yield, Dy (yd^3/Sq. Mi.):	27,951
Total Sediment/Debris Yield (yd^3):	16,596

Figure 7-15. Calculate Debris Yield – Post-Fire Conditions

Use Sediment/Debris Yield to Develop Bulked Hydrograph:

In some cases, the post-fire clear-water peak flows may be larger than the equivalent clear-water flows developed for representative conditions, as described in **Chapter 6**, so the appropriate clear-water hydrograph should be entered using the "*Sediment-Debris Yield and Bulking Tool,*" "*Clear-water Hydrograph*" sheet. For simplicity, assume the same clear-water hydrograph described in **Section 7.4.1** for this example.

Following the same method used for representative conditions, use the "*Sediment-Debris Yield and Bulking Tool,*" "*Sediment-Debris Distribution*" sheet to apply the post-fire sediment/debris yield to the clear-water hydrograph, as shown in **Figure 7-16.**

Sediment/Debris Data Inputs and Bulked Hydrograph Summary	
Watershed Area (Sq. Mi.):	0.59
Unit Sediment/Debris Yield, Dy (yd^3/Sq. Mi.):	27,951
Selected Bulking Exponent, *n* :	3.0
Peak Clear-Water Flow Rate (cfs):	459.0
Time of Peak (Hour):	0.900
Total Sediment/Debris Volume (yd^3):	16,596
Bulking Constant, *a* :	2.36E-06
Computed Sediment/Debris Volume (yd^3):	16,596
Peak Sediment/Debris Flow Rate (cfs):	228.4
Bulked Peak Flow Rate (cfs):	687.4
Computed Bulking Factor at Peak:	**1.50**

Use 1.5 as the bulking factor

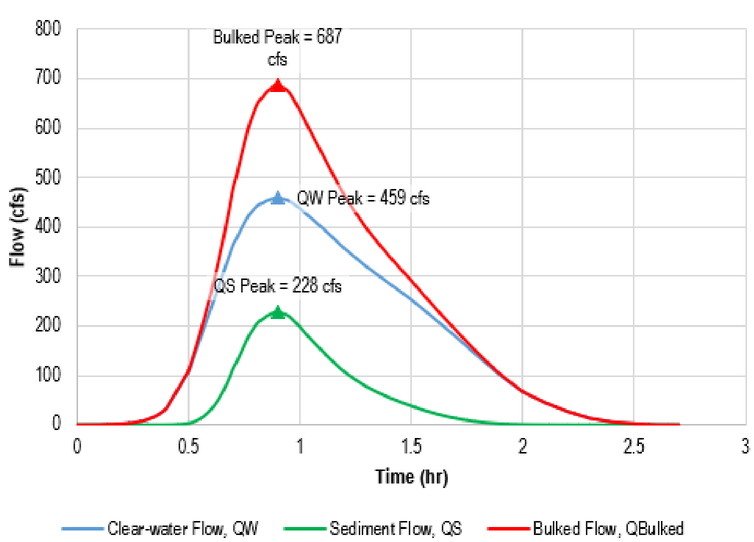

Figure 7-16. Sediment/debris Distribution Calculation for Post-Fire Conditions

The computed bulking factor at the flow peak is 1.5, which falls within the expected range, per **Section 7.3**.

If a peak bulking factor of less than 1.25 were computed, the "*Sediment-Debris Yield and Bulking Tool," "Design Hydrograph"* sheet would be used. A selected bulking factor of 1.25 would be entered, resulting in the bulked hydrograph is shown in **Figure 7-17** with a peak bulked flow of approximately 574 cfs.

Input Bulking Factor and Review	
Selected Bulking Factor:	1.25
Peak Clear-Water Flow Rate (cfs):	459.0
Peak Bulked Flow Rate (cfs):	573.8

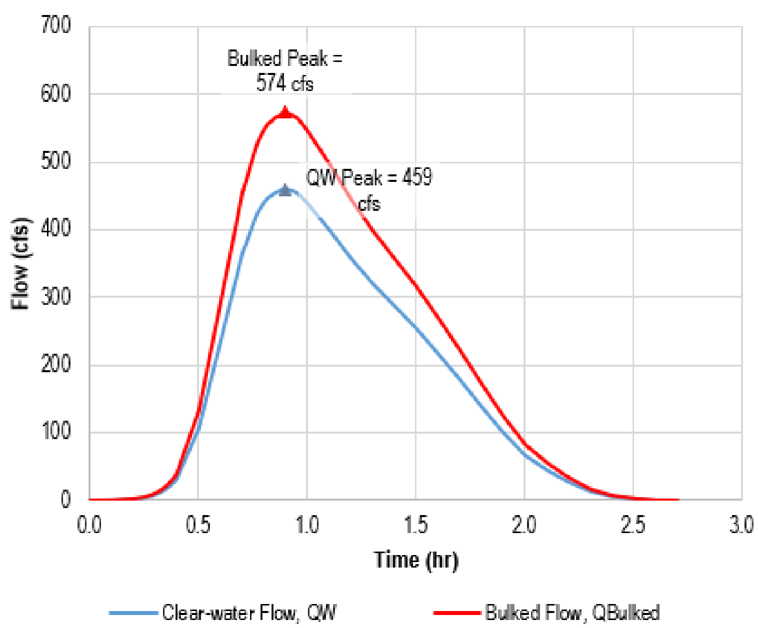

Figure 7-17. Design Hydrograph for Post-Fire Conditions

7.4.3. Summary of Example Results

Table 7-1 summarizes the resulting peak bulked flows for both representative and post-fire conditions, based on the detailed assessments. The table also includes the peak bulked flows computed using the standard, conservative bulking factors that could have been selected without performing a detailed assessment. In this example, the difference between the representative and post-fire peak flows reflects only the selected bulking factors, since the same clear-water hydrograph was assumed for both conditions.

