



San Diego County Hydrology Manual

April 2026



Prepared by the County of San Diego
Department of Public Works
Flood Control Section

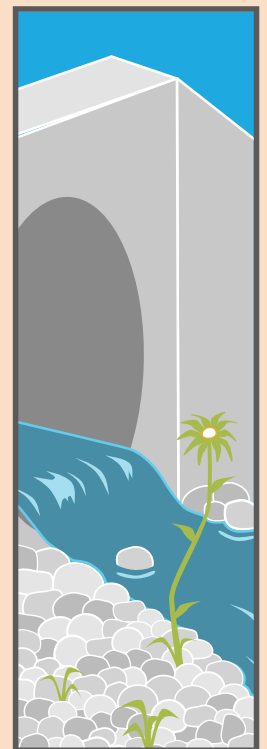
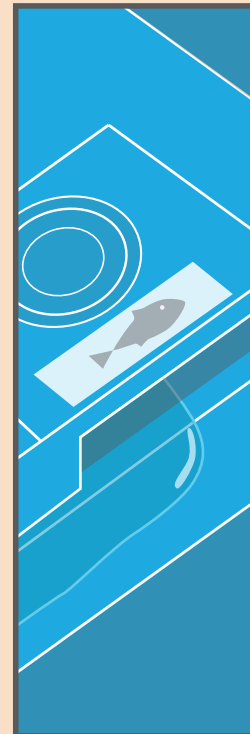
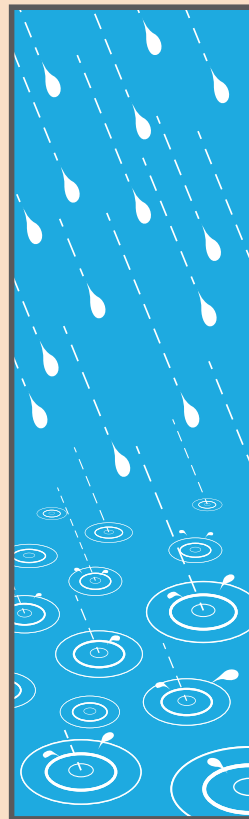
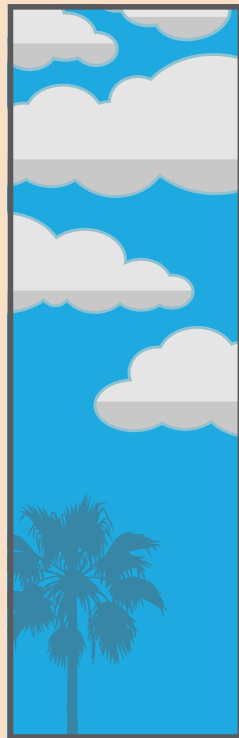


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*District is conterminous with the Board of Supervisor's District.

Original Manual

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The manual was based on analysis of rain and stream flow records collected in San Diego and information gathered from the National Resources Conservation Service, the National Weather Service, and the U.S. Army Corps of Engineers.

Current Manual

The current manual was developed by a technical advisory committee (TAC) under the direction of Flood Control District (FCD) Manager Sara Agahi and FCD Project Manager Ashkan Azarnia. Members of the TAC and their affiliation are listed below:

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The objective of this update was to incorporate the use of NOAA Atlas 14 rainfall data, sediment bulking factor recommendations, and the County’s new software tool, SDHydroTools, which can be used in preparation of hydrology calculations and studies.

ACRONYMS, ABBREVIATIONS, AND SYMBOLS

A	area
A _c	cross sectional area
A _s	soil loss in tons
ALERT	Automatic Local Evaluation in Real Time
AR	Atmospheric River
ARS	Agricultural Research Service
b	bottom width
BMP	best management practice
C	runoff coefficient
CASQA	California Stormwater Quality Association
C-factor	cropping management factor
CEQA	California Environmental Quality Act
cfs	cubic feet per second
CN	curve number
CN ₂	curve number adjusted to PZN = 2.0
CN ₃	curve number adjusted to PZN = 3.0
D	duration
d	normal depth
DPWFCS	Department of Public Works Flood Control Section
DU/A	dwelling units per acre
EPA	Environmental Protection Agency
FAA	Federal Aviation Administration
FEMA	Federal Emergency Management Agency
FIS	Flood Insurance Studies
fps	feet per second
ft	feet
ft ²	square feet
ft ³	cubic feet
GIS	Geographic Information System
H	head

H_{avg}	$(HB + HE) / 2$
HB	head at beginning
HDSC	Hydrometeorological Design Studies Center
HE	head at end
HEC	Hydrologic Engineering Center
HEC-HMS	Hydrologic Engineering Center Hydrologic Modeling System
hr	hour
I	rainfall intensity
I_a	Initial Abstraction
IECA	International Erosion Control Association
in	inches
$I_{(N)}$	rainfall intensity in inches per hour for a rainfall block N (used in the rational method hydrograph procedure)
L	watershed length
lb	pound
L_c	length to centroid
mi	mile
mi^2	square mile
MCB	Marine Corps Base
MRM	Modified Rational Method
MSCP	Multiple Species Conservation Plan
MUSLE	Modified Universal Soil Loss Equation
N	an integer
\bar{n}	basin factor
n	number of hydrograph ordinates
NCFR	Narrow Cold-Frontal Rainband
NFIP	National Flood Insurance Program
NOAA	National Oceanic and Atmospheric Administration
NRCS	Natural Resources Conservation Service
NWS	National Weather Service
OES	Office of Emergency Services
P	Total rainfall depth for a duration
P_6	precipitation over 6-hour period

P ₂₄	precipitation over a 24-hour period
P _(N)	incremental rainfall depth in inches for a rainfall block N (used in the rational method hydrograph procedure)
P _{T(N)}	rainfall depth in inches for the total duration T _{T(N)} for a rainfall block N (used in the rational method hydrograph procedure)
PFDS	Precipitation Frequency Data Server
PZN	Precipitation Zone Number
Q	discharge flow rate
Q _a	accumulated direct runoff in inches (over basin area)
Q _{AVG}	average flowrate originating from the entire basin
Q _(N)	peak flow rate in cubic feet per second for a rainfall block N (used in the rational method hydrograph procedure)
Q _t	accumulated volume of water at time t
q _i	discharge in cfs at the end of time interval i
q _{avg}	average flowrate in the gutter
q _p	interval peak discharge flow rate
Q _p	peak discharge flow rate
RM	Rational Method
RUSLE	Revised Universal Soil Loss Equation
S	potential maximum soil retention
s	slope
SanGIS	San Diego Geographic Information Source
SAVG	average slope
SCS	Soil Conservation Service
SDHydroTools	a computer program provided by the County for use with this manual
T _b	time of base
T _c	time of concentration (minutes)
T _i	initial time of concentration
T _l	lag time
T _p	time to peak
T _t	travel time
T _{T(N)}	total duration in minutes for a rainfall block N (used in the rational method hydrograph procedure)

USACE	United States Army Corps of Engineers
USGS	United States Geological Survey
USLE	Universal Soil Loss Equation
V	velocity
VOL	volume of runoff
Z	slope ratio
ΔE	change in elevation
ΔP	change in precipitation
ΔS	change in storage
ΔT	change in time
ΣCA	weighted runoff coefficient

GLOSSARY

Accumulated rainfall - (aka Cumulative rainfall, Potential maximum runoff) The total volume of rain that falls on a particular area over a specified time.

Accumulated direct runoff - (aka Excess rainfall) The volume of the rain of a given storm that falls at intensities exceeding the infiltration capacity of the watershed.

Agency - Governing body tasked with reviewing projects for conformance with requirements.

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.

Barren - Areas with 15% or less of the ground surface covered by plants or vegetative litter. It includes rockland, eroded land, and shaped or graded land. Barren land does not include fallow land.

Base flow - The part of the discharge that enters a stream channel from groundwater.

Basin area - (aka Drainage area, Watershed) The area of land that contributes water to the stream.

Broadleaf chaparral - Areas 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.

Close-seeded legumes or rotated pasture - Areas of alfalfa, sweetclover, timothy, etc. (and combinations) either planted in close rows or broadcast. This cover may be allowed to remain for more than a year so that year-round protection is given to the soil. The land

treatments used with row crops are also used with cover, except for row treatments if the seed is broadcast.

Confluence - Location where two or more streams join together and subsequently continue as one stream.

Contoured fields - Fields farmed as nearly as possible on the contour. Contouring affects runoff and infiltration due to the surface storage provided by the furrows because the storage prolongs the time during which infiltration can take place. The magnitude of storage depends not only on the dimensions of the furrows but also on the land slope, crop, and manner of planting and cultivation. Planting small grains or legumes on the contour makes small furrows that disappear because of climatic action during the growing season. The contour furrows used with row crops are either large when the crop is planted and made smaller by cultivation or small after planted and made smaller by cultivation, depending on the type of farming.

Contractor - Person(s) or company, being a licensed contractor with the State of California, responsible for constructing the project in conformance with the plans and applicable construction regulations.

County - Refers to the County of San Diego.

Cumulative rainfall - see Accumulated rainfall.

Deposition - An increase in the channel bed elevation due to collected sediment.

Desiltation basin - Consists of a depression and piping system that is designed to contain and slow the flow of stormwater to a sufficient velocity enabling sediments to fall out of suspension so they are not transported downstream.

Dimensionless unit hydrograph - A hydrograph plotted in dimensionless units with respect to flow and time.

Direct runoff - (aka Excess rainfall) See accumulated direct runoff.

Discharge volume - Total volume under the outflow hydrograph.

Drainage area - (aka Watershed, Basin area)

Dryland pasture - Equivalent to annual grass.

Engineer - Registered Engineer who is designated as the Engineer-of-Work for the project being evaluated.

Erosion - Process by which soil particles are displaced and moved from one location to another by the actions of wind or water.

Erosion control plans - Detailed plans prepared by the Engineer that depict the schematic location of erosion control devices to be implemented with a project. These are typically attached to grading plans being reviewed by an Agency.

Evergreen - Land planted to evergreen trees including citrus and avocado orchards and coniferous plantings. The effectiveness of this kind of land use is in part determined by the tree, the litter, and the ground cover. In these groves, the ground cover may be legumes alone or annual or perennial grasses with or without legumes. The ground cover may be entirely litter if the tree canopy is sufficiently dense to produce a substantial quantity of fallen leaves or needles (see Table 4-5). As with deciduous orchards, management practices affect the runoff potential of evergreen orchards. Tables 4-3 and 4-4 will help identify the appropriate hydrologic condition for different areas of the plantings.

Excess rainfall - (aka Accumulated direct runoff)

Fallow - Fallow land is land plowed but not yet seeded or tilled. It is more effective than barren land in reducing storm runoff.

Frequency of peak discharges - The same as that of the rainfall intensity for the given time of concentration.

Good rotations - Generally contain alfalfa or other close-seeded legume or grass to improve tilth and increase infiltration. Their hydrologic effects may carry over into succeeding years after the crop is removed, though normally the effects are minor after the second year.

Hydrograph - A graph showing, for a given point on a stream, the discharge flow rate of water with respect to time.

Hyetograph - A graph showing, for a given watershed, increments of average rainfall during successive units of time during a storm.

Initial time of concentration - Time required for runoff to travel across the initial subarea from the most remote point to the point of interest.

Inlet time - Time required for the stormwater to flow to the first inlet in the system.

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. For hydrologic purposes, dryland pasture is considered as annual grass.

Junction - (aka Confluence)

Lag - Time to 50% of total discharge volume at point of interest.

Length of overland flow - Length between the farthest point in the subarea number 1 to the design point.

Meadow - Land areas with seasonally high water table, locally called cienegas. Principal vegetation consists of sod-forming grasses interspersed with other plants. The grass is continuously grown, protected from grazing, and generally mowed for hay.

Narrowleaf chaparral - Land on which the principal vegetation consists of diffusely branched evergreen shrubs with fine needle-like leaves such as chamise and redshank. The shrubs are usually widely spaced and low in growth. If the narrowleaf chaparral shrubs are dense and high, the land should be included with broadleaf chaparral cover.

Natural hydrograph - Hydrograph created from data collected with stream gauges or other in-field methods.

NOAA Atlas 14 - A publication of the United States Department of Commerce National Oceanic and Atmospheric Administration (NOAA) National Weather Service (NWS), NOAA Atlas 14 contains precipitation frequency estimates (rainfall data) for the United States. NOAA Atlas 14 Volume 6 contains precipitation frequency estimates for the state of California.

Open brush - Principal vegetation consists of soft wood shrubs, usually grayish in color. Examples include California buckwheat, California sagebrush, black sage, white sage, and purple sage. It also includes vegetation on desert facing slopes where broadleaf chaparral predominates in an open shrub cover.

Orchards, deciduous - Land planted to such deciduous trees as apples, apricots, pears, walnuts, and almonds. The ground cover during the rainy season alters the hydrologic response to storm rainfall. Ground cover may be annual grass or perennial grass with or without legumes. Occasionally legumes are used alone. Use CN values that apply to the land use or the kind and condition of cover during storm periods; for example, annual grass CN values for annual grass or grass legume ground cover. If orchards are kept bare by disking or through the use of herbicides, use fallow curve numbers. Under typical management practices, ground cover in orchards varies in vegetative density and, consequently, in effectiveness in reducing runoff. Only orchards with more than

75% of the ground surface protected by cover should be considered in good hydrologic condition. See Tables 4-4 and 4-5.

Overland flow - Surface runoff that occurs in the form of sheet flow on the land surface without concentrating in clearly defined channels.

Peak discharge - (aka Peak runoff) The maximum instantaneous rate of discharge at a given point or from a given area, during a specified period.

Peak runoff - (aka Peak discharge)

Perennial grass - Areas on which the principal vegetation consists of perennial grass, either native or introduced, and which grows under normal dryland conditions. Examples are Stipa or needle grass, harding grass, and wheat grass. It does not include irrigated and meadow grasses.

Point of interest - (aka Design point, Collection point, Concentration point) Point at which Q_p is calculated.

Poor rotations - Generally one-crop land uses such as continuous corn (maize) or continuous wheat or combinations of row crops, small grains, and fallow.

Potential maximum runoff - see Accumulated rainfall.

Precipitation Frequency Data Server (PFDS) - A point and click internet interface developed to deliver NOAA Atlas 14 precipitation frequency estimates and associated information. The server is operated by NOAA's National Weather Service (NWS) Hydrometeorological Design Studies Center (HDSC) and is accessible at <https://hdsc.nws.noaa.gov/hdsc/pfds/>.

Project - Proposed development area being analyzed by the Engineer for erosion potential.

Rainfall - (aka Precipitation) Precipitation in the form of water.

Rainfall distribution - The manner in which depth of rainfall varies in space and time.

Rainfall intensity - The amount or rate of precipitation collected in a unit time interval. For the purposes of the rational method calculations it is based on the time of concentration for a selected storm frequency.

Rotations - Planned sequences of crops, with the purpose of maintaining soil fertility or reducing erosion or providing an annual supply of a particular crop. Hydrologically, rotations range from “poor” to “good” in proportion to the amount of dense vegetation in the rotation, and they are evaluated in terms of hydrologic effects.

Row crop - Any field crop (maize, sorghum, soybeans, sugar beets, tomatoes, tulips) planted in rows far enough apart so that most of the soil surface is exposed to rainfall impact throughout the growing season. At planting time, it is equivalent to fallow and may be so again after harvest. In most evaluations, average seasonal condition is assumed, but special conditions can be evaluated as shown in Chapter 10, NEH-4 (U.S. Department of Commerce 1985). Row crops are planted either in straight rows or on the contour, and they are in either a poor or good rotation.

Runoff coefficient - The fraction of the rainfall that runs off of the surface.

Runoff hydrograph - The hydrograph constituted by the surface runoff.

Runoff volume - The total quantity or volume of runoff during a specified time.

Scour - Effect that moving water will have on soil, depending on velocity/momentum of water, soil particle size, and soil cohesion.

SDHydroTools - A computer program provided by the County for use with this manual. The SDHydroTools computer program is a tool provided free of charge to hydrology

manual users in an effort to streamline traditionally complex efforts associated with hydrologic calculations.

Sediment - Eroded material suspended in or transported by wind or water.

Sedimentation - Deposition of eroded material in any one place.

Small grain - Areas of wheat, oats, barley, flax, etc. are planted in rows close enough that the soil surface is not exposed except during planting and shortly thereafter. Land treatment is the same as used with row crops.

Storm frequency - (aka Design frequency) Frequency is the reciprocal of return period. A frequency of $1/T$, or one in T years, corresponds to a return period of T years. A 25-year storm is one of a magnitude that occurs on average every 25 years.

Straight-row fields - Fields farmed in straight rows either up and down the hill or across the slope. Where land slopes are less than about 2%, farming across the slope in straight rows is equivalent to contouring.

Subarea - (aka Subbasin, Tributary area) The total area, projected upon a horizontal plane, of a drainage basin bounded by the basin perimeter and contributing overland flow to the stream.

Synthetic hydrograph - A hydrograph derived following an established formula, without the need for rainfall runoff data analysis.

Time of concentration - (aka Travel Time, Concentrated flow travel time, Time to peak) The time taken for a drop of water falling on the most remote point of a drainage basin to reach the outlet, where remoteness relates to time of travel rather than distance. Time to peak - The time from the beginning of runoff (on the rising side of the hydrograph) to the peak flow.

Travel time - (aka Transit time) Time required for stormwater to flow in a storm drain from the initial subarea to the point of interest.

Triangular hydrograph - A triangular hydrograph having the same volume as the curvilinear hydrograph.

Triangular unit hydrograph - A triangular hydrograph having the same volume as the curvilinear unit hydrograph.

Tributary - see Subarea.

Turf - Golf courses, parks, cemeteries, and similar lands where the predominant cover is irrigated, mowed, close-grown turf grasses. Parks in which trees are dense may be classified as woodland.

Unit hydrograph - The direct runoff hydrograph resulting from a rainfall event that has a specific temporal and spatial distribution and that lasts for a specific duration of time (thus there could be a 5-, 10-, 15-minute, etc., unit hydrograph for the same drainage area). The ordinates of the unit hydrograph are such that the volume of direct runoff represented by the area under the hydrograph is equal to 1 inch of runoff from the drainage area.

Vineyards - As with orchards, ground cover and land condition must be considered in estimating CN numbers. Use CN number which applies to the land use or the kind and condition of cover during storm periods: for example, use annual grass CN number for land with this ground cover. CN numbers for “disked” and “annual grass” cover crops are two examples tabulated. As with orchards, the quality of the cover must also be considered. See Tables 4-3, 4-4, and 4-5 for appropriate hydrologic conditions.

Watercourse distance - Length the water travels.

Watershed - (aka Basin, Drainage area)

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 single or in small clumps. Canopy density, the amount of ground surface shaded at high noon, is from 20 to 50%.

Woods (woodland) - Areas on which coniferous or broadleaf trees predominate. The crown or 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.

SECTION 1 INTRODUCTION

1.1 PURPOSE

The purpose of this manual is to provide a uniform procedure for flood and stormwater analysis within San Diego County. It provides a guide for policies and procedures based upon the science and data available to attain reasonable standardization of hydrology studies throughout the county. This manual is for general use within San Diego County and may be used by consultants, government agencies, resource agencies, planners, engineers, attorneys, and developers. This manual does not set policy outside of the County's jurisdiction. Please note that each city in the County has jurisdiction over projects within its area and may have policies and procedures differing from those in this manual.

Flood management hydrology deals with estimating flow peaks, volumes, and time distributions of storm runoff. The prediction of these characteristics is fundamental to the design of stormwater management facilities. Errors in the estimates may result in a stormwater management facility that is either undersized and fails to provide public safety and flood protection or oversized and costs more than necessary. On the other hand, the timing, distribution, and intensity of rainfall is a natural process subject to considerable variability. The science of hydrology attempts to make predictions based upon rainfall data and an understanding of the relationship between precipitation and runoff.

1.2 BACKGROUND

In the hydrologic analysis for a stormwater management facility, it must be recognized that there are many variable factors that contribute to discharge or runoff. Some of the factors that need to be recognized and considered on an individual site-by-site basis include the following:

- rainfall amount and storm distribution
- drainage area size, shape, and orientation

- ground cover and soil type
- land use
- slopes of terrain and stream(s)
- precipitation zone number condition (an index of watershed wetness from antecedent rains)
- storage potential (e.g., overbank, ponds, wetlands, reservoirs, channel)
- watershed development potential
- characteristics of the local drainage system

Thus, it is important to select a hydrologic procedure that reflects the actual physical situation encountered in the drainage area being considered. If local stream gauge data are available, these data can be used to calculate peak discharges and hydrographs. The user is referred to standard hydrology textbooks for statistical procedures that can be used to estimate design flood events from stream gauge data.

The criteria in this manual are based on a substantial review of many methods used in the United States. The procedures deemed most reliable have been incorporated herein and have been correlated with recorded hydrologic data and analysis of rainfall and runoff events that have occurred within the county. San Diego County is located between Marine Corps Base (MCB) Camp Pendleton and Orange and Riverside Counties on the north, the United States/Mexico international border on the south, Imperial County on the east, and the Pacific Ocean on the west. San Diego County encompasses approximately 4,260 square miles and includes a variety of topography, soil types, land uses, and climate, which affects rainfall and storm events. The Natural Resources Conservation Service (NRCS) has mapped San Diego County, and much of this manual has been based on the NRCS data.

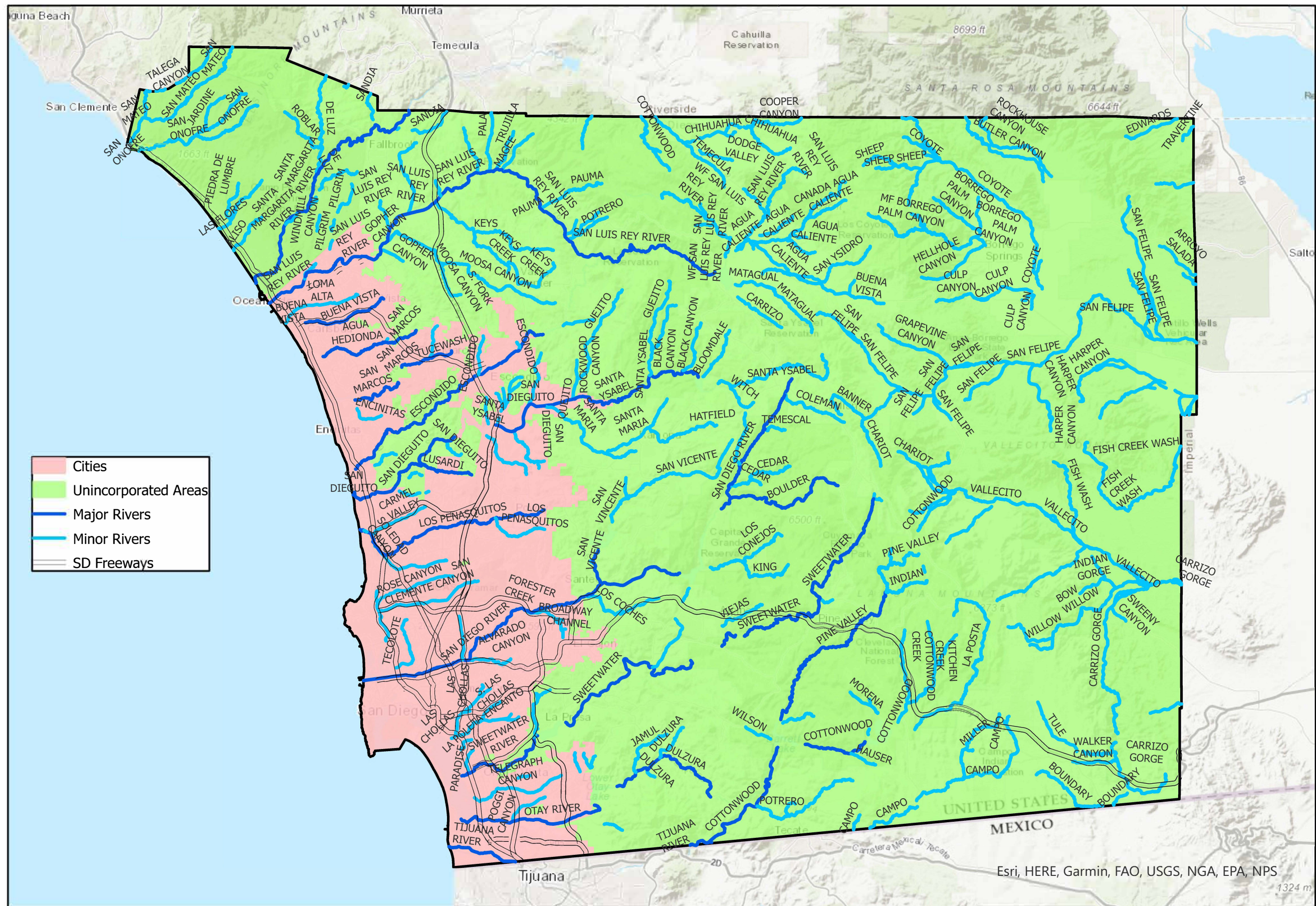
This manual references rainfall data from NOAA Atlas 14. A publication of the United States Department of Commerce National Oceanic and Atmospheric Administration (NOAA) National Weather Service (NWS), NOAA Atlas 14 contains precipitation frequency estimates (rainfall data) for the United States. NOAA Atlas 14 Volume 6 contains precipitation frequency estimates for the state of California.

1.2.1 Seasonal Precipitation Patterns of San Diego County

San Diego is an area of great climatic variation. The Salton Sea Basin and Pacific Basin divide separates the western and eastern watersheds of San Diego County. This divide follows the mountain ridgeline with elevations that vary from 3,000 to 6,000 feet above sea level. Precipitation that falls east of the divide flows to the Salton Sea Basin, while runoff from precipitation west of the divide flows down the western slope to the Pacific Ocean. The map of San Diego County, Figure 1-1, shows the major rivers within San Diego County.

Major & Minor Rivers in San Diego County

Figure 1-1



1.2.1.1 Primary (Cool-Season) Rainfall: October through April

Most of the precipitation in San Diego County occurs during the cool-season "rainy season," which typically lasts from October to April. During this time, cold frontal systems from the eastern Pacific generally move west to east across the region. The prevailing westerly winds push moist Pacific air from these weather systems up the Peninsular Ranges, which bisect the county. This process is known as orographic lift.

- **Windward (coastal & mountain) slopes:** As the air rises, it cools and condenses, leading to increased precipitation on the windward slopes (the coastal and mountain areas). This orographic effect often results in the highest rainfall totals occurring in the higher terrain of the county.
- **Leeward (desert) slopes:** After cresting the mountains, the air warms and dries on the descent, usually creating a pronounced *rain shadow* that sharply reduces precipitation in the desert region east of the range. *It's important to note that under rare circumstances, when exceptionally strong systems carry unusually high moisture, the rain-shadow effect can weaken or shift eastward, allowing substantially greater precipitation to reach the desert region.*
- **High-Elevation Snow:** Cool-season storms often drop the snow level to around 1300m (4,500 ft) and, on occasion, to 900m (3,000 ft) or lower, sometimes producing substantial accumulations of 6–12 in of snow or more.

The mean annual isohyets illustrated in Figure 1-2 and Figure 1-3 show this pattern of maximum precipitation on the windward side and deficits on the leeward side.

Although infrequent, specific weather patterns, such as cutoff low-pressure systems over Baja California or remnant tropical systems can advect moisture westward on easterly wind flow. These events reverse the usual pattern: deserts receive amounts typical of the coastal zone, while coastal areas see lighter, desert-like totals.

30 - Year Average Annual Precipitation Across San Diego County (1991-2020)

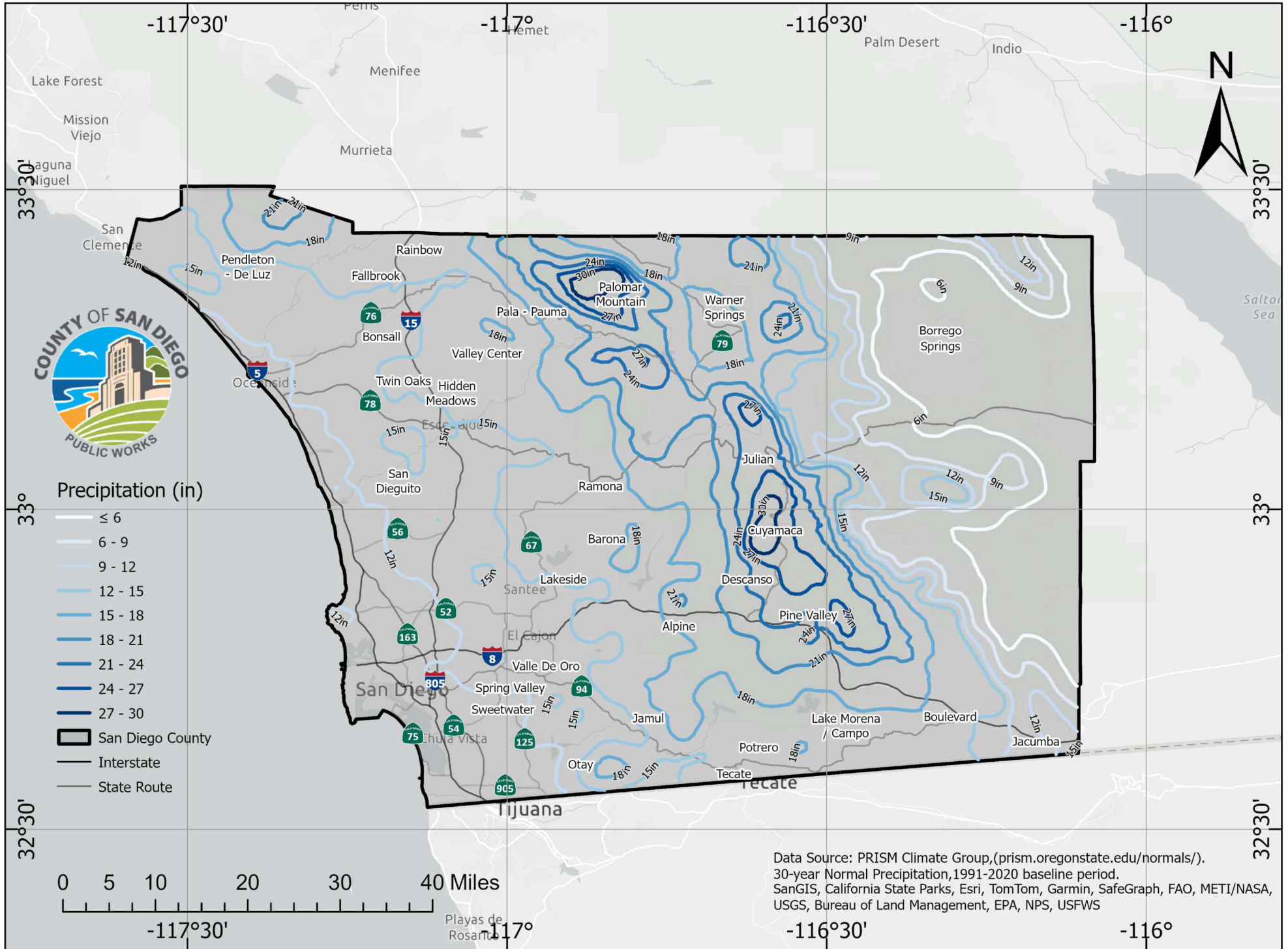


Figure 1-2

30 - Year Average Annual Precipitation Across San Diego County (1991-2020)

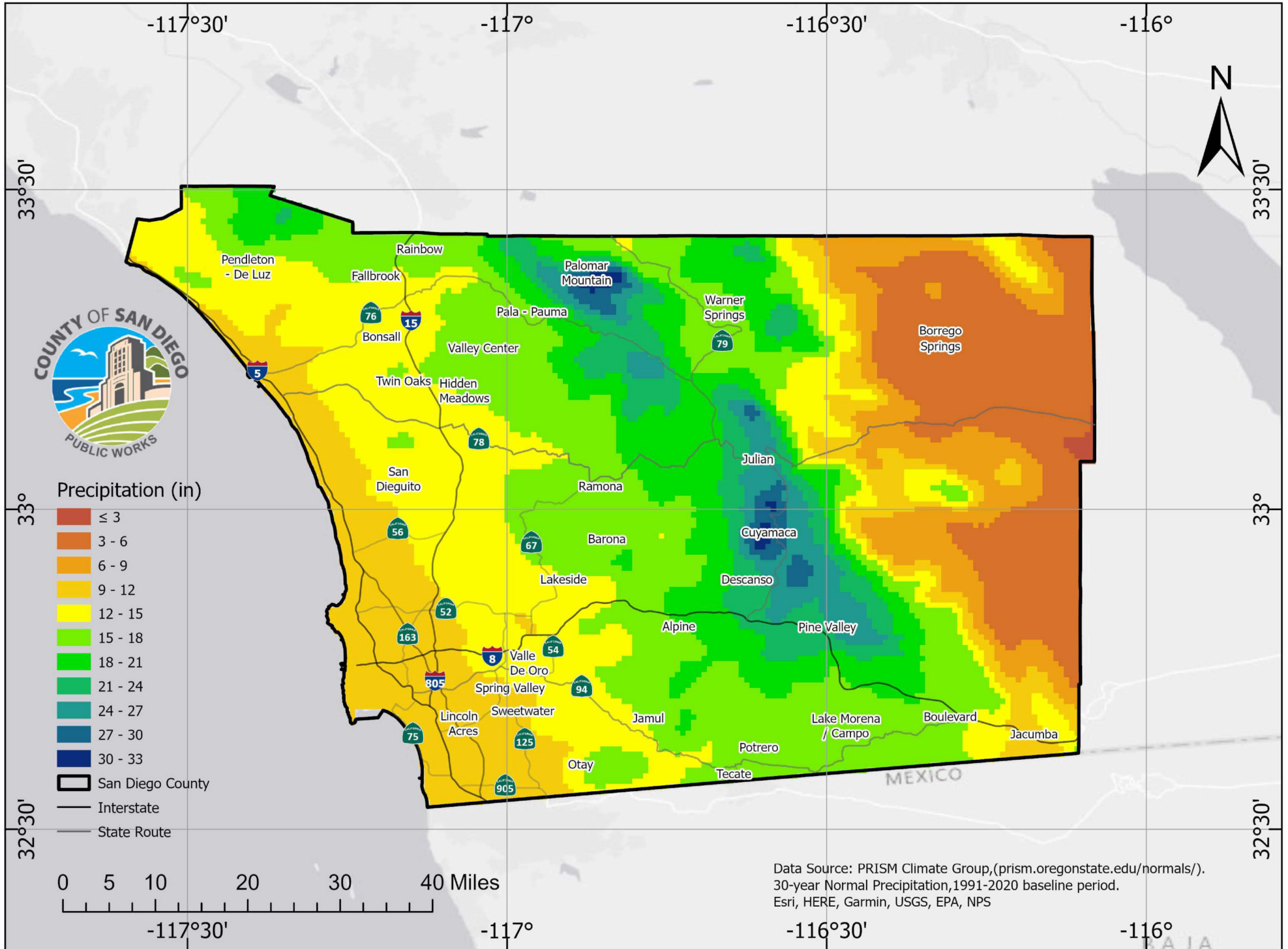


Figure 1-3

1.2.1.2 Secondary (Warm-Season) Rainfall: July through September (North American Monsoon)

A second, weaker precipitation pattern known as the North American Monsoon affects the region from July to September in most years. Moist, tropical air moves in from the south and east, primarily sourced from the tropical eastern Pacific and the Gulf of California, with occasional contributions of upper-level moisture from the Gulf of Mexico. Tropical cyclones, typically originating in the Eastern Pacific and, less frequently, from the Gulf of Mexico, can also transport moisture into the area.

- Monsoon moisture is episodic; dry, southwesterly flow can sweep it away for days at a time.
- When present, daytime heating over the mountains often triggers scattered thunderstorms. Localized downpours of > 1 inch of rain in under an hour are not unusual and can lead to flash flooding in mountainous and desert areas.
- The cool Pacific marine layer usually stabilizes the coastal zone, preventing storms from reaching the coast. The marine layer is a cool, shallow air mass formed by the cold waters of the Pacific Ocean. During the warm season, the cool air extends inland from the coast to the western foothills of the peninsular mountains. Under rare weather pattern setups, this stable layer can be eroded or mixed out, allowing monsoonal thunderstorms to move into coastal areas.

1.2.1.3 Transitional Periods: May – June and September – October

During late spring and early summer (May to June), as well as in early autumn (September to October), the county can experience a combination of the two seasonal precipitation patterns. While each season typically follows its expected weather patterns, there are occasional overlaps that introduce elements of both cool-season and monsoonal weather.

1.2.2 Weather Systems and Flooding

The Department of Public Works Flood Control Section (DPWFCS) studies and documents the major storms that occur during the local rainy season, which lasts from October to April. Most widespread flooding during this season is triggered by large, cool-season storm systems that form over the Pacific Ocean. Many of these systems contain

atmospheric rivers (ARs) - long, narrow corridors of unusually high atmospheric moisture that transport water vapor from tropical or subtropical regions into the mid-latitudes.

When an AR makes landfall and its moisture-laden winds encounter the steep western face of the Peninsular Ranges, the air is forced upward, cools, and produces heavy precipitation. A typical AR delivers roughly 1 to 2 inches of rain over a 24-hour period across areas west of and immediately over the mountains. Flooding becomes much more likely when an AR is:

- **Stronger than average**
- **Exceptionally moisture-rich**
- **Slow-moving or stalled over the region**

Under any of these conditions, storm totals can reach two to three times the usual 1 to 2-inch range. In the most intense cases, rainfall can extend well into the desert because the orographic rain-shadow effect weakens. One notable example occurred on Valentine's Day 2019, when Borrego Springs recorded 2.71 inches of rain.