Table 7-1. Summary of Flow Bulking Results

Watershed Conditions	Results Based on Detailed Assessment		Results Based on Standard, Conservative Bulking Factor	
	Peak Flow (cfs)	Bulking Factor	Peak Flow (cfs)	Bulking Factor
Representative	635.1	1.38	734.4	1.60
Post-fire	687.4	1.50	780.3	1.70

The larger of the two flows evaluated based on detailed assessment (687.4 cfs) is selected as the design bulked peak flow.

CHAPTER 8
DETENTION BASIN DESIGN

8.1. Detention Basin Considerations

Generally, the main purpose for inclusion of a stormwater detention basin in a flood control system is to reduce peak rates of runoff generated from an upstream watershed and to control peak flows into downstream areas. **Table 8-1** summarizes some of the advantages and disadvantages of detention basin use.

Table 8-1. Detention Basin Advantages and Disadvantages

Benefits	Potential Concerns
Reduce peak runoff rates to downstream areas.	Detention basins do not reduce total storm runoff volume (unless the groundwater recharge potential is large).
Basin reduces transport of sediments carried in floodwaters.	Ongoing maintenance of storage capacity, inflow and outflow facilities is critical.
Reduces size of downstream flood control facilities.	Basins increase the duration of flows, which may increase erosion effects downstream from the basin. Downstream erosion may be further increased due to sediment extraction in the basin.
Provides location for groundwater recharge if aquifer contact exists.	Improperly sized and placed basins may increase, rather than reduce downstream flooding potential (especially in large complex systems).
Provides location to concentrate floodwaters for treatment of contaminants.	Accumulated debris from runoff decreases flood control storage volume in a detention basin.
	Cost of debris removal and disposal.
	Detention basins in urban areas may become unsightly and/or vermin infested without ongoing maintenance.

The consideration of a detention basin system needs to address the various hydrologic, hydraulic, environmental, and flood control concerns listed above, as well as any other concern that may arise during the project study phase and determine the necessary mitigative measures that are acceptable to the County. Particular attention must be given to the interaction among the various elements of the overall drainage system. The uncoordinated placement of detention basins, without due consideration of existing or planned upstream and downstream detention facilities, as well as contributing tributary watersheds, may result in increased downstream peak discharges that exceed those projected under pre-development conditions without detention implementation.

8.2. Background

This chapter includes design parameters and procedures that serve as guidelines to ensure proper operations of detention basins. When necessary, these guidelines may be modified if approved in writing by the San Bernardino County Flood Control District (District).

8.3. Terminology

The following definitions are used in the discussion of detention basin design criteria:

Regional Detention Basin: A basin that is owned and operated by the District and can be part of its existing or proposed drainage system. The basin may be used jointly for other purposes. Its main role is to reduce the downstream peak flow rate, and the necessary downstream storm drain or channel size.

Local Detention Basin: A basin that is owned by an individual, organization, or municipality other than the District and is not part of the District's existing or planned drainage system. This type of basin reduces the downstream peak flow rate but will not be considered in downsizing future downstream storm drains, channels, or basins.

Joint Use Detention Basin: A regional or local detention basin that is used for a secondary purpose, such as a park, football field, parking lot, golf course, lake, etc.

Temporary Detention Basin: A local detention basin used to reduce downstream peak flow rates until ultimate storm drain facilities can be constructed as part of a phased development. The life of the basin shall generally not exceed 10 years.

Flow-Through Detention Basin: A detention basin designed so that runoff from the upstream watershed flows directly through the basin, typically passing through the basin's storage area before continuing downstream. The basin temporarily stores runoff to reduce peak flow rates, but it also allows continuous flow through it.

Flow-By Detention Basin: A detention basin located adjacent to a main channel or storm drain where runoff enters the basin only when the water level in the main channel or storm drain rises above a certain elevation. Under normal flow conditions, runoff bypasses the basin, and only higher flows are diverted into the basin for detention.

Debris Basin: A basin primarily designed to capture sediment, debris, and other transported materials originating from undeveloped upstream watersheds. Its purpose is to facilitate the design of downstream storm drains, channels, or detention basins based on clear-water flow, without incorporating a bulking factor. However, the application of a bulking factor downstream may still be warranted if the debris basin is not adequately sized to retain the total estimated debris generated by a 100-year storm event from the contributing watershed.

8.4. Detention Basin Design Concepts

8.4.1. Pre- and Post-Development Peak Flow Rates

When a regional, local, temporary, or joint use basin is required to mitigate downstream impacts due to increased flows from a development or when a basin is connected to a regional flood control facility, peak flow rates shall be calculated using the methodology outlined in this Manual, with the following conservative requirements to ensure adequate basin capacity, outlet size, stormwater management, and minimize downstream impacts:

1. Only 2-, 10-, 25-, and 100-year storms need to be analyzed.
2. The 10-year post-development peak flow shall be limited to 90% of the 5-year pre-development peak flow.
3. The 25-year post-development peak flow shall be limited to 90% of the 10-year pre-development peak flow.
4. The 100-year post-development peak flow shall be limited to 90% of the 25-year pre-development peak flow.
5. Additional studies shall be submitted where there exists more than one basin in the drainage area under review. The studies shall address the timing of peak flow rates from the basins to ensure that downstream flow rates are not increased.

All other independently constructed local retention and detention basins that have no current or planned future connection to a regional flood control district facility shall comply with the countywide policy.

8.4.2. Master Planned Downstream Drainage Facilities

When a regional or regional joint use basin is to be used to reduce the size of a master planned downstream drainage facility, the basin capacity and outlet size shall be such that the 100-year basin peak outflow rate does not exceed the downstream facility's design capacity.