The most extreme ARs may also include embedded Narrow Cold-Frontal Rainbands (NCFRs). These short-lived, highly concentrated bands typically last less than an hour and produce rainfall rates of 0.5 to 1 inch per hour, increasing the risk of flash flooding, especially in urban areas and over burn scars. When atmospheric moisture exceeds the 90th percentile, as it did during the catastrophic flooding in southern San Diego County on January 22, 2024, NCFRs can produce rates exceeding 2 inches per hour, leading to localized flash flooding.

Each summer, pulses of tropical moisture associated with the North American Monsoon flow north into San Diego County. This increased humidity fuels scattered thunderstorms over the mountains. In more intense weather setups, storms can also move east into the deserts and, on rarer occasions, west of the mountains toward the Pacific Ocean. With such moisture-rich air in place, rainfall rates in thunderstorms often exceed 1 inch per hour, which is intense enough to cause flash flooding in canyons and washes, particularly in the mountain and desert regions. A striking example occurred in August 2022, when radar estimated rainfall rates over 3 inches per hour along Sweeny Pass Road (County

Highway S-2). The deluge overwhelmed nearby washes and ultimately washed out the roadway.

From late spring through autumn, mid-latitude weather systems occasionally tap moisture streaming northward from Eastern Pacific tropical cyclones, further expanding monsoon thunderstorm coverage over the mountains and deserts. On infrequent occasions, the remnants of these storms have tracked directly across or near the county. Since 1975, the centers of tropical cyclones Kathleen (1976), Doreen (1977), Norman (1978), Nora (1997), Kay (2022), and Hurricane Hilary (2023) have all passed within roughly 100 nautical miles of San Diego County as tropical depressions or tropical storms. These storms have historically produced the most severe flooding in the desert. Still, all parts of San Diego County are susceptible to extreme rainfall and flash flooding when a decaying tropical cyclone or its moisture plume moves overhead.

In addition to spatial variation, climate varies with time. There are years with much less and years with much more than the typical annual rainfall. The 1916 flood resulted from a 2-week period in January during which the area received about 75-85% of the normal annual precipitation falling on already saturated soils from a late December storm. This resulted in the largest flood of that century. Figure 1-4 shows some of the 1916 flood devastation. At the end of that 2-week flood period, two reservoirs were destroyed, all road, rail, and telegraph lines out of the county were destroyed and the only way to travel out of the county or communicate with the rest of the world was by boat from San Diego Bay.

Both the 1980 and 1983 rain seasons were about 80% above average (Figure 1-2 & Figure 1-3). More flood damage occurred in 1980 because the major storms occurred in February after the reservoirs were full from rain in January and the previous fall.

1.2.2.1 January 22, 2024 - Flash Flood Event

On January 22, 2024, a powerful winter storm struck San Diego County, resulting in one of the most intense short-duration rainfalls ever recorded by the County's rain gauge network. The Road Division 1 Headquarters rain gauge in Spring Valley recorded 2.45

inches of rain in just one hour, surpassing the NOAA Atlas-14 threshold for 1-hour, 1 in 1,000-year precipitation return interval for that location.

The heavy rainfall caused rapid and destructive runoff across the Spring Valley area and southeastern portions of the City of San Diego. San Diego Police and Fire personnel responded to hundreds of emergency calls, conducting more than 200 water rescues of residents trapped in their homes and vehicles.

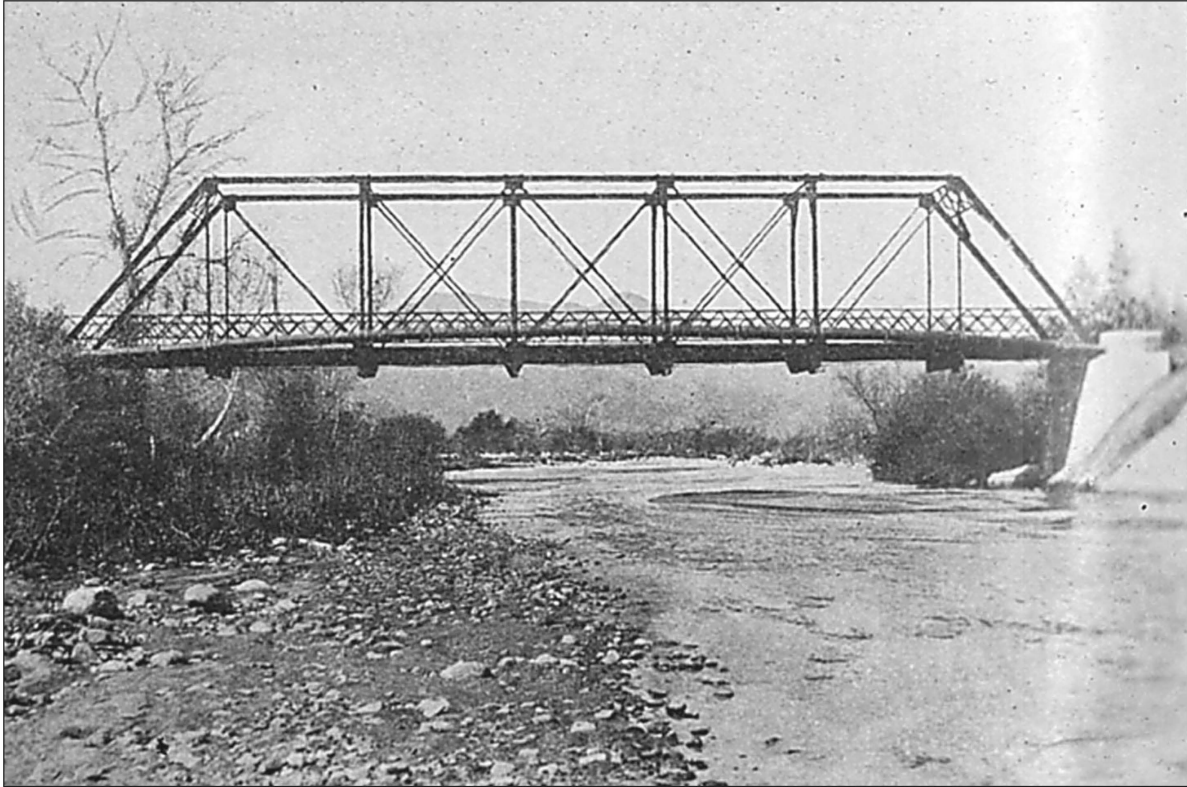
Impacts

- **Structural damage:** According to FEMA, 4,856 structures in the City of San Diego, National City, Spring Valley, and neighboring communities sustained flood damage.
- **Federal assistance:** FEMA disbursed approximately \$27 million to affected residents and businesses for property repairs, rental assistance, and temporary housing.

1.2.3 Flood Damage Prevention Structures and Programs

1.2.3.1 Flood Management

The first strategy to protect life and property from the dangers of flooding, is to identify areas in San Diego County that may be inundated by a large flood (e.g. 100-year frequency) and then regulating or restricting development in those identified areas. The regulations, in San Diego County, require building structures to be located outside of the floodway and to protect the public when building structures in the floodplain. This has been the primary focus of the flood control program in San Diego County since the 1960s. The procedure consists of developing floodplain or alluvial fan maps that identify flood hazard areas (Federal Emergency Management Agency [FEMA] and County Regulated). Figure 1-5 shows the rivers and creeks that have floodplain maps. The areas with alluvial fan maps (Borrego area) are also shown.



Highway Bridge across San Luis Rey River at Pala before the Flood of January 1916



Wreck of Highway Bridge across San Luis Rey River at Pala after the Flood of January 1916

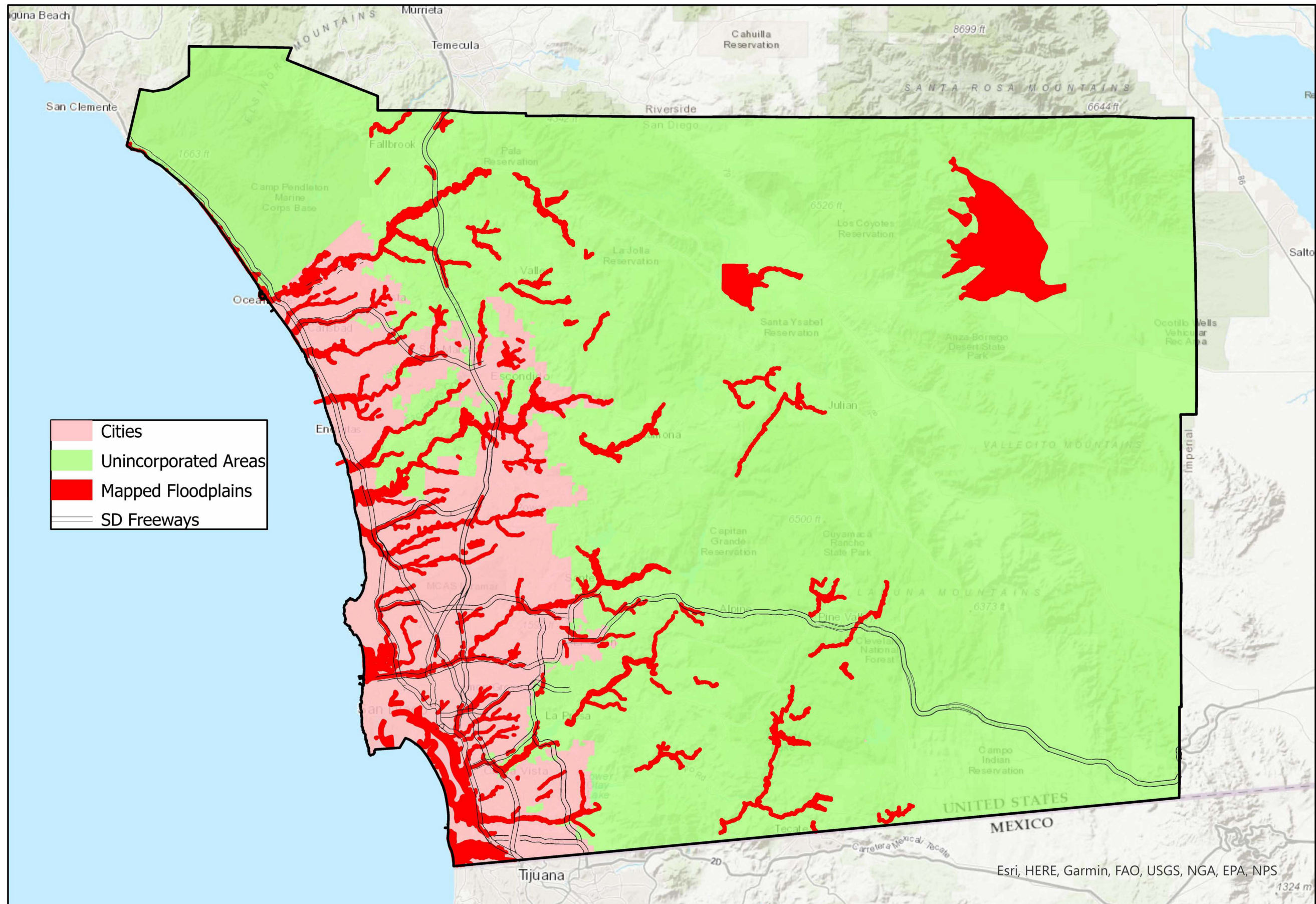
Devastation Caused by the 1916 Flood

F I G U R E

1-4

Mapped Floodplains in San Diego County

Figure 1-5



County floodplain maps were based on flood flows developed with the criteria in place at the time of development. Flood prone areas that are delineated on said maps are compared with historic floods in the same areas. The flood designations correlate well with historic events and data.

1.2.3.2 Construction of Flood Control Facilities

A second approach can be used to protect people who have already built in a flood-prone area. In this situation, a flood control structure could be constructed. Some examples of major facilities managed by the County are identified below:

- Los Coches Creek Channel
- Central Ave Drainage Improvements
- Woodside Ave Drainage Improvements
- Spring Valley Creek Channel
- Sweetwater River Levee System
- Forrester Creek Channel

A map of western San Diego County showing the constructed flood control facilities and major reservoirs is shown in Figure 1-6.

1.2.3.3 Maintenance

Stormwater management facilities and conveyance facilities (e.g., detention basins, channels, culverts, storm drain outfalls, etc.) require routine removal of accumulated sediment and/or overgrown vegetation to maintain the level of service that they have been designed for. Whether the facility is lined or unlined, deposition of sediment and/or growth of vegetation within the facility can reduce the capacity of the facility to convey stormwater, which can result in flooding. The lack of a regularly scheduled program of maintenance is a factor that reduces the effectiveness of a stormwater management system and often results in adverse environmental impacts that may be difficult and costly to correct.

Resource agency permits may be required for facility maintenance. Any stormwater management facility or conveyance facility has the potential to be required to have permits for maintenance activities from the United States Army Corps of Engineers (USACE), Regional Water Quality Control Board (RWQCB), California Department of Fish and Wildlife (CDFW), United States Fish and Wildlife Service (USFWS), and/or the California Coastal Commission where applicable. This includes lined and unlined conveyance facilities, inlets and outlets of culverts, storm drain outfalls, and detention basins. If sedimentation occurs within the facility or at the inlet or outlet of a facility, vegetation can take hold and can support a sensitive or protected species.

When applicable, appropriate permits shall be obtained from the resource agencies to allow the facilities to be maintained to the conditions used in the design. To facilitate this process, during project design determine the type of routine maintenance activities the facility will require (e.g., sediment removal, removal of woody vegetation), delineate areas to be maintained on project plans, and consult with resource agencies to establish the baseline condition of the facility and the maintenance thresholds and actions. Resource agency requirements may limit the allowable equipment to be used, maintenance actions, access points, and/or maintenance schedule. Resource agencies may require notifications and studies of alternatives prior to performing maintenance. Mitigation will be required if the facility supports a sensitive or protected species. The requirements should be determined

while the project is in design in order to incorporate design features to facilitate maintenance and to plan for maintenance costs.

Maintenance is the responsibility of the owner of the facility. A funding mechanism shall be established at the project outset to ensure proper maintenance. In the case of privately maintained stormwater management facilities, every effort should be taken to ensure the responsible party and successors are made aware of the facilities maintenance requirements. The facility shall be planned to permit easy access by maintenance personnel and equipment. Access easements or rights-of-way are required for operation and should be designed for use during both dry and wet weather. The San Diego County Hydraulic Design Manual provides requirements for maintenance access. Outside of the County of San Diego, local agencies should be consulted to determine maintenance requirements.

1.2.4 Preparation for Floods

1.2.4.1 Flood Insurance

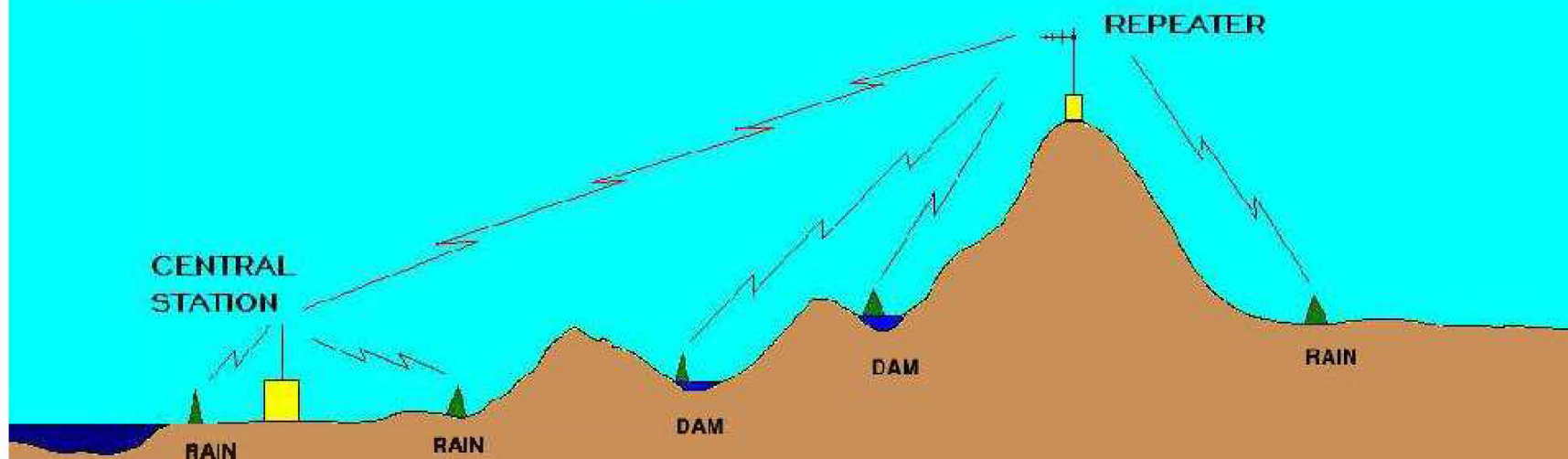
The National Flood Insurance Program (NFIP) provides flood insurance to property owners, renters and businesses, and having this coverage helps them recover faster when floodwaters recede. The NFIP works with communities required to adopt and enforce floodplain management regulations that help mitigate flooding effects. Most of the cities and the County of San Diego participate in the NFIP through FEMA. Any resident in a participating community can purchase flood insurance for their home and/or the contents of their structure from [FEMA Flood Insurance](#). Most insurance companies make flood policies available to their customers. Residents can use [FEMA Flood Maps](#) to understand the risk of flooding on their property. Using a flood map, residents can see the relationship between their property and the areas with the highest risk of flooding.

1.2.4.2 Real Time Rain and Reservoir Monitoring during Flood Periods

The Automatic Local Evaluation in Real Time (ALERT) System is a flood warning system that reports real-time rainfall and water levels at dams and rivers. Radios transmit information to the base station located in the DPWFCS office and the NWS office in

Rancho Bernardo (Figure 1-7). DPWFCS personnel coordinate closely with the Office of Emergency Services (OES) and the NWS during periods of major storms to provide information about magnitudes and locations of flooding. DPWFCS developed the first countywide ALERT system in California and is responsible for ongoing maintenance and operation (Figure 1-7). The ALERT system is coordinated with the data collection and analysis program that provides the data that is the basis for this manual. Figure 1-8 displays the spatial distribution of San Diego County’s ALERT Flood Warning System monitoring network as of August 2025. The map identifies the locations of rain gauges, stream gauges, lake-level gauges, combined rain and stream gauges, combined rain and lake-level gauges, and sites that include both rain gauges and weather stations throughout the county. Station locations are shown in relation to the San Diego County boundary, major drainage features, watershed boundaries, and tribal reservation lands. The figure demonstrates the extensive geographic coverage of the ALERT network, spanning the county’s coastal, inland valleys, foothills, mountains, and desert. These stations support flood monitoring and warning operations in both urban and remote watersheds.

ALERT STORM DATA SYSTEM



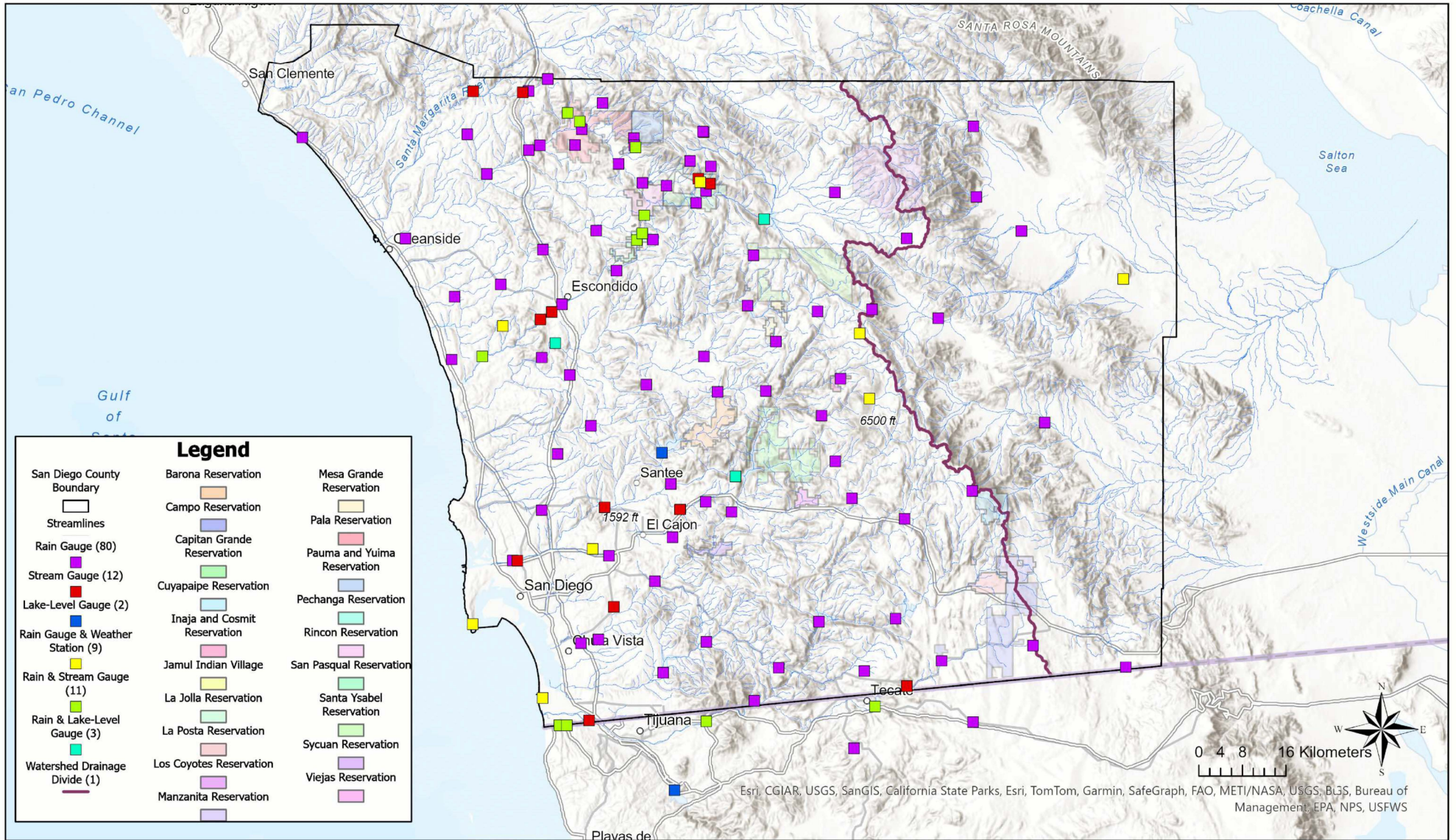
Automatic Local Evaluation in Real-Time

- ◆ Real-time radio telemetry, continuous operations
- ◆ System not dependent on commercial power
- ◆ Rain, stream, lake, weather, water quality sensors
- ◆ Data maintained in base station computers
- ◆ Internet access for select sensors

San Diego County ALERT Flood Warning System: Station Locations

Current as of August 2025

Figure 1-8



1.3 POLICIES AND INTENDED USE

1.3.1 Application of Design Standards

It is the policy of the County of San Diego that hydrologic studies for the purpose of flood management, including studies supporting design of new facilities and project-specific studies evaluating level of service of existing facilities, adhere to the criteria presented in this manual. Flood flows computed using the methods presented herein serve as the benchmark for determining the level of service provided by existing and planned facilities.

1.3.2 Exceptions to Design Standards

The County of San Diego recognizes that it is not possible to prescribe design standards and procedures for all situations. As noted in the San Diego County Hydraulic Design Manual, there are many already developed areas within the San Diego Region that do not fully conform to the drainage standards presented in the Hydraulic Design Manual. Exceptions to the standards of the Hydraulic Design Manual may be made in certain circumstances as described in the Hydraulic Design Manual. While exceptions may be made to design standards, it is expected that flood flows shall be computed consistently with the procedures presented in this Hydrology Manual, which allows for consistent evaluation of the level of service provided by drainage and flood control facilities. The 100-year peak flow rates computed using the methods presented in this Hydrology Manual serve as the performance standard for design of drainage and flood control facilities, except for potential exceptions described below (note the DPWFCS reserves the right to reject any or all computations):

- When there is already an established flood flow from a County of San Diego drainage master plan, a nearby development or road project, a FEMA Flood Insurance Study, or a recorded flow (i.e., stream gauge data), the County of San Diego may evaluate its applicability for design on a case-by-case basis. See Section 2 for further information about sources of established flood flows.
- When there is a critical facility that requires a higher level of protection than the 100-year storm event, hydrologic methods to be applied when designing to

extraordinary protection levels may be considered on a case-by-case basis at the discretion of the County of San Diego.

- Watershed-wide planning-level studies may apply alternative procedures when approved by the County of San Diego, with the understanding that studies supporting project-level design shall conform to the procedures of this Hydrology Manual.

1.4 ORGANIZATION OF MANUAL

This manual is organized into eight sections. Section 1 is the introduction to the manual and includes background information and a flowchart of the review process. Section 2 details other sources of established flood flow information.

Section 3 discusses the Rational Method and Modified Rational Method. Section 4 discusses the NRCS hydrologic method. Please note that in Sections 3 and 4 the concept of infiltration is not covered. It is only on rare occasions that the engineer needs to adjust for infiltration, specifically channel infiltration. Always check with the agency of jurisdiction before incorporating an infiltration adjustment into any calculations. Section 5 addresses erosion and sedimentation. Section 6 discusses hydrographs created based on Rational Method and Modified Rational Method study results. Section 7 addresses water quality considerations as they relate to hydrology. Finally, Section 8 is dedicated to the bibliography for this manual. Whenever possible, figures and examples are included in the sections to clarify the procedures. In each section, the use of applicable computer programs is discussed.

1.5 DATA AVAILABLE FROM THE COUNTY OF SAN DIEGO

The DPWFCS maintains a long-term program of hydrologic data collection and recording, including rainfall, streamflow, and erosion. These data are maintained by the DPWFCS in a single data bank and analyzed to derive hydrologic parameters for use in the various simulation models and for other flood control purposes, such as flood warning systems. County of San Diego Planning and Development Services (PDS) also collects and maintains groundwater data.

[NRCS](#) has also developed several maps showing hydrologic parameters. The maps most directly applicable to flood flow computations are the hydrologic soil group maps and the ground cover maps. Other maps, such as those showing erosion and brush conversion, are also used for flood flow computations. [SANGIS](#) is an additional resource that is available to the public and can be used for determining land use data and soil types.

A major function of the DPWFCS is the coordination of the flood control projects with the studies of other agencies. This coordination allows for consistent results. The County of San Diego has done a considerable amount of work with floodplain studies and flood control designs. These existing studies may be used as a reference (see Section 2).

1.6 DRAINAGE REPORT REQUIREMENTS AND COMPONENTS

Drainage reports should follow the guidelines indicated in Figure 1-9 when using the Unit Hydrograph Method and revised to support calculations using other methods such as the Rational Method. Examples of acceptable Declarations of Responsible Charge are presented in Figure 1-10.

1.7 USE OF COMPUTER PROGRAMS

Any computer program used to support an engineer's calculations in preparation of a hydrology report should include good documentation of the calculations. The report should document information including input, steps followed, and key maps used. The output from the program's calculations should be presented in summary but also in detail in the

report. The goal of the documentation is to clearly present the operations performed by the computer. Check with the local jurisdiction before using any software to determine its applicability. Any software used needs to follow the processes required by this manual.

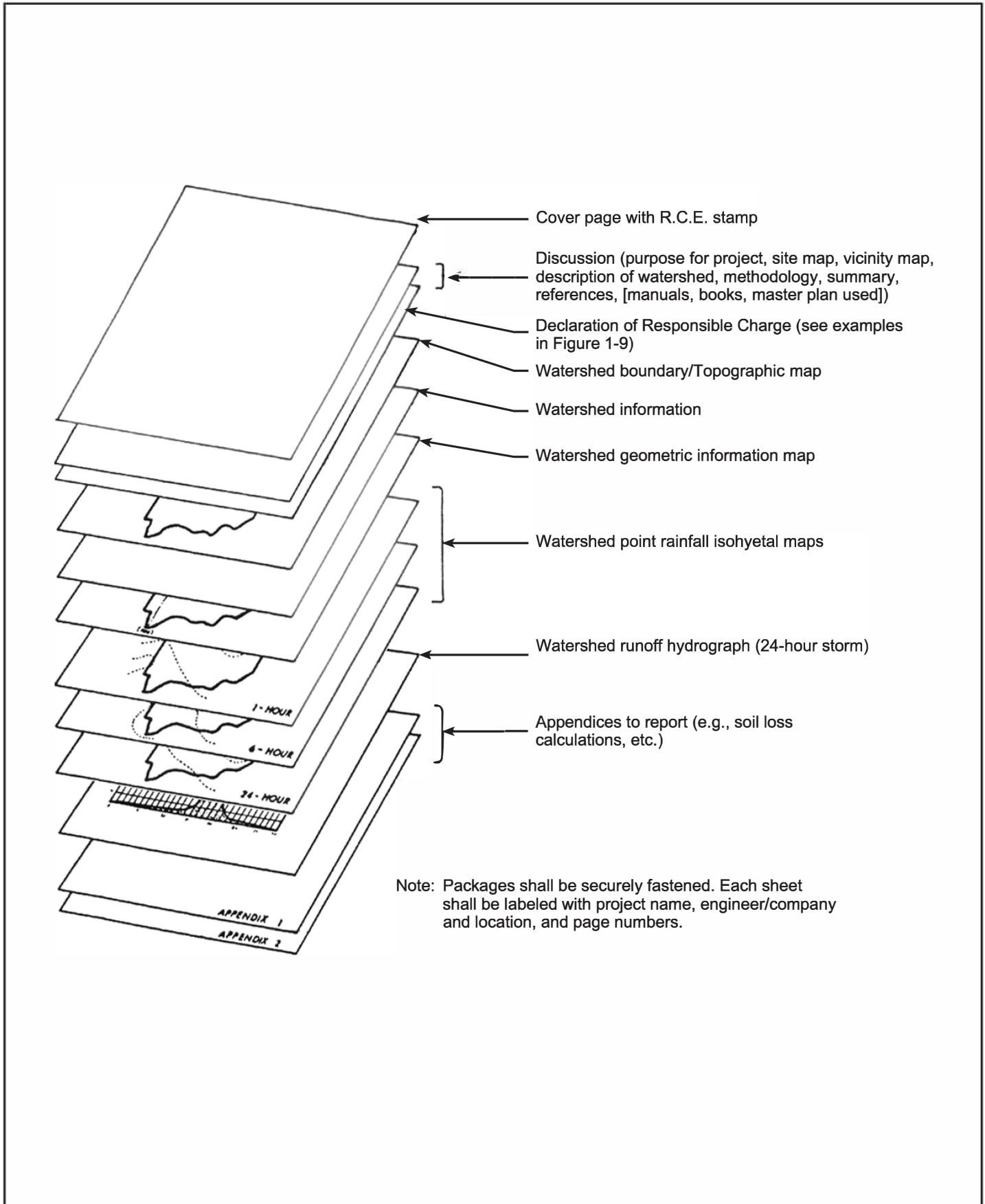
A computer program titled SDHydroTools is provided with this manual. The SDHydroTools computer program includes modules to:

- Obtain NOAA Atlas 14 rainfall depth data for the 100-year storm event for locations within San Diego County.
- Interpolate precipitation depths and intensities using log-log interpolation.
- Prepare a nested storm hyetograph for use with the NRCS hydrologic method presented in Section 4 of this manual.
- Calculate a peak flow rate and generate a hydrograph using the NRCS hydrologic method presented in Section 4 of this manual.
- Prepare a rational method hydrograph using the procedure presented in Section 6 of this manual.

The SDHydroTools computer program is a tool provided free of charge to Hydrology Manual users in an effort to streamline traditionally complex efforts associated with hydrologic calculations. Use of this tool is optional and the County will not be held liable for any errors or other negative impacts associated with its use. In addition to the SDHydroTools computer program provided by the County, the following public domain programs are acceptable:

- USACE HEC: *Hydrologic Modeling System HEC-HMS*
- Other programs that are accepted by FEMA but have been pre-approved by local jurisdiction prior to use.

Any software used needs to follow the processes required by this manual.



**Required Report Format for Unit Hydrograph Study
San Diego County Hydrology Manual**

F I G U R E

1-9

DECLARATION OF RESPONSIBLE CHARGE:

I HEREBY DECLARE THAT I AM THE ENGINEER OF WORK FOR THIS PROJECT, THAT I HAVE EXERCISED RESPONSIBLE CHARGE OVER THE DESIGN OF THE PROJECT AS DEFINED IN SECTION 6703 OF THE BUSINESS AND PROFESSIONS CODE, AND THAT THE DESIGN IS CONSISTENT WITH CURRENT STANDARDS.

I UNDERSTAND THAT THE CHECK OF PROJECT DRAWINGS AND SPECIFICATIONS BY THE CITY OF SAN DIEGO IS CONFINED TO A REVIEW ONLY AND DOES NOT RELIEVE ME, AS ENGINEER OF WORK, OF MY RESPONSIBILITIES FOR PROJECT DESIGN.



ENGINEER OF WORK:

SAN DIEGO LAND SURVEYING & ENGINEERING, INC.
9619 CHESAPEAKE DRIVE, SUITE 204
SAN DIEGO, CALIFORNIA 92123
PHONE: (858) 565-8362
PROJECT NO. 452-214-56
ROBINSON-1TS.DWG

03-01-2001

MICHAEL L. SMITH, R.C.E. 35471 DATE
REGISTRATION EXPIRES SEPTEMBER 30, 2003

DECLARATION OF RESPONSIBLE CHARGE

I, HEREBY DECLARE THAT I AM THE CIVIL ENGINEER OF WORK FOR THIS PROJECT, THAT I HAVE EXERCISED RESPONSIBLE CHARGE OVER THE DESIGN OF THE PROJECT AS DEFINED IN SECTION 6703 OF THE BUSINESS AND PROFESSIONS CODE, AND THAT THE DESIGN IS CONSISTENT WITH CURRENT DESIGN.

I UNDERSTAND THAT THE CHECK OF PROJECT DRAWINGS AND SPECIFICATIONS BY THE CITY OF SAN DIEGO IS CONFINED TO A REVIEW ONLY AND DOES NOT RELIEVE ME, AS ENGINEER OF WORK, OF MY RESPONSIBILITIES FOR PROJECT DESIGN.

2/12/04

EDWIN E. PEACE
R.C.E. 37586
EXP. 9-30-04

DATE

DECLARATION OF RESPONSIBLE CHARGE

I HEREBY DECLARE THAT I AM THE ENGINEER OF WORK FOR THIS PROJECT. THAT I HAVE EXERCISED RESPONSIBLE CHARGE OVER THE DESIGN OF THE PROJECT AS DEFINED IN SECTION 6703 OF THE BUSINESS AND PROFESSIONS CODE, AND THAT THE DESIGN IS CONSISTENT WITH CURRENT STANDARDS.

I UNDERSTAND THAT THE CHECK OF PROJECT DRAWINGS AND SPECIFICATIONS BY THE CITY OF SAN DIEGO IS CONFINED TO A REVIEW ONLY AND DOES NOT RELIEVE ME, AS ENGINEER OF WORK, OF MY RESPONSIBILITIES FOR PROJECT DESIGN.

3-27-2001

M. SCOTT LILLIBRIDGE R.C.E. 52504 DATE



FIGURE

Example Declarations of Responsible Charge

1-10

SECTION 2 REGIONAL FLOOD FLOW INFORMATION AND SELECTION OF HYDROLOGIC METHOD AND DESIGN CRITERIA

2.1 REGIONAL FLOOD FLOW INFORMATION

Political entities are responsible for establishing and regulating flood control functions. In most watersheds of major size or importance (especially urban areas) flood flows are already established through one or more of the following activities.

1. Master Plan Development – The County of San Diego and the cities within the County have established master plans, most of which have been in effect for many years. For example, the County adopted the Comprehensive Flood Control and Drainage Plans in the mid-1970s. These plans have been updated at various times. These analyses are for planning purposes ONLY. Flowrates and capacities for specific projects should be based upon independent, detailed, site-specific hydrologic and hydraulic analyses in accordance with this manual and the County’s Hydraulic Design Manual.
2. Studies for Development and Road Projects – These studies provide the basis for the design of projects such as channels and pipes, which are included in a major project. Once approved by a city or the County of San Diego, the flood flows form a basis for other nearby projects.
3. Flood Insurance Studies (FIS) – Rivers and creeks that have floodplain or alluvial fan maps may be identified through the Flood Insurance Study for San Diego County. USACE used the flood flow frequency analysis for FIS in the 1970s. USACE also used the frequency analysis method for the smaller flows and a rain/runoff analysis method to determine flows for the larger recurrence intervals. These maps and related information (including flood flows) are usually available at the offices of the political entity (city or county) that has land use jurisdiction. The local government and FEMA must approve any modification to the established flood flows.

It is important to review the land use basis for the FEMA hydrology studies. The studies are based on the existing land use at the time the study was completed. Since many of the FEMA maps were originally prepared in 1970, urbanization may have caused increased flows.

FEMA maps can be viewed at the [SanGIS Web site](#) or the [National Flood Hazard Layer \(NFHL\) Viewer](#). Maps may be viewed and printed with various overlays such as the May 2012 (or latest) FEMA panels, roads, city boundaries, and property parcel boundaries.

FEMA FIS flow rates can be used for FEMA regulatory purposes such as remapping, and no-rise certification, but all facilities need to be designed using the methodology outlined within this Hydrology Manual and the San Diego County Hydraulic Design Manual.

4. Recorded Flows – The United States Geological Survey (USGS), the County of San Diego, and other agencies have collected streamflow and flood stage data at numerous locations throughout San Diego County. Recorded flow information may be available from USGS Water Data for the Nation, USGS National Water Dashboard monitoring location pages, County-maintained stream gauge records, FEMA Flood Insurance Studies, and other approved hydrologic studies.

Historic flood information should also be reviewed where applicable. A major historic reference is the USGS Water Supply Paper 426, *Southern California Floods of January, 1916*, which documents one of the largest and most damaging flood events in San Diego County's recorded history. The appendix to State of California Bulletin 112 also provides historical flood information for San Diego County.

Recorded flow data should be evaluated for applicability before use in design. The engineer should consider the period of record, watershed conditions at the time of measurement, gage location, rating curve limitations, regulation or diversion effects,

land use changes, and whether the recorded data are representative of current or proposed project conditions.