The downstream drainage facility should be designed according to the following standards:

1. Open channel design capacities shall be per the San Bernardino County Flood Control District Standard. A bulking factor is not necessary when the basin is designed to handle debris, and the downstream channel is lined.
2. Pressure flow closed conduits shall be designed such that the hydraulic grade line is below the ground or street surface. In those reaches where no surface flow will be intercepted (now or in the future), a hydraulic grade line that encroaches on or is slightly higher than the ground or street surface will be acceptable.
3. Non-pressure flow closed conduit capacities shall be based on a flow depth no greater than 0.8 times the conduit's diameter or height.

8.4.3. Downstream Erosion

Where downstream erosion is a concern, the duration of permissible flow velocities for all frequency storms shall not be substantially increased unless other forms of mitigation are provided. This can be accomplished by reducing the peak flow rate further than that required above. Refer to *Handbook of Hydraulics* by King and Brater, and *Open-Channel Hydraulics* by Chow for permissible flow velocities design.

8.4.4. Debris Considerations

When there is a potential for debris entering the basin, the basin capacity shall be increased or a desilting basin provided to accommodate the debris production generated from a 100-year storm, plus 20% due to maintenance uncertainties. Additional considerations include:

1. For all basins where a significant amount of debris accumulation is anticipated, a debris disposal area or areas shall be provided within a reasonable hauling distance.
2. The 100-year debris yield shall be based on **Chapter 7** procedures.
3. Local basins shall not be constructed in areas where there is a potential for debris entering the basin (i.e., locations where flows are directed to the basin by natural drainage courses or earth graded channels which handle flows from undeveloped watersheds). It is recommended that flows from the developed area be conveyed to the basin through a hard-lined facility, then the basin outlet discharges into a natural drainage course or earthen channel.
4. Local detention basins shall not be fed by natural drainage courses or earth channels with undeveloped watersheds greater than 0.5 sq. mi.
5. Regional detention basins with undeveloped watersheds shall generally be flow-by basins or have a separate debris basin (flow-through basin) upstream of the detention basin.

8.4.5. Outlet Drain

1. The outlet pipe for all basins except temporary basins shall be a minimum 24-inch reinforced concrete pipe (RCP) (1350-D minimum) for local basins and a minimum 36-inch RCP (1350-D minimum) for regional basins. The outlet pipe or conduit shall be encased with cut-off collars per the *Los Angeles Flood Control Design Manual – Debris Dams and Basins* or designed per “Section 10.21. Cut-and-Cover Conduit Details” of the Bureau of Reclamation’s publication *Design of Small Dams*.
   a. Reinforced concrete collars generally from 2 to 3 feet high, 12 to 18 inches wide, and spaced from 7 to 10 times their height shall be provided.
   b. All joints for pipes not encased shall be rubber gasketed.
   c. The pipe shall be capable of withstanding H20 live loads plus the applicable dead loads.
   d. Erosion control measures shall be provided at the outlet of the basin outlet pipe.
   e. Temporary basin outlet pipes may be a minimum 24-inch corrugated metal pipe (CMP), 12 gauge with seep rings. Design considerations shall be as stated above.
2. A metered outlet structure may be necessary to provide the necessary flow attenuation for all frequency storms. “V”-shaped weirs and notched weirs are preferred over other

alternatives because they do not plug with debris and trash as easily as other designs.

3. All detention basin outlets should be sized so the basin will drain within 24 hours after the basin reaches its 100-year peak depth/volume. If the basin does not drain in 24 hours, further studies using longer duration storms will be necessary. The basin storage volume (capacity) may need to be increased to accommodate subsequent storms.
4. Trash racks shall be provided at the inlet to the basin outlet structure(s).
5. Anti-vortex devices shall be provided where warranted.
6. A depth gauge shall be provided on the basin outlet structure to monitor debris deposition and basin operation.

8.5. Flow-Through Basin Analysis

8.5.1. Introduction

Two types of routing are important in watershed planning: reservoir routing and streamflow routing. This section covers reservoir routing using the Modified Puls routing method and **Chapter 5** presents streamflow routing. In addition to the hydrologic criteria established in this Manual, San Bernardino County's hydraulic design criteria must also be considered.

Chapter 5 of this Manual explains how to use the unit hydrograph methodology for watershed hydrologic studies. The approach develops a 24-hour duration synthetic critical storm pattern, which is composed of peak rainfall intensities for a specified return frequency (e.g., peak 5-minute, 30-minute, 1-hour, 3-hour, 6-hour, and 24-hour) nested together with the storm peak 5 minutes of rainfall defined to occur at storm time of hour 16.

Use of this synthetic storm pattern subjects a study watershed to all durations of storm events (for a given storm frequency) up to 24 hours. That is, the 24-hour critical storm pattern for floodwater detention basin studies will include all durations of critical storm events up to 24 hours. Therefore, in many cases only one design storm is necessary for analysis of the detention basin flood routing.

8.5.2. Detention Basin Analysis

8.5.2.1. Detention Basin Routing Procedure

The Modified Puls routing method may be used for detention basin routing studies. The basin routing relationships are based upon **Equation 8-1**:

Equation 8-1 Basin Routing

$$I - O = \Delta S/\Delta t$$

where:

I = Basin inflow rate (cfs)
O = Basin outflow rate (cfs)
ΔS = Change in basin storage volume during the time step (cubic feet)
Δt = Time step (seconds)

Equation 8-1 is approximated by replacing the variables I and O by an average value during the timestep using **Equation 8-2** and **Equation 8-3**.

Equation 8-2 Basin Inflow Approximation

$$I = \frac{(I_1 + I_2)}{2}$$

where:

I_1 = The inflow rate at the beginning of a time period (cfs)
I_2 = The inflow rate at the end of the subject time period (cfs)

Equation 8-3 Basin Outflow Approximation

$$O = \frac{(O_1 + O_2)}{2}$$

where:

O_1 = The outflow rate at the beginning of a time period (cfs)
O_2 = The outflow rate at the end of the subject time period (cfs)

Substituting **Equation 8-2** and **Equation 8-3** into the basin routing equation of **Equation 8-1** and rearranging terms gives **Equation 8-4**:

Equation 8-4 Basin Routing Equation

$$(S_2 + O_2\Delta t/2) = (S_1 - O_1\Delta t/2) + (I_1 + I_2)\Delta t/2$$

where:

The right side of the equation is known from the previously computed values of storage, S_1, outflow, O_1, and the average basin inflow $(I_1 + I_2)/2$ for time step, Δt.

The solution to the basin routing problem requires the following information:

1. Known initial conditions for basin storage and outflow.
2. A routing timestep, Δt.
3. The basin inflow hydrograph.
4. Basin volume vs. depth and outflow vs. depth relationships.

Although manual Modified Puls routing computations are possible, they are cumbersome and provide multiple opportunities for errors; consequently, using HEC-HMS or other County-approved software is recommended for detention routing calculations.

8.5.2.2. Multiple Day Design Storm

Due to the interaction of watershed size, time of concentration, percentage of peak discharge reduction, and basin volume, the critical storm duration is generally not known (in advance) for a watershed flood control system that includes one or several detention basins. As a result, the use of the 24-hour design storm may not be the "critical" storm for flow-through detention basin design purposes, and a longer duration design storm may be needed. **Figure 8-1** illustrates the multiple day design storm for a 2-day duration. Longer duration design storms are developed in a similar fashion.

The multiple day design storm uses the structure of **Figure 8-1** for all flow-through detention basin systems. Successive day storms are developed and added in front of (i.e., to the left) the previously developed design storm patterns until the detention basin system demonstrates no increase in the required basin volume due to the further extension of the design storm pattern. By increasing the basin outlet capacity, the critical duration can be reduced.

From **Figure 8-1**, it is seen that the multiple day design storm is constructed from an arrangement of rainfalls of identical x-year return frequency. That is, even though a 2-day or longer duration multi-day storm is being used to test the detention basin's level of flood protection, the multiple day design storm still contains no more than x-year rainfall depths for the extended duration. Each of the 24-hour storm patterns are constructed by scaling the peak 24-hour design pattern according to a ratio of the respective 24-hour precipitation values.

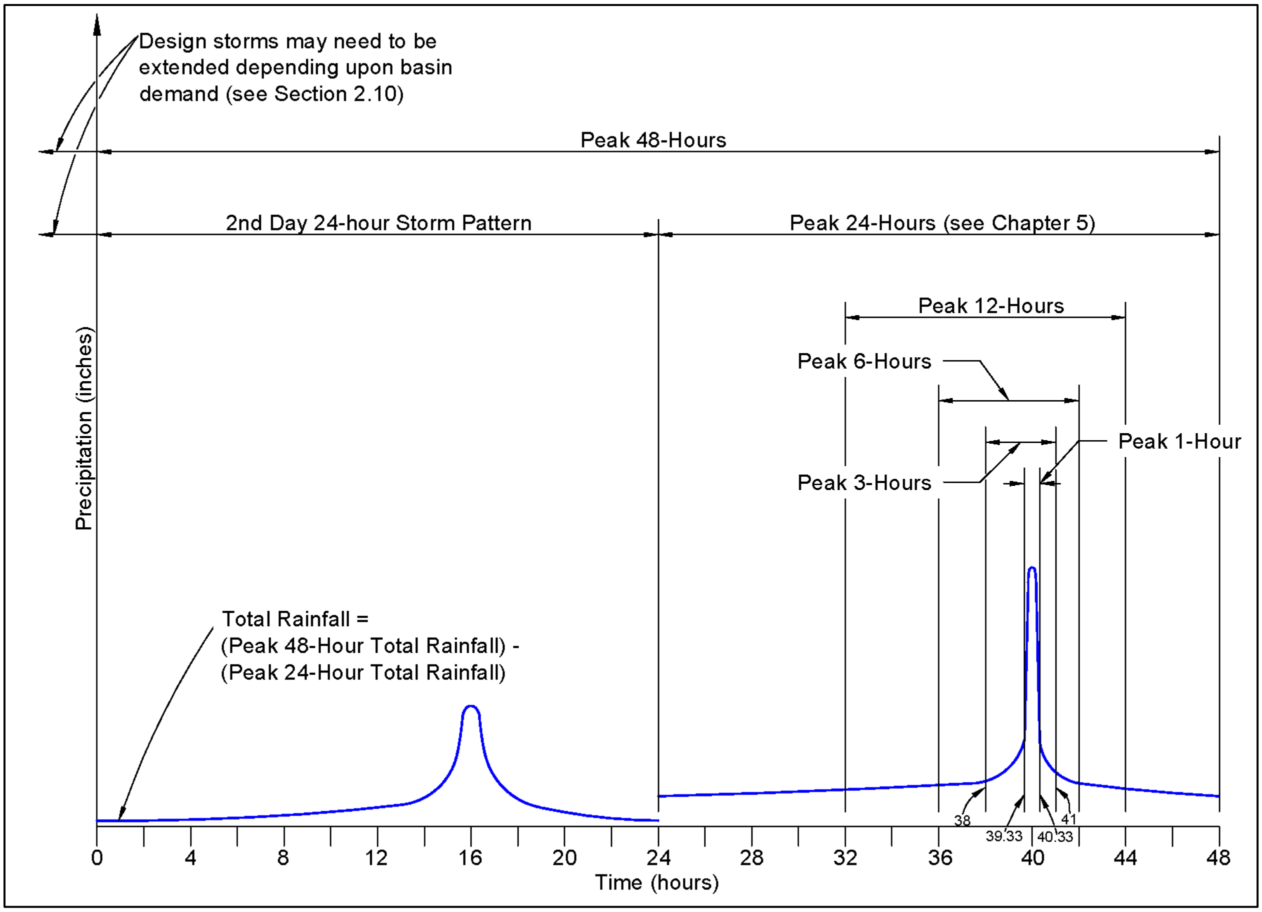

Figure 8-1. Multiple Day Design Storm Flow-Through Basin

8.6. Flow-By Basin Analysis (Hydrograph Separation)

8.6.1. Introduction

This process models the effect of a channel flowing by a detention basin or structure that intercepts and diverts flow away from the channel which exceeds a specified flowrate. **Figure 8-2** shows the main elements of a typical flow-by basin.