If an engineer bases the design upon a previously calculated flow rate, those analyses will need to be certified by the current engineer (as his/her own) and resubmitted as justification for the current project.

Any available flow rate should be compared to the currently calculated flow rate, and any significant differences should be discussed.

2.2 TYPES OF HYDROGRAPHS

This classification is a partial list, suitable for use in watershed work.

1. Natural hydrographs. Obtained directly from the flow records of a gauged stream.
2. Synthetic hydrographs. Obtained by using watershed parameters and storm characteristics to simulate a natural hydrograph.
3. Unit hydrograph. A natural or synthetic hydrograph for 1 inch of excess rainfall. The excess rainfall is assumed to occur uniformly over the watershed in a specified time.
4. Dimensionless hydrograph. Made to represent many unit hydrographs by using the T_p and the peak discharge rates as basic units and plotting the hydrographs in ratios of these units. Also called index hydrograph.

2.3 SELECTION OF HYDROLOGIC METHOD AND DESIGN CRITERIA

Design Frequency – For hydrologic analysis, a 100-year design frequency shall be used. Specific design criteria for various drainage infrastructure such as storm drains, roadway drainage, inlets, and culverts are provided in the County of San Diego Hydraulic Design Manual.

Design Method – The choice of method to determine flows (discharge) shall be based on the size of the watershed area. For an area 0 to approximately 1 square mile, the Rational

Method or the Modified Rational Method shall be used. For watershed areas larger than 1 square mile the NRCS hydrologic method shall be used. Please check with the governing agency for any variations to these guidelines.

Refer to Section 5.6 for guidance on the applicability, design criteria, and hydrologic methods used to account for sediment bulking in flow calculations for infrastructure design in areas that may be subject to sediment or debris flow hazards.

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SECTION 3 RATIONAL METHOD AND MODIFIED RATIONAL METHOD

3.1 THE RATIONAL METHOD

The Rational Method (RM) is a mathematical formula used to estimate the maximum runoff rate from a given rainfall intensity. It has particular application in urban storm drainage, where it is used to estimate peak runoff rates from small urban and rural watersheds for the design of storm drains and small drainage structures. The RM is recommended for analyzing the runoff response from drainage areas up to approximately 1 square mile in size. It should not be used in instances where there is a junction of independent drainage systems or for drainage areas greater than approximately 1 square mile in size. In these instances, the Modified Rational Method (MRM) should be used for junctions of independent drainage systems in watersheds up to approximately 1 square mile in size (see Section 3.4). The NRCS Hydrologic Method should be used for watersheds greater than approximately 1 square mile in size (see Section 4).

The RM can be applied using any design storm frequency (e.g., 100-year, 50-year, 10-year, etc.). The local agency determines the design storm frequency that must be used based on the type of project and specific local requirements. A discussion of design storm frequency is provided in Section 2.3 of this manual.

3.1.1 Rational Method Formula

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

$$Q = C I A$$

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

- C = runoff coefficient, proportion of the rainfall that runs off the surface (no units)
- I = average rainfall intensity for a duration equal to the T_c for the area, in inches per hour (Note: If the computed T_c is less than 5 minutes, use 5 minutes for computing the peak discharge, Q)
- A = drainage area contributing to the design location, in acres

Combining the units for the expression CIA yields:

$$\left(\frac{1 \text{ acre} \times \text{inch}}{\text{hour}}\right) \left(\frac{43,560 \text{ ft}^2}{\text{acre}}\right) \left(\frac{1 \text{ foot}}{12 \text{ inches}}\right) \left(\frac{1 \text{ hour}}{3,600 \text{ seconds}}\right) \Rightarrow 1.008 \text{ cfs}$$

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

The RM formula is based on the assumption that for constant rainfall intensity, the peak discharge rate at a point will occur when the raindrop that falls at the most hydraulically remote point in the tributary drainage basin arrives at the point of interest.

Unlike the NRCS hydrologic method (discussed in Section 4), the RM and/or MRM do not create hydrographs and therefore do not add separate subarea hydrographs at collection points. Instead, the RM develops peak discharges in the main line by increasing the T_c as flow travels downstream. The MRM combines flows from different subareas based on ratios of contributing areas, times of concentration, and intensities.

Assumptions inherent to the RM as applied in this Manual:

- The discharge flow rate resulting from any constant I is maximum when the I lasts as long as or longer than the T_c .
- The storm frequency of peak discharges is the same as that of I for the given T_c .

- The fraction of rainfall that becomes runoff (or the runoff coefficient, C described in section 3.1.2) is assumed independent of I or precipitation zone number (PZN) condition (PZN Condition is discussed in Section 4.1.2.4).
- The peak rate of runoff is the only information produced by using the RM.

3.1.2 Runoff Coefficient

Table 3-1 lists the estimated runoff coefficients for urban areas. The concepts related to the runoff coefficient were evaluated in a report entitled *Evaluation, Rational Method "C" Values* (Hill, 2002) that was reviewed by the Hydrology Manual Committee (at the time, in 2002). The Report is available at San Diego County Department of Public Works, Flood Control Section and on the San Diego County Department of Public Works web page.

The runoff coefficients are based on land use and soil type. Soil type can be determined from the soil type map provided in Appendix A and available from SANGIS. An appropriate runoff coefficient (C) for each type of land use in the subarea should be selected from this table and multiplied by the percentage of the total area (A) included in that class. The sum of the products for all land uses is the weighted runoff coefficient ($\Sigma[CA]$). Good engineering judgment should be used when applying the values presented in Table 3-1, as adjustments to these values may be appropriate based on site-specific characteristics. The impervious percentages shown in Table 3-1 for each County land use element are suitable for planning-level studies. For project-level studies and design of storm water conveyance systems, the actual impervious percentage (% Impervious) existing or proposed, as applicable, shall govern the selected value for C . The runoff coefficient can also be calculated for an area based on soil type and impervious percentage using the following formula:

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

Where: C_p = Pervious Coefficient Runoff Value for the soil type (shown in Table 3-1 as Undisturbed Natural Terrain/Permanent Open Space, 0% Impervious). Soil type can be determined from the soil type map provided in Appendix A and available from SANGIS.

The values in Table 3-1 are typical for most urban areas. However, if the basin contains rural or agricultural land use, parks, golf courses, or other types of nonurban land use that are expected to be permanent, the appropriate value should be selected based upon the soil and cover and approved by the local agency.

**Table 3-1
RUNOFF COEFFICIENTS FOR URBAN AREAS**

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

*The values associated with 0% impervious may be used for direct calculation of the runoff coefficient as described in Section 3.1.2 (representing the pervious runoff coefficient, Cp, for the soil type), or for areas that will remain undisturbed in perpetuity. Justification must be given that the area will remain natural forever (e.g., the area is located in Cleveland National Forest).

DU/A = dwelling units per acre

NRCS = National Resources Conservation Service

3.1.3 Rainfall Intensity

The rainfall intensity (I) is the rainfall in inches per hour (in/hr) for a duration equal to the T_c for a selected storm frequency. Once a particular storm frequency has been selected for design and a T_c calculated for the drainage area, the rainfall intensity can be determined from precipitation intensity data pairs provided in NOAA Atlas 14. Discussion of NOAA Atlas 14 and procedures to obtain data from NOAA Atlas 14 are provided in Appendix B of this manual.

NOAA Atlas 14 provides precipitation depth data and precipitation intensity data for selected durations (e.g., 5 minutes, 10 minutes, etc.) ranging from 5 minutes to 60 days. Typical rational method times of concentration should fall between approximately 5 minutes and 30 minutes. For a time of concentration less than 5 minutes, use the 5-minute precipitation intensity for the rational method calculation. For a time of concentration that is between given data points from NOAA Atlas 14 (e.g., 8 minutes), use log-log interpolation to interpolate the intensity for the time of concentration, using the following equation:

$$I_{T_c} = K \cdot T_c^n$$

$$\text{with } K = I_1/T_1^n = I_2/T_2^n$$

$$n = \text{Log}(I_2/I_1)/\text{Log}(T_2/T_1)$$

Or

$$I_{T_c} = \text{Log}^{-1}(\text{Log} I_1 + (\text{Log}(T_c / T_1)) (\text{Log}(I_2 / I_1) / (\text{Log}(T_2 / T_1))))$$

Or

$$I_{T_c} = I_1 \cdot (T_c / T_1)^n$$

$$\text{with } n = \text{Log}(I_2 / I_1) / (\text{Log}(T_2 / T_1))$$

Where:

I_{T_c} = Intensity at T_c (inches per hour)

T_c = time of concentration (minutes)

T_1 and T_2 are the bounding times (minutes) of given data points from NOAA Atlas 14 and $T_1 < T_c < T_2$

I_1 = Intensity at T_1 (inches per hour) from NOAA Atlas 14

I_2 = Intensity at T_2 (inches per hour) from NOAA Atlas 14

The SDHydroTools computer program provided with this manual includes a module for interpolation of precipitation depths or intensities between given data pairs.

3.1.4 Time of Concentration

The Time of Concentration (T_c) is the time required for runoff to flow from the most remote part of the drainage area to the point of interest (note that the most remote point is not necessarily the highest or farthest point in the watershed as remoteness relates to time of travel rather than distance). The T_c is composed of two components: initial time of concentration (T_i) and travel time (T_t). Methods of computation for T_i and T_t are discussed below. The T_i is the time required for runoff to travel across the surface of the most remote subarea in the study, or “initial subarea.” Guidelines for designating the initial subarea are provided within the discussion of computation of T_i . The T_t is the time required for the runoff to flow in a watercourse (e.g., swale, channel, gutter, pipe) or series of watercourses from the initial subarea to the point of interest. For the RM, the T_c at any point within the drainage area is given by:

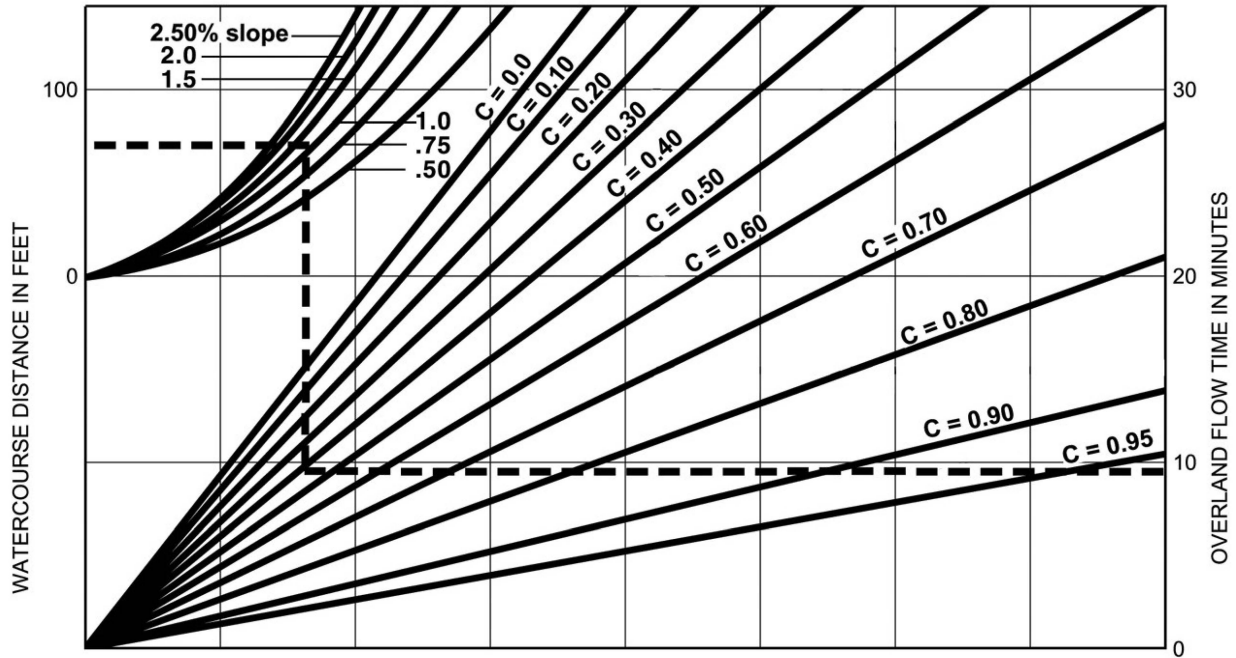
$$T_c = T_i + T_t$$

Methods of calculation differ for natural watersheds (non-urbanized) and for urban drainage systems. When analyzing storm drain systems, the designer must consider the possibility that an existing natural watershed may become urbanized during the useful life of the storm drain system. Future land uses must be used for T_c and runoff calculations, and can be determined from the local Community General Plan.

3.1.4.1 Initial Time of Concentration

The initial time of concentration is typically based on sheet flow at the upstream end of a drainage basin. The Overland Time of Flow (Figure 3-1) is approximated by an equation developed by the Federal Aviation Agency (FAA) for analyzing flow on runways (FAA, 1970). The usual runway configuration consists of a crown, like most freeways, with sloping pavement that directs flow to either side of the runway. This type of flow is uniform in the

direction perpendicular to the velocity and is very shallow. Since these depths are $\frac{1}{4}$ of an inch (more or less) in magnitude, the relative roughness is high. Some higher relative roughness values for overland flow are presented in Table 14 of the *HEC-HMS Technical Reference Manual (USACE, March 2000)*.



EXAMPLE:

Given: Watercourse Distance (D) = 70 Feet
 Slope (s) = 1.3%
 Runoff Coefficient (C) = 0.41
 Overland Flow Time (T) = 9.5 Minutes

$$T = \frac{1.8 (1.1-C) \sqrt{D}}{\sqrt[3]{s}}$$

SOURCE: Airport Drainage, Federal Aviation Administration, 1965

FIGURE

3-1

Rational Formula - Overland Time of Flow Nomograph

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

- Urban Areas – This “runway type” runoff includes:
 - 1) Flat roofs, sloping at 1% ±
 - 2) Parking lots at the extreme upstream drainage basin boundary (at the “ridge” of a catchment area).

Even a parking lot is limited in the amounts of sheet flow. Parked or moving vehicles would “break-up” the sheet flow, concentrating runoff into streams that are not characteristic of sheet flow.
 - 3) Driveways are constructed at the upstream end of catchment areas in some developments. However, if flow from a roof is directed to a driveway through a downspout or other conveyance mechanism, flow would be concentrated.
 - 4) Flat slopes are prone to meandering flow that tends to be disrupted by minor irregularities and obstructions. Maximum Overland Flow lengths are shorter for the flatter slopes (see Table 3-2).

- Rural or Natural Areas - The FAA equation is applicable to these conditions since (.5% to 10%) slopes that are uniform in width of flow have slow velocities consistent with the equation. Irregularities in terrain limit the length of application.
 - 1) Most hills and ridge lines have a relatively flat area near the drainage divide. However, with flat slopes of 0.5% ±, minor irregularities would cause flow to concentrate into streams.
 - 2) Parks, lawns and other vegetated areas would have slow velocities that are consistent with the FAA Equation.

The concepts related to the initial time of concentration were evaluated in a report entitled *Initial Time of Concentration, Analysis of Parameters* (Hill, 2002) that was reviewed by the Hydrology Manual Committee (at that time, in 2002). The Report is available at San Diego County Department of Public Works, Flood Control Section and on the San Diego County Department of Public Works web page.

Note that the Initial Time of Concentration should be reflective of the general land-use at the upstream end of a drainage basin.

Table 3-2 provides limits of the length (Maximum Length (L_M)) of sheet flow to be used in hydrology studies. Initial T_i values based on average C values for the Land Use Element are also included.

Table 3-2

**MAXIMUM OVERLAND FLOW LENGTH (L_M)
 & INITIAL TIME OF CONCENTRATION (T_i)**

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

*See Table 3-1 for more detailed description

3.1.4.1A Planning Considerations

The purpose of most hydrology studies is to develop flood flow values for areas that are not at the upstream end of the basin. Another example is the Master Plan, which is usually completed before the actual detailed design of lots, streets, etc. are accomplished. In these situations it is necessary that the initial time of concentration be determined without detailed information about flow patterns.

To provide guidance for the initial time of concentration design parameters, Table 3-2 includes the Land Use Elements and other variables related to the Time of Concentration. The table development included a review of the typical “layout” of the different Land Use Elements and related flow patterns and consideration of the extent of the sheet flow regimen, the effect of ponding, the significance to the drainage basin, downstream effects, etc.

3.1.4.1B Computation Criteria

- (a) Developed Drainage Areas With Overland Flow - T_i may be obtained directly from the chart, “Rational Formula – Overland Time of Flow Nomograph,” shown in Figure 3-1 or from Table 3-2. This chart is based on the Federal Aviation Agency (FAA) equation (FAA, 1970). For the short rain durations (<15 minutes) involved, intensities are high but the depth of flooding is limited and much of the runoff is stored temporarily in the overland flow and in shallow ponded areas. In developed areas, overland flow is limited to lengths given in Table 3-2. Beyond these distances, flow tends to become concentrated into streets, gutters, swales, ditches, closed conduits, etc.

- (b) Natural Or Rural Watersheds – These areas usually have an initial subarea at the upstream end with sheet flow. The sheet flow length is limited to 50 to 100 feet as specified in Table 3-2. The Overland Time of Flow Nomograph, Figure 3-1 or Table 3-2, can be used to obtain T_i . The initial time of concentration can excessively affect the magnitude of flow further downstream in the drainage basin. For instance, variations in the initial time of concentration for an initial subarea of one acre can change the flow further downstream where the area is 400 acres by 100%. Therefore, the initial time of concentration is limited (see Table 3-2).

The Rational Method procedure included in the original Hydrology Manual and Design and Procedure Manual included a 10 minute value to be added to the initial time of concentration developed through the Kirpich Formula (see Figure 3-2) for a natural watershed. That procedure is superseded by the procedure above to use Table 3-2 or Figure 3-1 to determine T_i for the appropriate sheet flow length of the initial subarea. The values for natural watersheds given in Table 3-2 vary from 13 to 7 minutes, depending on slope. If the total length of the initial subarea is greater than the maximum length allowable based on Table 3-2, add the travel time based on the Kirpich formula for the remaining length of the initial subarea.

3.1.4.2 Travel Time

The T_t is the time required for the runoff to flow in a watercourse (e.g., swale, channel, gutter, closed conduit) or series of watercourses from the initial subarea to the point of interest. The T_t is computed by dividing the length of the flow path by the computed flow velocity. Since the velocity normally changes as a result of each change in flow rate or slope, such as at an inlet or grade break, the total T_t must be computed as the sum of the T_t 's for each section of the flow path. Use Figure 3-4 to estimate time of travel for street gutter flow. Velocity in a channel can be estimated by using the nomograph shown in Figure 3-5 (Manning's Equation Nomograph).

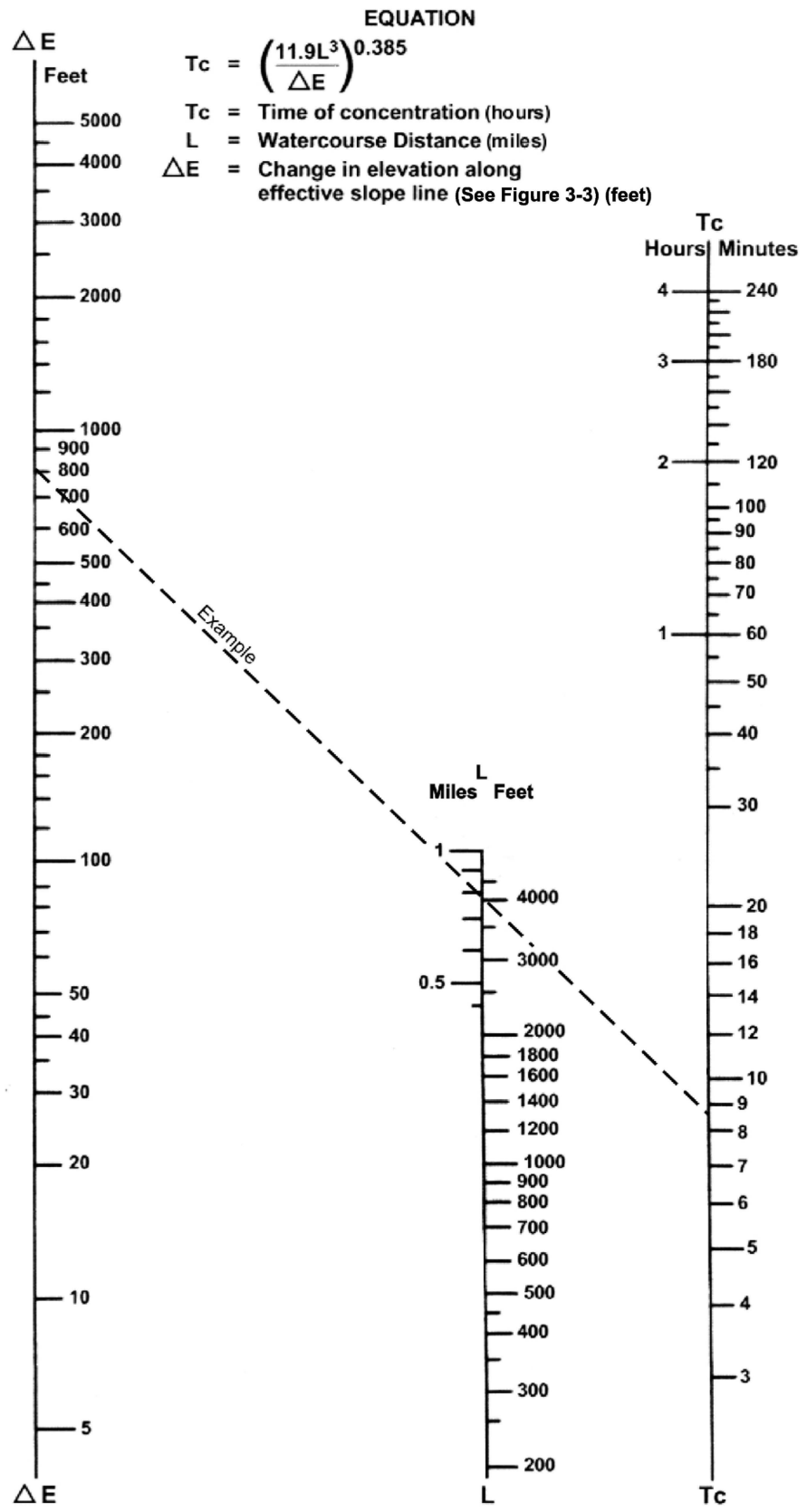
- (a) Natural Watersheds – This includes rural, ranch, and agricultural areas with natural channels. Obtain T_t directly from the Kirpich nomograph in Figure 3-2 or from the equation. This nomograph requires values for length and change in elevation along the effective slope line for the subarea. See Figure 3-3 for a representation of the effective slope line.

This nomograph is based on the Kirpich formula, which was developed with data from agricultural watersheds ranging from 1.25 to 112 acres in area, 350 to 4,000 feet in length, and 2.7 to 8.8% slope (Kirpich, 1940). A maximum length of 4,000 feet should be used for the subarea length. Typically, as the flow length increases, the depth of flow will increase, and therefore it is considered a concentration of flow at points beyond lengths. However, because the Kirpich formula has been shown to be applicable for watersheds up to 4,000 feet in length (Kirpich, 1940), a subarea may be designated with a length up to 4,000 feet provided the topography and slope of the natural channel are generally uniform.

Justification needs to be included with this calculation showing that the watershed will remain natural forever. Examples include areas located in the Multiple Species Conservation Plan (MSCP), areas designated as open space or rural in a community's General Plan, and Cleveland National Forest.

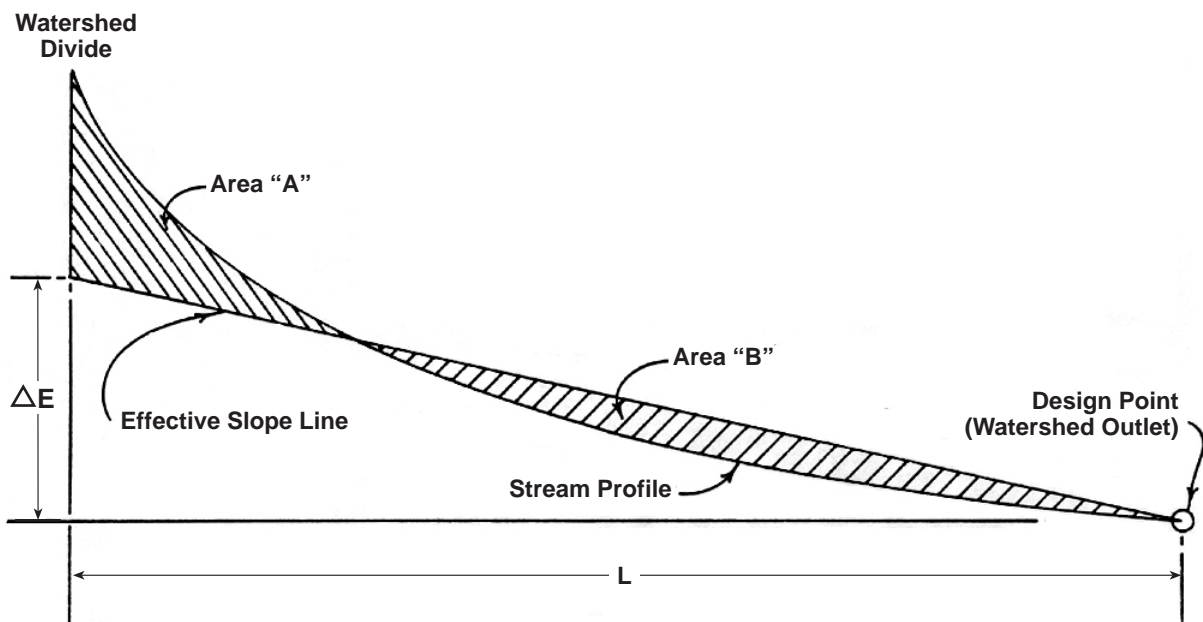
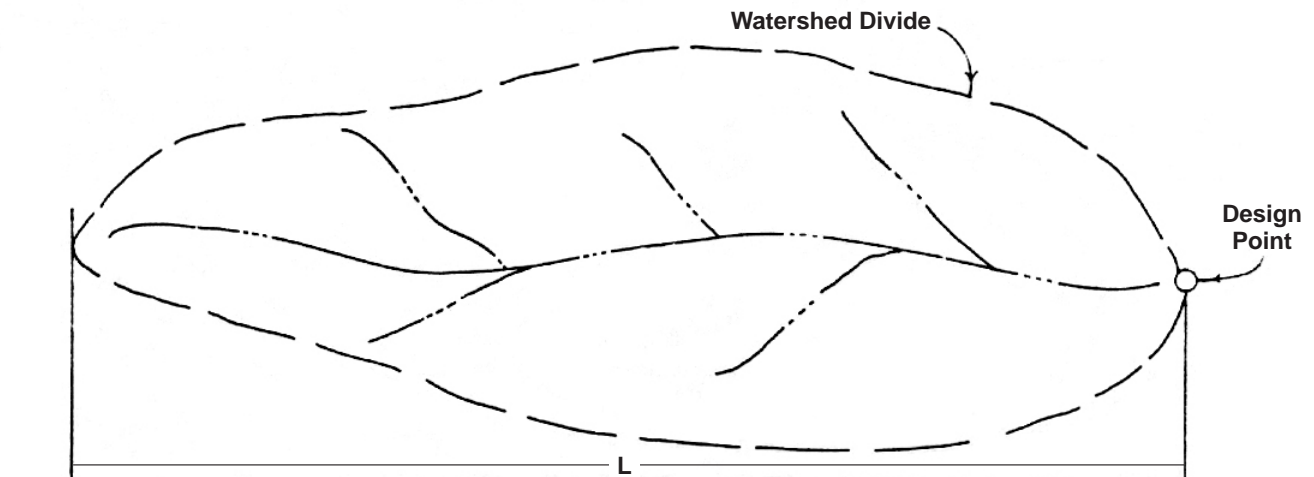
- (b) Urban Watersheds - Flow through a closed conduit where no additional flow can enter the system during the travel, length, velocity and T_t are determined using the peak flow in the conduit. In cases where the conduit is not closed and additional flow from a contributing subarea is added to the total flow during travel (e.g., street flow in a gutter), calculation of velocity and T_t is performed using an assumed average flow based on the total area (including upstream subareas) contributing to the point of interest. The Manning equation is usually used to determine velocity. Discharges for small watersheds typically range from 2 to 4 cfs per acre, depending on land use, drainage area, and slope and rainfall intensity.

Note: The MRM should be used to calculate the peak discharge when there is a junction from independent subareas into the drainage system. The effective slope is only for use with the Kirpich formula (not for use with the NRCS method).



SOURCE: California Division of Highways (1941) and Kirpich (1940)

**Nomograph for Determination of
Time of Concentration (T_c) or Travel Time (T_t) for Natural Watersheds**



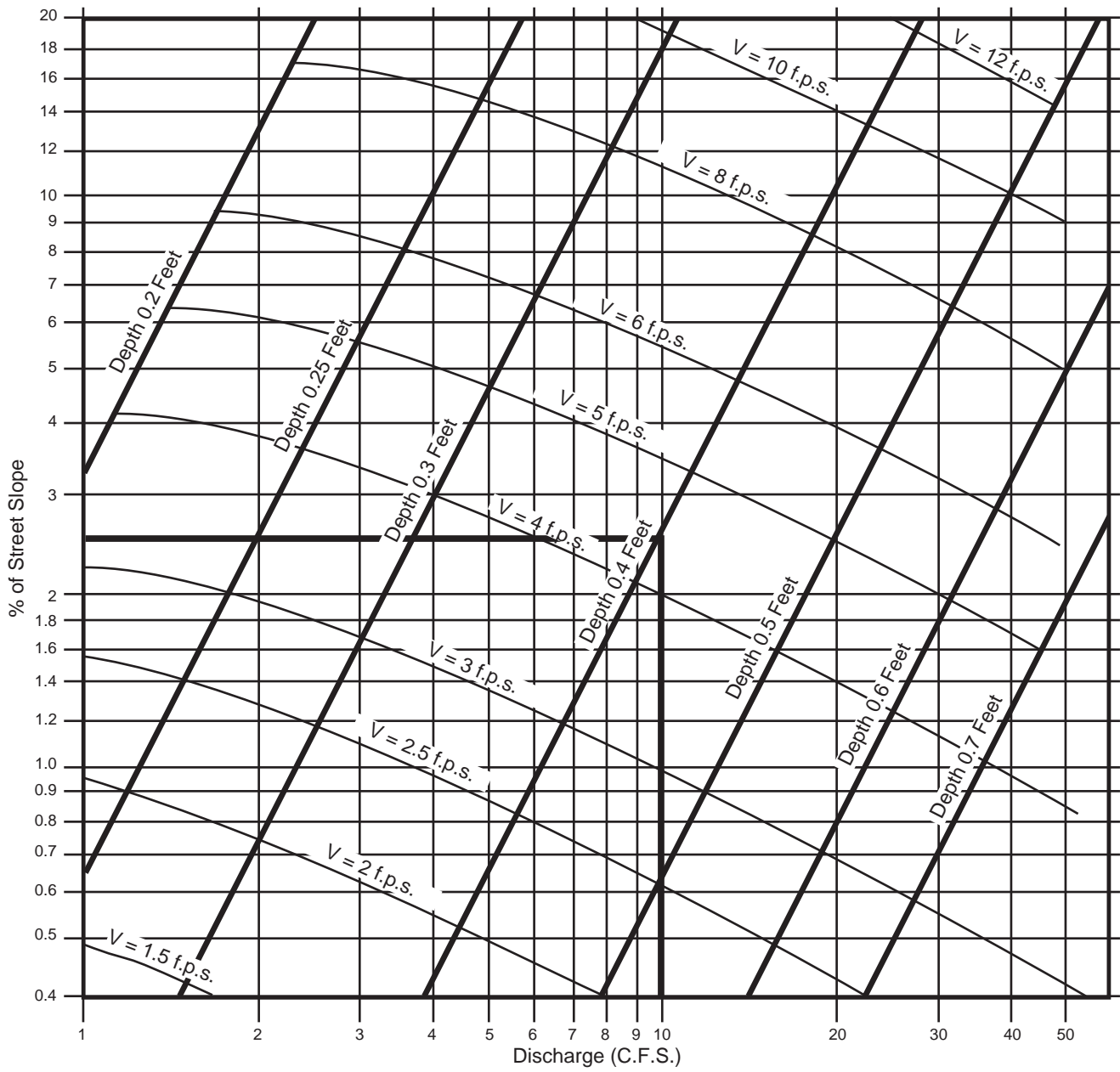
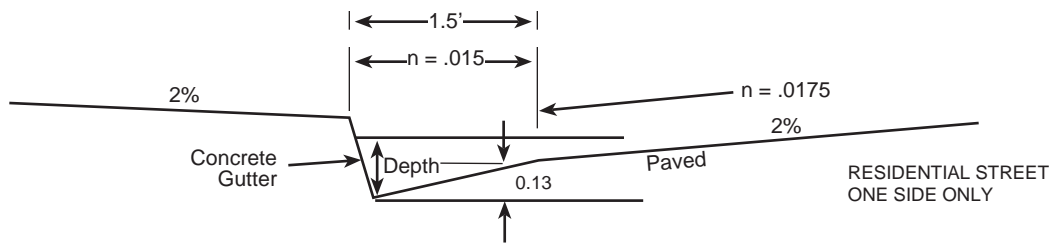
Area "A" = Area "B"

SOURCE: California Division of Highways (1941) and Kirpich (1940)

FIGURE

Computation of Effective Slope for Natural Watersheds

3-3

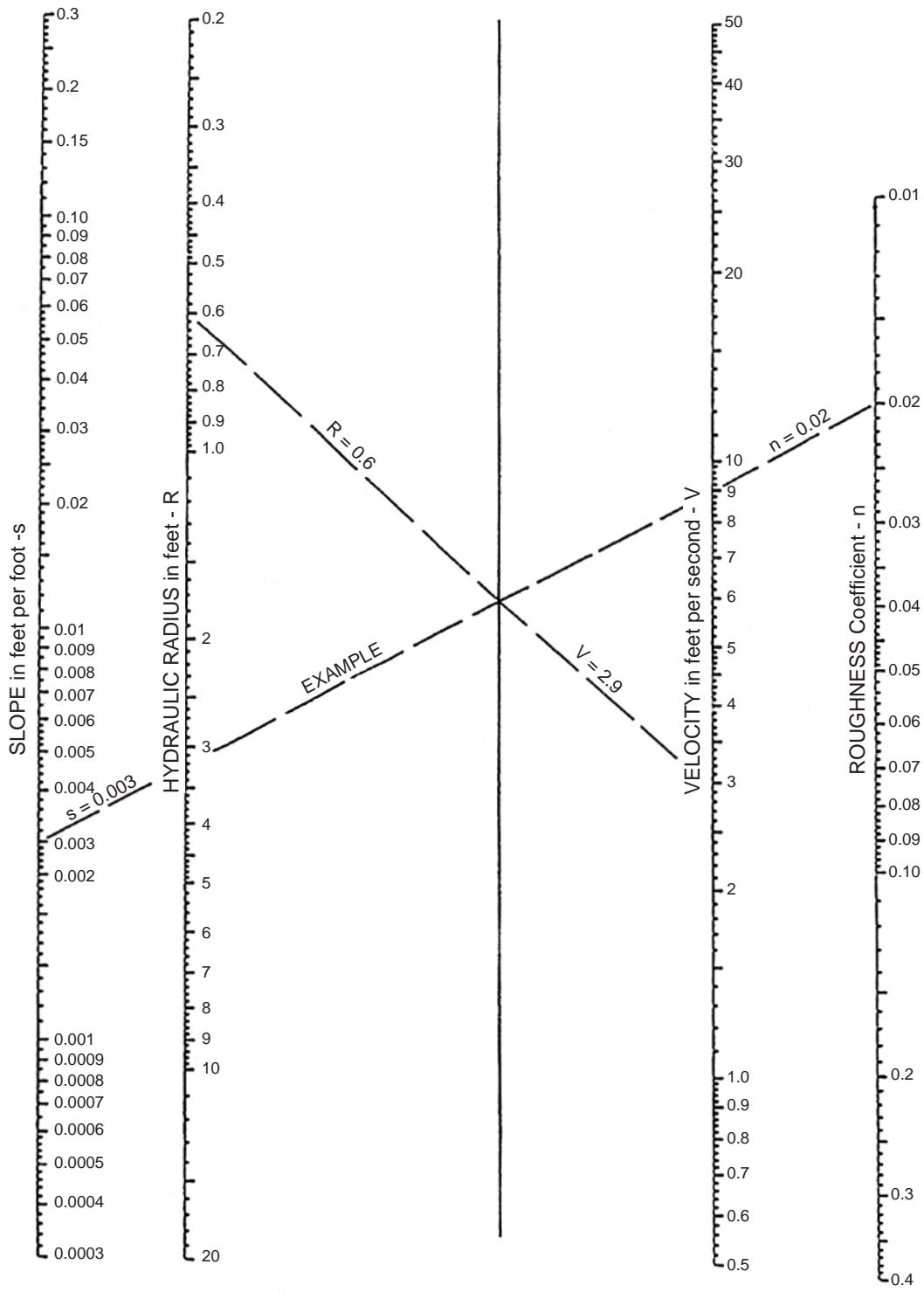


EXAMPLE:
 Given: $Q = 10$ $S = 2.5\%$
 Chart gives: Depth = 0.4, Velocity = 4.4 f.p.s.