8.6.2. Multiple Day Design Storm Criteria

A multiple day storm may be required to guarantee that the basin has an adequate storage capacity remaining when a peak 24-hour storm event occurs. For a multiple day storm condition involving a flow-by basin system, the peak rainfall intensities of the selected x-year return frequency should be incorporated within each 24-hour duration (up to the appropriate mass rainfall volume with the desired return frequency) such as is shown in **Figure 8-3**.

8.6.3. Flow-By Basin Volume Analysis: Weir Structure Efficiency

Flow-by intercept weirs are installed along the side of a channel to divert storm runoff into a detention basin when the flow level in the main channel rises above the weir crest. **Figure 8-2** illustrates a typical intercept weir flow-by design.

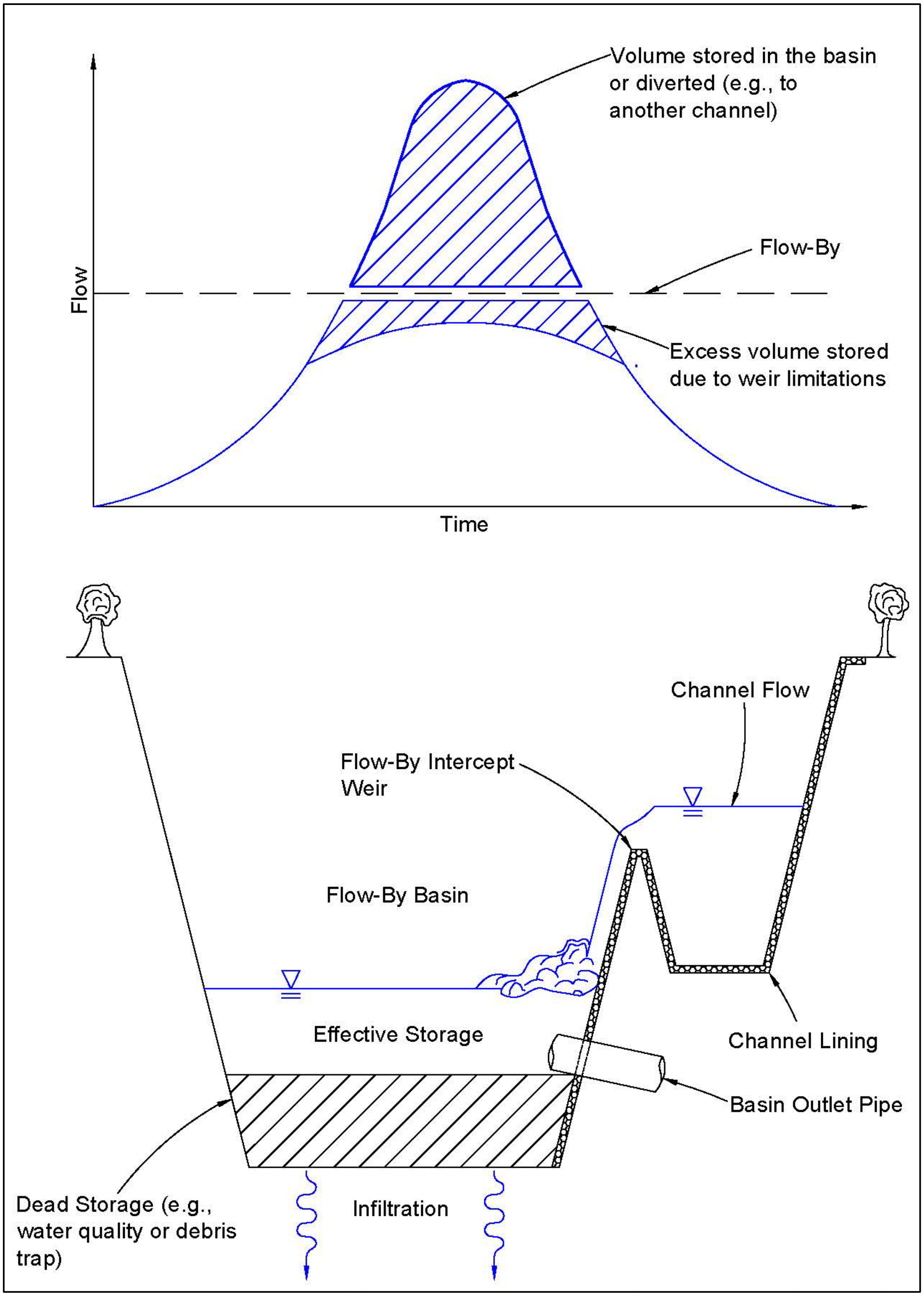

Figure 8-2. Flow-By Basin Concept and Elements

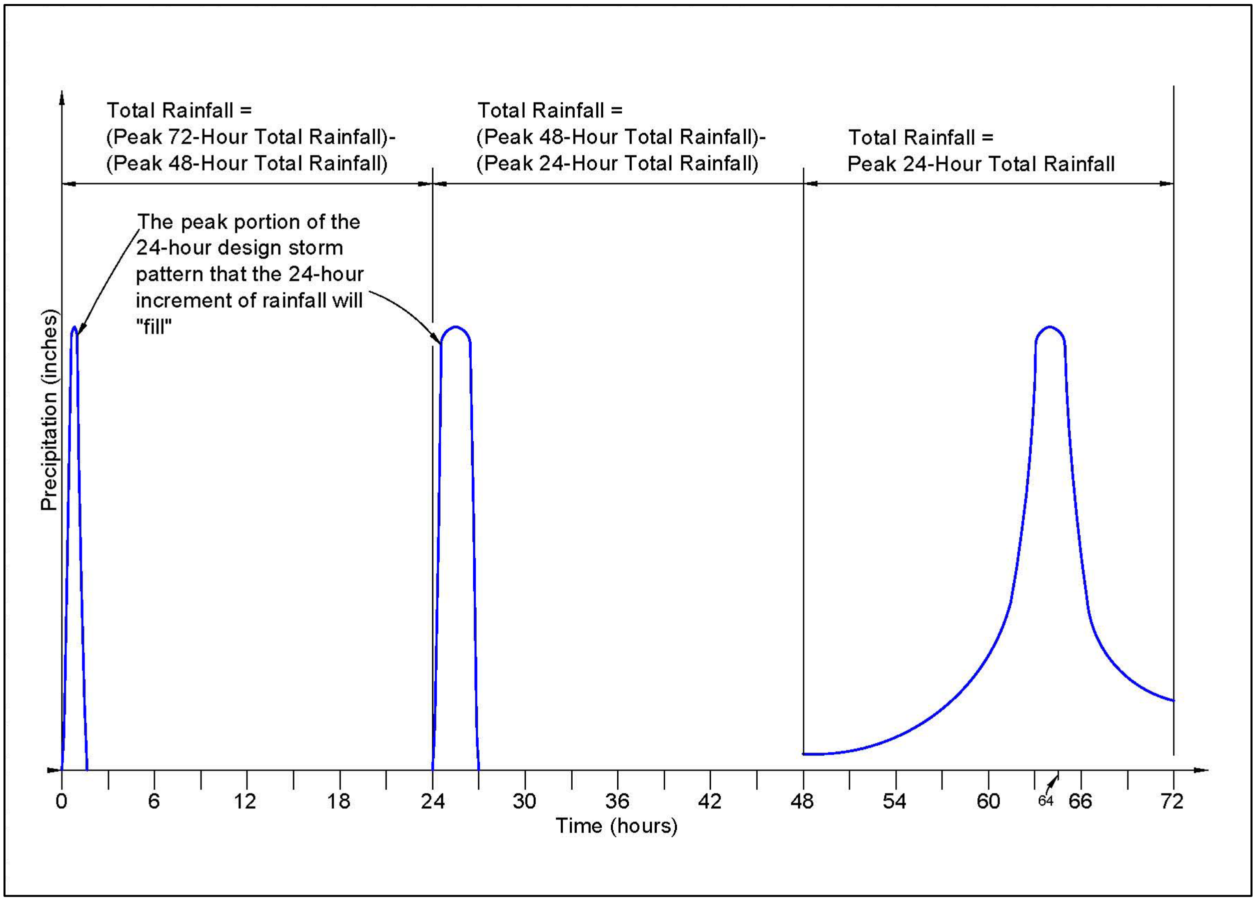

Figure 8-3. Flow-By Basin Critical Storm Pattern (3-Day Example)

8.7. Water Surface Elevation and Depth

8.7.1. Local Basins

1. When feasible, the 100-year design water surface elevation should be at or below existing natural ground. Generally, no more than 50% of the basin’s 100-year storage depth should be above existing ground (i.e., 50% or more of the 100-year minimum storage depth must be below the lowest ground elevation outside of the basin).
2. The necessary storage depth for debris plus the 2-year flow attenuation shall be below the existing ground.
3. The basin’s maximum 100-year water depth for design should be 6 feet or less.
4. When site conditions warrant and safety can be assured, the above depth requirements may be modified if the following conditions are met:
   a. The detention basin is designed in accordance with the Los Angeles County Flood Control District’s “Design Manual – Debris Dams and Basins.”
   b. The basin embankment is constructed of material, or has a solid core, which does not allow seepage or piping to occur due to rodent holes.
5. Local detention basins should not be constructed over impervious surfaces (e.g., pavement or concrete) to ensure that infiltration into the underlying soil can occur. However, for conservative design purposes, infiltration rates are not credited in the basin routing calculations.

8.7.2. Regional Basins

1. Depths shall be approved by the Flood Control District.
2. Basins with heights greater than or equal to 25 feet and capacity greater than or equal to 15 acre-feet, or a capacity greater than or equal to 50 acre-feet and a height greater than or equal to 6 feet, shall be reviewed and approved by the California Department of Water Resources Division of Safety of Dams (DSOD). (See **Figure 8-4**).