SOURCE: San Diego County Department of Special District Services Design Manual

Gutter and Roadway Discharge - Velocity Chart

EQUATION: $V = \frac{1.49}{n} R^{2/3} s^{1/2}$



SOURCE: USDOT, FHWA, HDS-3 (1961)

Manning's Equation Nomograph

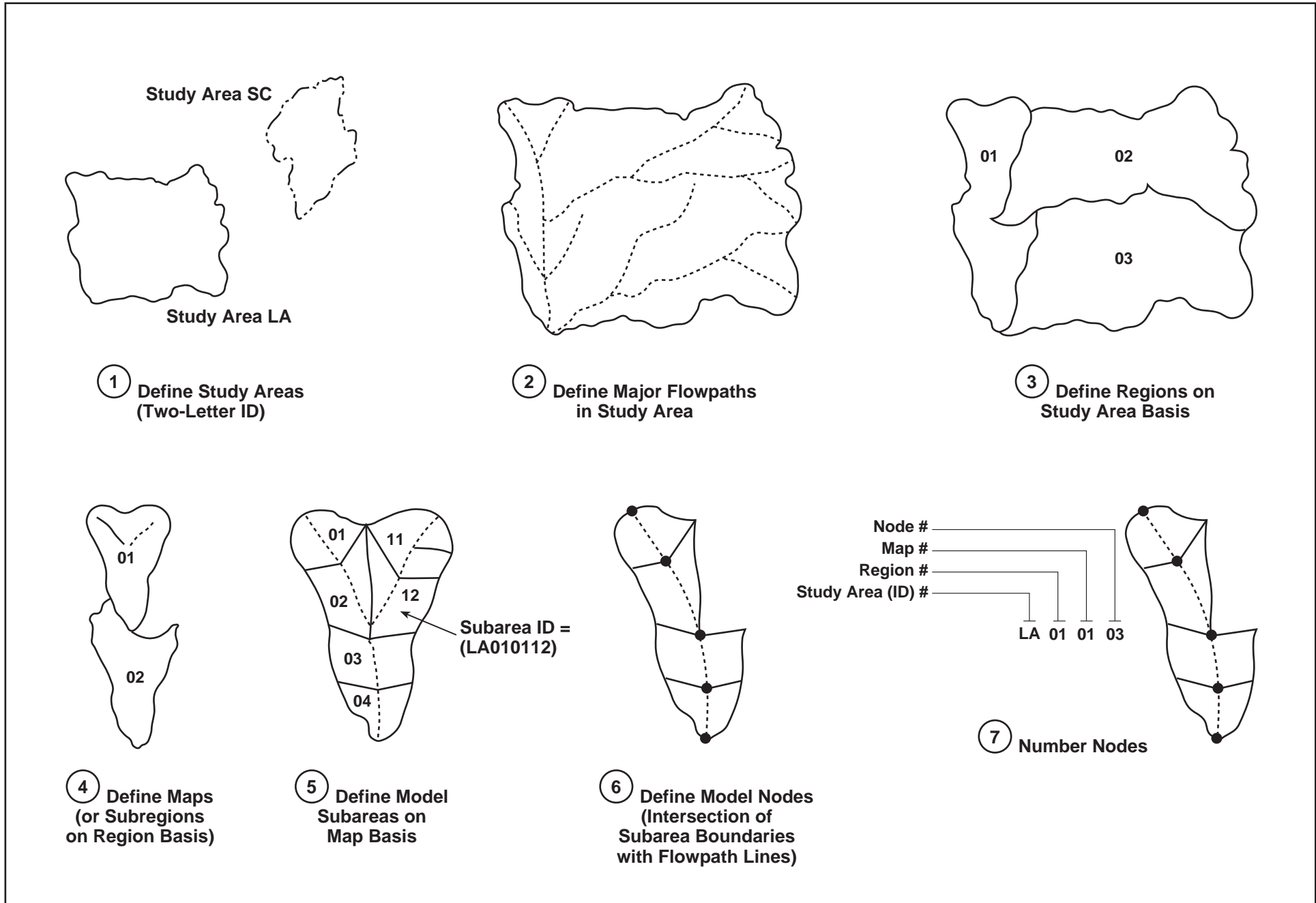
FIGURE

3-5

3.2 DEVELOPING INPUT DATA FOR THE RATIONAL METHOD

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

1. On a topographic base map, outline the overall drainage area boundary, showing adjacent drains, existing and proposed drains, and overland flow paths.
2. Verify the accuracy of the drainage map in the field.
3. Divide the drainage area into subareas by locating significant points of interest. These divisions should be based on topography, soil type, and land use. Ensure that an appropriate first subarea is delineated. For natural areas, the first subarea flow path length should be less than or equal to 4,000 feet plus the overland flow length (Table 3-2). For developed areas, the initial subarea flow path length should be consistent with Table 3-2. The topography and slope within the initial subarea should be generally uniform.
4. Working from upstream to downstream, assign a number representing each subarea in the drainage system to each point of interest. Figure 3-6 provides guidelines for node numbers for geographic information system (GIS)-based studies.
5. Measure each subarea in the drainage area to determine its size in acres (A).
6. Determine the length and effective slope of the flow path in each subarea.
7. Identify the soil type for each subarea.



GIS/Hydrologic Model
Data Base Linkage Setup:
Nodes, Subareas, Links

8. Determine the runoff coefficient (C) for each subarea based on Table 3-1. If the subarea contains more than one type of development classification, use a proportionate average for C . In determining C for the subarea, use future land use taken from the applicable community plan, Multiple Species Conservation Plan, National Forest land use plan, etc.
9. Calculate the CA value for the subarea.
10. Calculate the $\Sigma(CA)$ value(s) for the subareas upstream of the point(s) of interest.
11. Gather NOAA Atlas 14 precipitation intensity data pairs to be used for determining intensity at each time of concentration. Procedures to obtain data from NOAA Atlas 14 are provided in Appendix B of this manual.

See Section 3.3 for a description of the RM calculation process.

3.3 PERFORMING RATIONAL METHOD CALCULATIONS

This section describes the RM calculation process. Using the input data, calculation of peak flows and T_c 's should be performed as follows:

1. Determine T_i for the first subarea. Use Table 3-2 or Figure 3-1 as discussed in Section 3.1.4. If the watershed is natural, the travel time to the downstream end of the first subarea can be added to T_i to obtain the T_c . Refer to paragraph 3.1.4.2 (a).
2. Determine I for the subarea from NOAA Atlas 14 precipitation intensity data pairs. If T_i or T_c is less than 5 minutes, use the 5 minute time to determine intensity for calculating the flow. If T_i or T_c is between given data points from NOAA Atlas 14 (e.g., 8 minutes), use log-log interpolation to interpolate the intensity for T_i .
3. Calculate the peak discharge flow rate for the subarea, where $Q_p = \Sigma(CA) I$.
In case that the downstream flow rate is less than the upstream flow rate, due to the long travel time that is not offset by the additional subarea runoff, use the upstream peak flow for design purposes until downstream flows increase again.

4. Estimate the T_t to the next point of interest.
5. Add the T_t to the previous T_c to obtain a new T_c .
6. Continue with step 2, above, until the final point of interest is reached.

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

3.4 MODIFIED RATIONAL METHOD (FOR JUNCTION ANALYSIS)

The purpose of this section is to describe the steps necessary to develop a hydrology report for a small watershed using the MRM. It is necessary to use the MRM if the watershed contains junctions of independent drainage systems. The general process description for using this method, including an example of the application of this method, is described below.

The engineer should only use the MRM for drainage areas up to approximately 1 square mile in size. If the watershed will significantly exceed 1 square mile then the NRCS method described in Section 4 should be used. The engineer may choose to use either the RM or the MRM for calculations for up to an approximately 1-square-mile area and then transition the study to the NRCS method for additional downstream areas that exceed approximately 1 square mile. The transition process is described in Section 4.

3.4.1 Modified Rational Method General Process Description

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

values will be used for the MRM calculations. After the independent drainage systems have been combined, RM calculations are continued to the next point of interest.

3.4.2 Procedure for Combining Independent Drainage Systems at a Junction

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

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

Let Q_1 , T_1 , and I_1 correspond to the tributary area with the shortest T_c . Likewise, let Q_2 , T_2 , and I_2 correspond to the tributary area with the next longer T_c ; Q_3 , T_3 , and I_3 correspond to the tributary area with the next longer T_c ; and so on. When only two independent drainage systems are combined, leave Q_3 , T_3 , and I_3 out of the equation. Combine the independent drainage systems using the junction equation below:

Junction Equation: $T_1 < T_2 < T_3$

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

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

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

Calculate Q_{T1} , Q_{T2} , and Q_{T3} . Select the largest Q and use the T_c associated with that Q for further calculations (see the three Notes for options). If the largest calculated Q 's are equal (e.g., $Q_{T1} = Q_{T2} > Q_{T3}$), use the shorter of the T_c 's associated with that Q .

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

Note #1: At a junction of two independent drainage systems that have the same T_c , the tributary flows may be added to obtain the Q_p .

$$Q_p = Q_1 + Q_2; \text{ when } T_1 = T_2; \text{ and } T_c = T_1 = T_2$$

This can be verified by using the junction equation above. Let Q_3 , T_3 , and $I_3 = 0$. When T_1 and T_2 are the same, I_1 and I_2 are also the same, and T_1/T_2 and $I_2/I_1 = 1$. T_1/T_2 and I_2/I_1 are cancelled from the equations. At this point, $Q_{T1} = Q_{T2} = Q_1 + Q_2$.

Note #2: In the upstream part of a watershed, a conservative computation is acceptable. When the times of concentration (T_c 's) are relatively close in magnitude (within 10%), use the shorter T_c for the intensity and the equation $Q = \Sigma(CA)I$.

Note #3: An optional method of determining the T_c is to use the equation $I = \Sigma(CA)/Q$ to determine the intensity that would create the combined flow rate that has been determined from the junction equation, and then find the T_c that corresponds to that intensity from the NOAA Atlas 14 data, using log-log interpolation.

The advantage in this option is that the T_c is consistent with the peak flow Q and avoids inappropriate fluctuation in downstream flows in some cases.

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SECTION 4 THE SOIL CONSERVATION SERVICE – NRCS HYDROLOGIC METHOD

The Soil Conservation Service (SCS) (now called the Natural Resources Conservation Service [NRCS]) hydrologic method (NRCS hydrologic method) requires basic data similar to the RM: drainage area, a “runoff curve number” (CN) describing the proportion of rainfall that runs off, time to peak (T_p), the elapsed time from the beginning of unit effective rainfall to the peak flow for the point of concentration, and total rainfall (P). The NRCS approach, however, is more sophisticated in that it also considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. Results of the NRCS approach are more detailed, in the form of a runoff hydrograph. Details of the methodology can be found in the NRCS National Engineering Handbook (NEH), Section 4 (NEH-4) (USDA, 1985). The NRCS hydrologic method should be used for study areas approximately 1 square mile and greater in size. The NRCS hydrologic method may be used for the entire study area, or the RM or MRM may be used for approximately 1 square mile of the study area and then transitioned to the NRCS hydrologic method using the procedure described in Section 4.4.

The NRCS method includes the following basic steps:

1. Delineation of the watershed on a map and determination of watershed physical characteristics including location of centroid, total length and length to centroid, soil type, and land use/land treatment,
2. Determination of time to peak, the elapsed time from the beginning of unit effective rainfall to the peak flow for the point of concentration, and/or lag time, the elapsed time from the beginning of unit effective rainfall to the instant that the summation hydrograph for the point of concentration reaches 50% of ultimate discharge,

3. Determination of frequency of design storm, and determination of total rainfall amount for the design storm and precipitation zone number (PZN) for the watershed location,
4. Preparation of incremental rainfall distribution,
5. Adjustment of incremental rainfall depths based on watershed area,
6. Determination of composite curve number (CN) for the watershed, which will represent different combinations of land use and soil type within the drainage area and describe the proportion of rainfall that runs off,
7. Adjustment of CN based on the PZN Condition,
8. Determination of excess rainfall amounts using the PZN adjusted composite CN for the watershed and the depth-area adjusted incremental rainfall distribution,
9. Using the dimensionless unit hydrograph approach, development of the hydrograph of direct runoff from the drainage area.

4.1 CONCEPTS AND EQUATIONS OF THE NRCS HYDROLOGIC METHOD

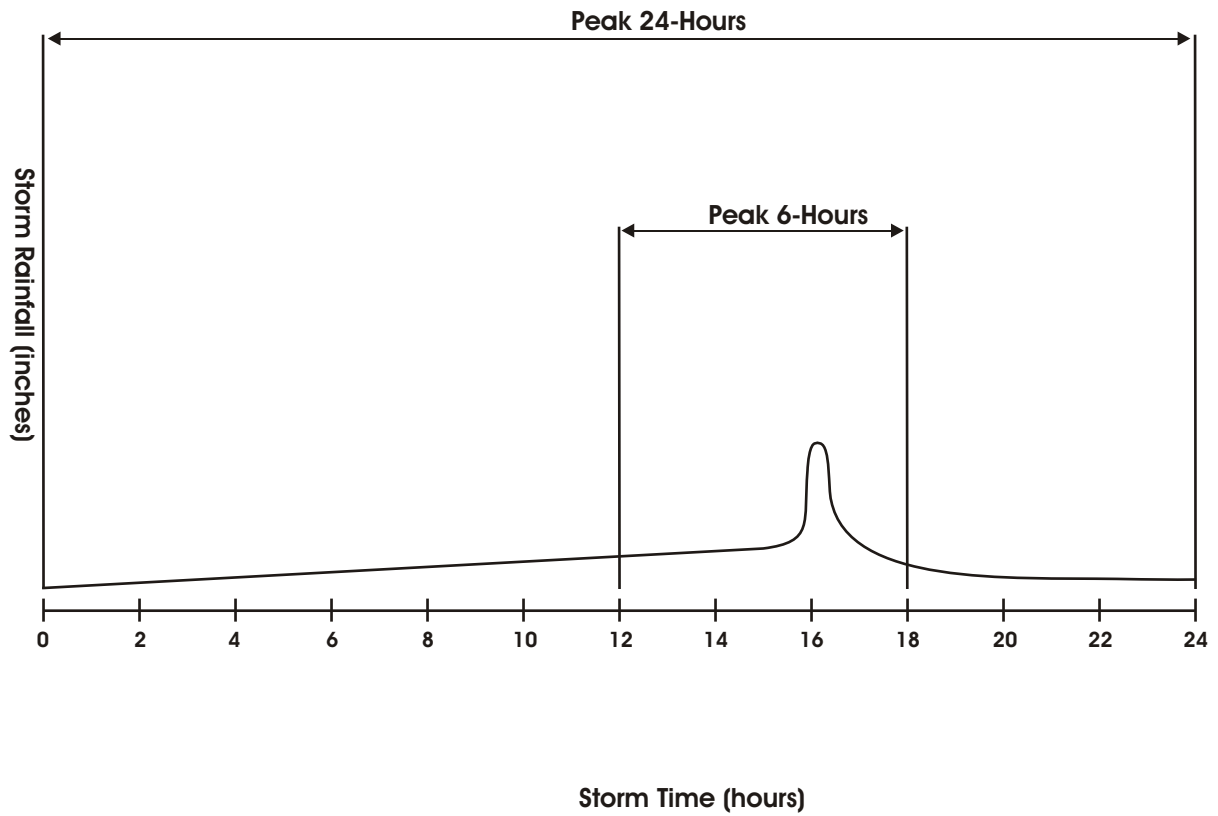
4.1.1 Rainfall Distribution

The hydrograph of storm runoff from a drainage area is based in part on the time distribution of rainfall during the storm. The variation in rainfall intensity that occurs from the beginning of the storm through the storm peak and the end of the storm is represented in the time distribution of rainfall. The time distribution of rainfall during a storm can be represented graphically as a hyetograph, a chart showing increments of average rainfall during successive units of time during a storm.

The rainfall distribution adopted for this manual is a nested storm pattern, based on the United States Army Corps of Engineers (USACE), Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) *Technical Reference Manual (USACE, 2000)*. A 24-hour nested storm shall be used for flood flow computations. The peak of the nested storm will occur at hour 16 of the 24-hour storm. The nested storm will be distributed about hour 16 of the 24-hour storm using a (2/3, 1/3) distribution. The nested storm pattern with (2/3, 1/3) distribution is shown in Figure 4-1. The nested storm is described below in Section 4.1.1.1, and the (2/3, 1/3) distribution is described below in Section 4.1.1.2.

The nested storm pattern with (2/3, 1/3) distribution supersedes the Type B and Type C rainfall distributions that were used in the 1993 edition of this manual for the westerly and easterly drainage areas of San Diego County, respectively. The nested storm pattern is appropriate for both the westerly and easterly drainage areas of San Diego County. A limitation of the Type B and Type C distributions was that each distribution was created for and applicable to the 6-hour and 24-hour durations only and required separate analyses to be prepared for each duration. In most cases, the 6-hour storm duration produced a higher peak flow rate, while the 24-hour storm duration generated a greater volume of runoff. Use of the nested storm pattern will eliminate the need for separate analyses for 6-hour and 24-hour storm durations. The Figures and Tables describing the 6-hour and 24-hour Type B and Type C rainfall distributions have been removed from this manual. The Figures and Tables describing these rainfall distributions can be obtained from the 1993 edition of this manual or from San Diego County DPWFCS if necessary for forensic study or other research.

In addition to the nested storm pattern with (2/3, 1/3) distribution, a rainfall depth-area adjustment based on the United States Department of Commerce, National Oceanic and Atmospheric Administration (NOAA) Atlas 14, *Precipitation-Frequency Atlas of the United States, Volume VI, California* has been adopted with this manual. The rainfall depth-area adjustment based on NOAA Atlas 14 supersedes the Pacific Coastal Climate Area Reduction Ratio and the Arid and Semiarid Climate Area Reduction Factor given in the 1993 edition of this manual. The rainfall depth-area adjustment is discussed below in Section 4.1.1.3.



Time Distribution of Rainfall:
Nested Storm Pattern, with $(2/3, 1/3)$
Distribution

FIGURE
4-1

4.1.1.1 Nested Storm Pattern

The nested storm pattern is a synthetic storm with the maximum rainfall intensities for a given storm frequency nested for duration between 5 minutes and 24 hours. The maximum 5 minutes of rainfall is nested within the maximum 10 minutes; the maximum 10 minutes is nested within the maximum 15 minutes; and so forth until the 24-hour storm pattern is developed. Figure 4-1 shows the nested storm pattern. This hypothetical storm pattern is referred to in HEC-HMS Technical Reference Manual as a “balanced storm”, because of the consistent depth-frequency relation used for each peak duration interval. Use of a balanced storm permits the construction and arrangement of a storm event such that an average rainfall intensity of a specified frequency is provided for all durations – including one that matches the time-response characteristics of the particular watershed being analyzed. Every watershed is sensitive to a particular duration of rainfall that will produce the peak discharge, usually a duration approximating the time of concentration of the watershed. A nested duration design storm, analogous to a “balanced hydrograph”, ensures that each watershed will receive the design frequency depth of rainfall for its critical duration. Durations longer or shorter than the critical duration have little effect on peak discharge, although longer durations have considerable effect on total volume of runoff. Section 4.3.2 provides further guidance for the creation of the rainfall distribution.