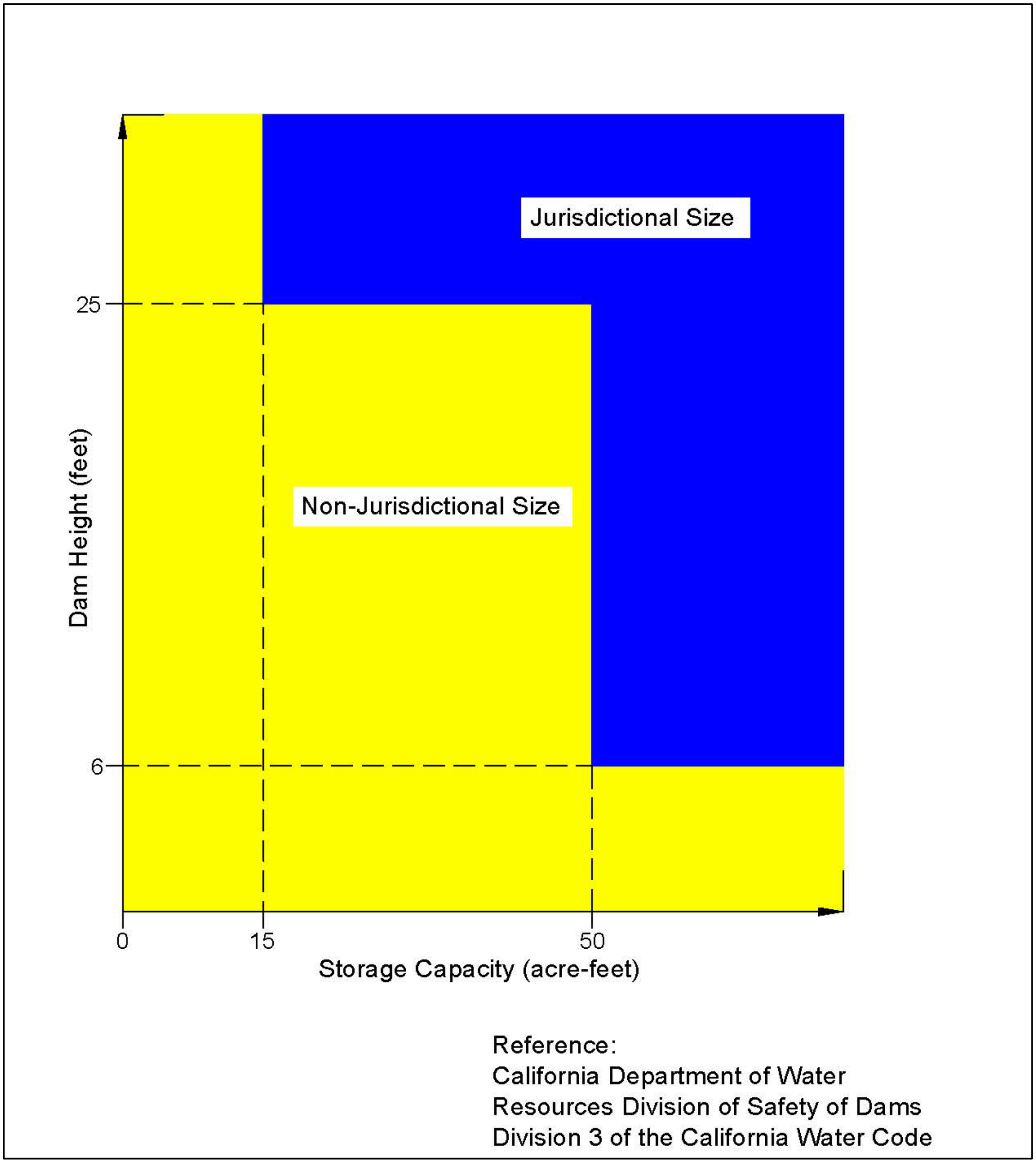

Figure 8-4. Division of Safety of Dams Jurisdictional Size Chart

8.7.3. Joint Use Basins

1. Depths should be shallow and compatible with the secondary use.
2. Depths for parking lots, tennis courts, or other similar joint use basins should be no greater than 6 inches to 12 inches.
3. The allowable depth in most cases will be site-specific and shall be approved by all agencies involved.

8.8. Emergency Spillway

1. All detention basin spillways shall be designed to pass the fully developed 1,000-year peak flow rate or that peak flow rate required by the State's DSOD, whichever is greater.
2. Spillway outflows shall be adequately conveyed to a storm drain, drainage channel, street, or an established watercourse.
3. Generally, all spillway structures shall be constructed of reinforced concrete. For temporary detention basins with an expected life less than 10 years, the spillway may be constructed with grouted rock or other forms of approved protection designed to resist maximum design velocities. However, when the spillway crest for a temporary detention basin is more than 3 feet above the flowline of the facility the spillway outlets into, the spillway shall be constructed of reinforced concrete.
4. Generally, the spillway crest shall be at or above the basin's 100-year design water surface elevations. However, in the case of a debris basin, this may not always be the case.

8.9. Freeboard to the Top of Embankment

1. Local and temporary basins shall have a minimum of 1 foot of freeboard above the 1,000-year High-Water Level (HWL) at the emergency spillway or 2 feet of freeboard above the 100-year HWL within the basin, whichever is more stringent.
2. Regional basins shall have a minimum 2 feet of freeboard above the 1000-year HWL on the emergency spillway. For basins with larger surface areas the freeboard shall be increased due to possible wave action. Also, a seismic seiche analysis shall be provided to determine the necessary freeboard.
3. Joint use basins shall conform to the applicable local or regional freeboard requirements. For smaller basins such as parking lot and tennis court basins, the freeboard conditions may be reduced.

8.10. Basin Embankment

1. Basin side slopes should be 3H:1V or flatter on the wet side and 2H:1V or flatter on the dry side. Steeper slopes may be acceptable on a case-by-case basis if they are rock-lined and recommended in the project soils and geotechnical report (see items 3 and 4 below for expanded requirements for report).
2. Levee top width.
   a. Regional and local basins: 20 feet minimum.
   b. Joint use: Site-specific.

   c. Refer to **Section 8.13** (3).

3. For design of the embankment abutments and adjacent slopes, a geological and geotechnical report shall be prepared by a geologist and geotechnical engineer with a demonstrated expertise in earth fill dam design. The report shall be reviewed and approved by the Flood Control District and shall include, but not necessarily be limited to:
   a. Site geology, including bedding, foliation, fracture, joint, fault, and landslide plane attitudes.
   b. Seismic conditions, including fault locations and potential seismic surface movements respective loadings and parameters of seismic shaking.
   c. Potential impact of reservoir loading on geologic structure should be evaluated.
   d. Detailed descriptions, locations, and logs of all field explorations.
   e. Field and laboratory tests and analysis descriptions and results.
   f. Groundwater table elevation and analysis of near surface groundwater movement.
   g. Recommended design parameters including, but not limited to, the following for the dam and its natural abutments and slopes adjacent reservoir area:
      i. Lateral earth loadings.
      ii. Shear strengths.
      iii. Bearing capacities.
      iv. Permeability.
      v. Slope stability analysis when saturated and during rapid drawdown conditions.
      vi. Sieve analysis.
      vii. Sand equivalents.
      viii. Liquefaction analysis and if proposed mitigation is required.
      ix. Scour analysis if required.
      x. Seismic Seiche Analysis (if applicable).
      xi. CBC Chapter 18A.
   h. Special design and construction recommendations including, but not limited to the following:
      i. Foundation preparation requirements. Including foundation evaluation of spillway and reinforced concrete box inlet/outlet structures.
      ii. Suitability of materials for embankments (gradation, sand equivalent, etc.) and abutments.
      iii. Compaction methods and minimum requirements.
      iv. Seepage and piping control provisions.
      v. Potential for settlement.
      vi. Seismic considerations.
      vii. Necessity of an impervious core or shear key.
      viii. Erosion control of abutments.
      ix. Minimum design factors of safety are listed in **Table 8-2** below.

Table 8-2. Minimum Design Factors

Design Factor	Without Seismic	With Seismic
Embankment, Abutment, and Adjacent Slope Stability	1.5	1.1
Seepage – Piping	1.5	---

4. Regional basins and local basins not meeting the depth and side slope requirements set forth previously shall be designed in accordance with the Los Angeles County Flood Control District's "Design Manual – Debris Dams and Basins."

8.11. Basin Floor

1. A low-flow channel shall be provided from the basin inlet(s) to the basin outlet,
   a. Where basin slopes exceed 2% or produce erosive flow velocities, the low-flow channel should be protected from erosion with reinforced concrete, rock lining, or other form of approved erosion protection.
   b. Joint use basins.
      i. A low-flow channel or conduit should be provided to conduct minor flows around the dual-use facilities wherever possible. Low-flow channels may not be necessary for parking lot basins or other similar joint uses.
      ii. Low-flow channels may be grass-lined if there exists a maintenance program which includes mowing and maintenance of turf in good condition and velocities of flow through the various stages of discharge are low enough to be nonerosive.
   c. Earth basin floors shall slope at a minimum 0.5% grade to the low-flow channel.
   d. Earth basin floors shall have a minimum grade of 0.5% from the inlet to the outlet.

8.12. Inlet Structures

1. Energy dissipators and/or erosion protection shall be provided where storm drains enter the detention basin. Plans must be approved by the Flood Control District if the basin is to be operated and maintained by the Flood Control District.
2. An invert stabilization measure such as a reinforced concrete spillway and/or energy dissipators shall be provided where natural drainage courses or channels enter the detention basin.
3. Energy dissipators may be required when the inletting flow velocities exceed 5 fps.
4. Inletting storm drains shall be a minimum 24-inch RCP (1350-D).

8.13. Access

1. Access to any type of detention basin area shall be provided by a roadway capable of handling two-way traffic (from a public street or public access to the parcel upon which the basin is constructed).
2. Access shall be maintained under all weather conditions.

3. A 20-foot-wide access roadway shall be provided along the top of embankment, across the spillway, and around the basin. The purpose of this requirement is to ensure continuous maintenance access to and around the basin. Where it can be demonstrated that the recommended top width is not required for structural integrity or maintenance purposes, the criteria may be modified, subject to approval by the Flood Control District.
    a. If access across the spillway is not provided, minimum 40-foot by 60-foot turnarounds shall be constructed on both sides of the spillway.
    b. If adequate maintenance access already exists, this requirement may be amended for local, temporary, or joint use basins.
4. Access ramps shall be provided to the basin floor, including any coffer dam invert areas constructed to separate water detention from the face of the dam.
    a. A minimum of one 15-foot-wide ramp shall be provided for local basins.
    b. A minimum of two 15-foot-wide ramps shall be provided for regional basins.
5. The maximum roadway or access ramp slope shall be 10%.
6. The minimum access and roadway inside turning radius shall be 35 feet.

Access Exemption: Local basins constructed independently shall comply with the local code and applicable policies.

8.14. Fencing

1. Regional basins:
    a. Shall be enclosed with 6-foot chain link fencing in accordance with San Bernardino County Flood Control District standards or as otherwise approved by the District.
    b. Chain link fencing shall include a 1-foot-wide horizontal orange stripe painted at the midpoint of the fence height.
    c. Access to the basins shall be gated and locked. Access ingress gates shall be set back a minimum of 40 feet from the edge of the road and shall open inward toward the facility.
2. Joint use basins shall be site-specific and must accommodate the needs of all agencies utilizing the basin.
3. Local basins constructed independently shall comply with the development code and all applicable standards.

8.15. Right-Of-Way

1. Adequate space must be provided for the construction and maintenance of the basin(s), including all fill and cut slopes. This should also encompass sufficient space for an access road connecting a dedicated public street to the basin. The acceptance of maintenance responsibilities will be evaluated on a case-by-case basis.
2. Regional basins shall be dedicated to the Flood Control District in fee title.
3. Local basins required to mitigate downstream impacts or connected to a regional flood control facility shall be covered by an adequate San Bernardino County Drainage

Easement with no maintenance obligation assigned to San Bernardino County or the Flood Control District.

4. All other independently constructed local basins that have no current or planned future connection to a regional flood control district facility shall comply with the local code and applicable policies.

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