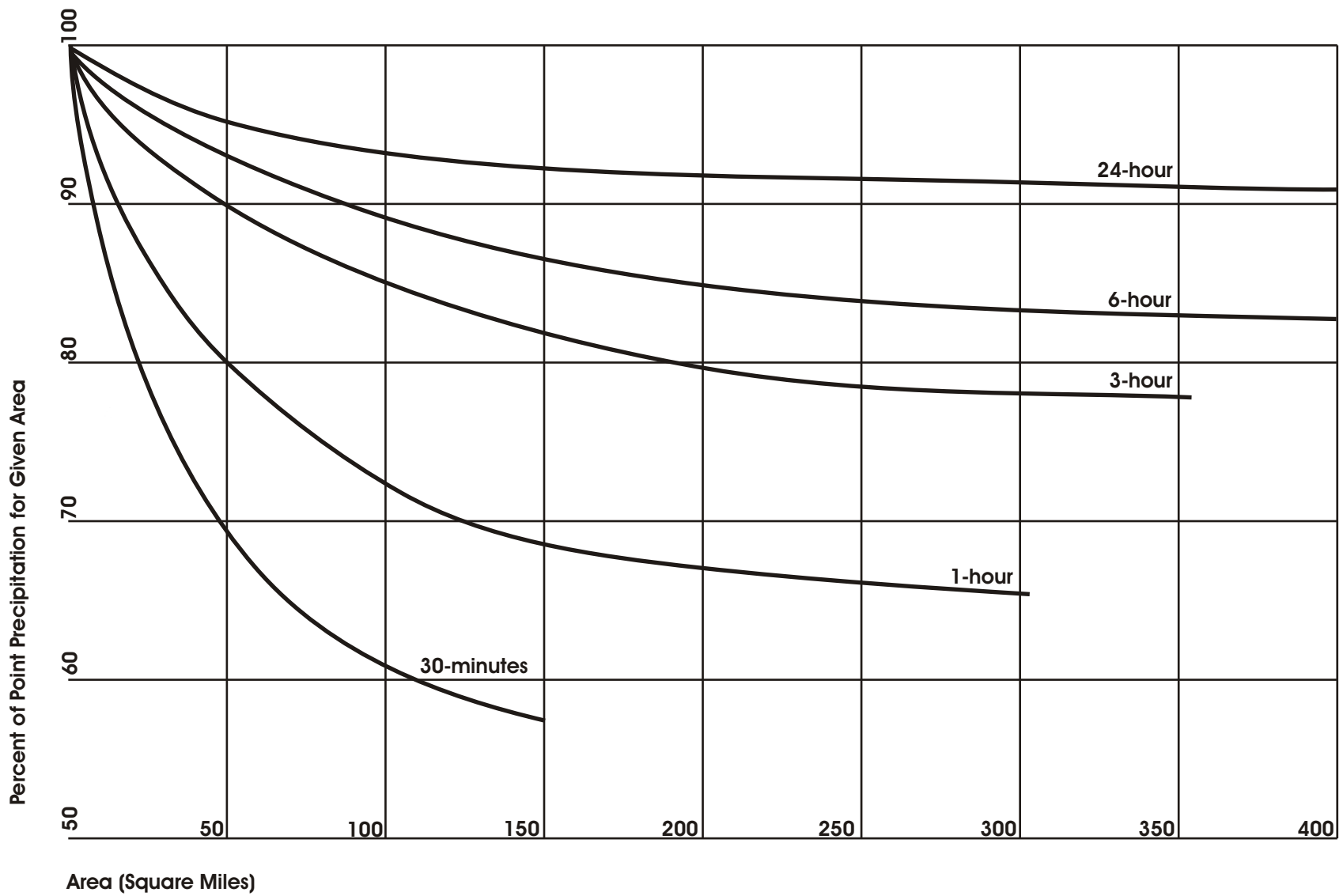
4.1.1.2 Shape of Rainfall Distribution Hyetograph

For preparation of the hyetograph (a chart showing increments of average rainfall during successive units of time during a storm) for a hydrologic study, increments of average rainfall shall be calculated using the methods described in Section 4.3.2, and a (2/3, 1/3) rainfall distribution nested about hour 16 of the 24-hour storm shall be used to distribute the rainfall increments. HEC-HMS Technical Reference Manual suggests a (1/2, 1/2) distribution in which the peak rainfall intensity is placed at the center of the storm; however, other distributions are not ruled out. A sensitivity analysis was performed for Orange County, California watersheds (Hromadka, 1987) to determine the effect of these two storm distributions on peak flow rates. It was concluded that a reasonable variation of the design storm pattern shape would have a negligible effect on the modeling output of

peak flow rate. However, the distribution of runoff volume varies within the runoff hydrograph depending upon the design storm pattern rainfall distribution. In reservoir or detention basin design the impact on total storage volume required would be significant. This is the reason the (2/3, 1/3) distribution was adopted for this manual. Figure 4-1 shows the (2/3, 1/3) distribution nested about hour 16 of the 24-hour storm.

4.1.1.3 Rainfall Depth-Area Adjustment

Precipitation frequency estimates from NOAA Atlas 14 (i.e., the rainfall data that will be used to build a hyetograph for a study watershed) are point estimates and are not directly applicable for an area. The average rainfall over a given area will be less than the maximum point value in the area. NOAA Atlas 14 establishes a rainfall depth-area adjustment that may be applied to the point rainfall values. Figure 4-2 gives the adjustment to the point rainfall value for various rainfall durations as a function of watershed area. Table 4-1 provides the depth-area adjustment data points that are built into the SDHydroTools computer program that is provided with this manual (the SDHydroTools computer program is discussed in Section 1 and use of the program for NRCS hydrologic method studies is discussed in Section 4.3). These data points were obtained from Figure 4-2. For consistency between studies, it is recommended that the depth-area adjustment factors be interpolated from Table 4-1. The depth-area adjustment may be applied for watersheds approximately 1 square mile or greater in size. The depth-area adjustment should be applied to the incremental rainfall amounts prior to arranging the incremental rainfall amounts in the (2/3, 1/3) distribution.



Source: NOAA Atlas 2 Precipitation-Frequency Atlas of the Western United States Volume IX-California, 1973

Rainfall Depth-Area Adjustment Curves

FIGURE

4-2

**Table 4-1
 RAINFALL DEPTH-AREA ADJUSTMENT DATA POINTS**

Watershed Area (square miles)	Rainfall Depth-Area Adjustment for Duration				
	30-Minute	1-Hour	3-Hour	6-Hour	24-Hour
0	1.000	1.000	1.000	1.000	1.000
5	0.942	0.970	0.980	0.985	0.990
10	0.900	0.947	0.970	0.980	0.985
20	0.834	0.900	0.952	0.963	0.975
30	0.768	0.858	0.932	0.950	0.964
40	0.730	0.830	0.915	0.940	0.958
50	0.692	0.800	0.900	0.928	0.952
60	0.663	0.778	0.883	0.920	0.948
70	0.645	0.760	0.872	0.912	0.945
80	0.630	0.746	0.862	0.904	0.942
90	0.620	0.735	0.853	0.896	0.938
100	0.610	0.722	0.845	0.890	0.935
125	0.588	0.700	0.830	0.878	0.930
150	0.572	0.685	0.818	0.865	0.925
175	0.572	0.672	0.808	0.858	0.922
200	0.572	0.666	0.798	0.851	0.918
225	0.572	0.660	0.790	0.845	0.915
250	0.572	0.655	0.787	0.842	0.914
300	0.572	0.652	0.782	0.838	0.912
350	0.572	0.652	0.780	0.830	0.910
400	0.572	0.652	0.780	0.828	0.908

4.1.2 Runoff Curve Number

The hydrograph of storm runoff from a drainage area is also based in part on the physical characteristics of the watershed. The principal physical watershed characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types, and land slope. The NRCS method uses a combination of soil conditions and land uses (ground cover) and land treatment (generally agricultural practices) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CNs), indicate the runoff potential of an area. The higher the CN, the higher the runoff potential. The CN does not account for land slope. However, in the NRCS hydrologic method land slope is accounted for in the determination of watershed lag time (see Section 4.1.3).

The CN values in Table 4-2 are suitable for preparing hydrographs in accordance with the methods shown in Chapters 10 and 16 of NEH-4 and summarized in Section 4.2 of this manual. The CN values are based on hydrologic soil group and land use/land treatment. Tables 4-3, 4-4, and 4-5 provide descriptions of some of the terms used in Table 4-2, including vegetative condition and cover density. See the glossary for descriptions of land uses and hydrologic conditions listed in Tables 4-2 through 4-5. When a drainage area has more than one land use, a composite CN can be calculated and used in the analysis (see Section 4.2.3). It should be noted that when composite CNs are used, the analysis does not take into account the location of the specific land uses but treats the drainage area as a uniform land use represented by the composite CN.

Note: The CN values in Table 4-2 are unadjusted for PZN Condition. These are suitable where the PZN adjustment factor = 2.0, which represents the average PZN Condition. The PZN Condition and PZN adjustment factor are discussed in Section 4.1.2.4.

**Table 4-2
 RUNOFF CURVE NUMBERS¹ FOR PZN CONDITION = 2.0**

Cover Description	Cover Treatment or Practice ²	Hydrologic Condition ³	Average Percent Impervious Area ⁴	Curve Numbers for Hydrologic Soil Groups:			
				A	B	C	D
Developing urban areas and newly graded areas (pervious areas only, no vegetation).....				77	86	91	94
Impervious areas: Paved parking lots, roofs, and driveways (excluding right-of-way).....				98	98	98	98
Residential districts by average lot size: ⁴							
1/8 acre or less (town houses).....			65%	77	85	90	92
1/4 acre.....			38%	61	75	83	87
1/3 acre.....			30%	57	72	81	86
1/2 acre.....			25%	54	70	80	85
1 acre.....			20%	51	68	79	84
2 acres.....			12%	46	65	77	82
Streets and roads.....	Paved; curbs and storm drains (excluding right-of-way).....			98	98	98	98
	Paved; open ditches (including right-of-way).....			83	89	92	93
	Gravel (including right-of-way).....			76	85	89	91
	Hard surface (including right-of-way).....			74	84	90	92
	Dirt (including right-of-way).....			72	82	87	89
Urban districts ⁴	Commercial and business.....		85%	89	92	94	95
	Industrial.....		72%	81	88	91	93
Western desert urban areas:							
Natural desert landscaping (pervious areas only) ⁵				63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders).....				96	96	96	96

Table 4-2 (Continued)
RUNOFF CURVE NUMBERS¹ FOR PZN CONDITION = 2.0

Cover Description	Cover Treatment or Practice ²	Hydrologic Condition ³	Average Percent Impervious Area ⁴	Curve Numbers for Hydrologic Soil Groups:			
				A	B	C	D
Close-seeded legumes or rotated pasture.....	Straight row	Poor.....	66	77	85	89	
		Good.....	58	72	81	85	
	Contoured.....	Poor.....	64	75	83	85	
		Good.....	55	69	78	83	
	Contoured and terraced	Poor.....	63	73	80	83	
		Good.....	51	67	76	80	
Cultivated land	Without conservation treatment		72	81	88	91	
	With conservation treatment		62	71	78	81	
Fallow.....	Bare soil		77	86	91	94	
		Poor.....	76	85	90	92	
		Good.....	74	83	88	90	
Farmsteads (buildings, lanes, driveways, and surrounding lots)			59	74	82	86	
		Poor.....	58	74	83	87	
		Fair.....	44	65	77	82	
Irrigated pasture.....		Good.....	33	58	72	79	
			(see glossary description)				
		Poor.....	57	73	82	86	
Orchards (deciduous)		Fair.....	44	65	77	82	
		Good.....	33	58	72	79	
		Poor.....	72	81	88	91	
Orchards (evergreen).....		Good.....	67	78	85	89	
		Poor.....	70	79	84	88	
		Good.....	65	75	82	86	
		Poor.....	72	81	88	91	
Row crops.....	Straight row	Good.....	67	78	85	89	
		Poor.....	70	79	84	88	
	Contoured.....	Good.....	65	75	82	86	
		Poor.....	72	81	88	91	

Table 4-2 (Continued)
RUNOFF CURVE NUMBERS¹ FOR PZN CONDITION = 2.0

Cover Description	Cover Treatment or Practice ²	Hydrologic Condition ³	Average Percent Impervious Area ⁴	Curve Numbers for Hydrologic Soil Groups:			
				A	B	C	D
Small grain	Straight row	Poor	65	76	84	88	
		Good.....	63	75	83	87	
	Contoured.....	Poor	63	74	82	85	
		Good.....	61	73	81	84	
Vineyards ⁶	Disked		76	85	90	92	
	Annual grass or legume cover .	Poor	65	78	85	89	
		Fair	50	69	79	84	
Annual grass (Dryland pasture).....		Good.....	38	61	74	80	
		Poor	67	78	86	89	
		Fair	50	69	79	84	
Barren.....		Good.....	38	61	74	80	
Meadow.....			78	86	91	93	
		Poor	63	77	85	88	
		Fair	51	70	80	84	
Open space (lawns, parks, golf courses, cemeteries, etc.) ⁷	Grass cover <50%	Good.....	30	58	72	78	
		Poor	68	79	86	89	
		Fair	49	69	79	84	
Pasture or range land	Grass cover >75%	Good.....	39	61	74	80	
		Poor	68	79	86	89	
		Fair	49	69	79	84	
Perennial grass.....		Good.....	39	61	74	80	
		Poor	67	79	86	89	
		Fair	50	69	79	84	
		Good.....	38	61	74	80	

Table 4-2 (Continued)
RUNOFF CURVE NUMBERS¹ FOR PZN CONDITION = 2.0

Cover Description	Cover Treatment or Practice ²	Hydrologic Condition ³	Average Percent Impervious Area ⁴	Curve Numbers for Hydrologic Soil Groups:			
				A	B	C	D
Turf ⁸		Poor.....		58	74	83	87
		Fair.....		44	65	77	82
		Good.....		33	58	72	79
Water surfaces (during floods)				97	98	99	99
Broadleaf chaparral		Poor.....		53	70	80	85
		Fair.....		40	63	75	81
		Good.....		31	57	71	78
Desert shrub—major plants include saltbush, greasewood, creosotebush, blackbrush, bursage, palo verde, mesquite, and cactus		Poor.....		63	77	85	88
		Fair.....		55	72	81	86
		Good.....		49	68	79	84
Herbaceous—mixture of grass, weeds, and low-growing brush, with brush the minor element		Poor.....		9	80	87	93
		Fair.....		9	71	81	89
		Good.....		9	62	74	85
Narrowleaf chaparral		Poor.....		71	82	88	91
		Fair.....		55	72	81	86
Oak-aspen—mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple, and other brush		Poor.....		9	66	74	79
		Fair.....		9	48	57	63
		Good.....		9	30	41	48
Open brush		Poor.....		62	76	84	88
		Fair.....		46	66	77	83
		Good.....		41	63	75	81

Table 4-2 (Continued)
RUNOFF CURVE NUMBERS¹ FOR PZN CONDITION = 2.0

Cover Description	Cover Treatment or Practice ²	Hydrologic Condition ³	Average Percent Impervious Area ⁴	Curve Numbers for Hydrologic Soil Groups:			
				A	B	C	D
Pinyon-juniper-pinyon, juniper, or both; grass understory		Poor.....		⁹	75	85	89
		Fair.....		⁹	58	73	80
		Good.....		⁹	41	61	71
Sagebrush with grass understory		Poor.....		⁹	67	80	85
		Fair.....		⁹	51	63	70
		Good.....		⁹	35	47	55
Wood or forest land.....		Thin stand, poor cover		45	66	77	83
		Good cover.....		25	55	70	77
Woods (woodland).....		Poor.....		45	66	77	83
		Fair.....		36	60	73	79
		Good.....		28	55	70	77
Woodland-grass combination.....		Poor.....		57	73	82	86
		Fair.....		44	65	77	82
		Good.....		33	58	72	79

¹ Average runoff condition, and $I_a = 0.2S$.

² Hydrologic practices described as “straight row” and “contoured” are defined in the glossary.

³ For definition of hydrologic condition, see Tables 4-3, 4-4, and 4-5.

⁴ 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. If the impervious area is not directly connected, the NRCS method has an adjustment to reduce the effect.

⁵ Composite CNs for natural desert landscaping should be computed based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CNs are assumed equivalent to desert shrub in poor hydrologic condition.

⁶ See glossary.

⁷ CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

⁸ Includes lawns, cemeteries, golf courses and parks with ground cover of mowed and irrigated perennial grass.

⁹ CNs for Group A have not been developed.

Table 4-3

CLASSIFICATION OF NATIVE PASTURE OR RANGE

Vegetative Condition	Hydrologic Condition
Heavily grazed. Has no mulch or has plant cover on less than 50% of the area.	Poor
Not heavily grazed. Has plant cover on 50% to 75% of the area.	Fair
Lightly grazed. Has plant cover on more than 75% of the area.	Good

Table 4-4

AIR-DRY WEIGHT CLASSIFICATION OF NATIVE PASTURE OR RANGE

Cover density	Plant and litter air-dry weight (tons per acre):		
	Less than 0.5	0.5 to 1.5	More than 1.5
Less than 50%	Poor	Poor+	Fair
50% to 75%	Poor+	Fair	Fair+
More than 75%	Fair	Fair+	Good

Table 4-5

CLASSIFICATION OF WOODS

Vegetative Condition	Hydrologic Condition
Heavily grazed or regularly burned. Litter, small trees, and brush are destroyed.	Poor
Grazed but not burned. There may be some litter but these woods are not protected.	Fair
Protected from grazing. Litter and shrubs cover the soil.	Good

4.1.2.1 Hydrologic Soil Group

Soil properties influence the relationship between rainfall and runoff since soils have differing rates of infiltration. Based on infiltration rates, the NRCS has divided soils into four hydrologic soil groups.

Group A

Soils have high infiltration rate when thoroughly wetted; chiefly deep, well-drained to excessively drained sand, gravel, or both. Rate of water transmission is high; thus runoff potential is low.

Group B

Soils have moderate infiltration rate when thoroughly wetted; chiefly soils that are moderately deep to deep, moderately well drained to well drained, and moderately coarse textured. Rate of water transmission is moderate.

Group C

Soils have slow infiltration rate when thoroughly wetted; chiefly soils that have a layer impeding downward movement of water, or moderately fine to fine textured soils that have a slow infiltration rate. Rate of water transmission is slow.

Group D

Soils have very slow infiltration rate when thoroughly wetted; chiefly clays that have a high shrink-swell potential, soils that have a high permanent water table, soils that have a claypan or clay layer at or near the surface, or soils that are shallow over nearly impervious material. Rate of water transmission is very slow.

A list of soils throughout San Diego County and their hydrologic classification is located on the map in Appendix A. Soil Survey maps can be obtained from local NRCS offices for use in estimating soil type. GIS soil data layer can also be obtained from [SanGIS](#). Consideration should be given to the effects of urbanization on the natural hydrologic soil group. If heavy equipment can be expected to compact the soil during construction or if

grading will mix the surface and subsurface soils, appropriate changes should be made in the soil group selected.

4.1.2.2 Land Use/Land Treatment (Ground Cover)

Ground cover is used in combination with soil type to determine CN. Ground cover includes both land use and land treatment. Land use is defined as the type of watershed cover and includes every kind of vegetation, litter, mulch, and fallow as well as nonagricultural uses such as water surfaces (lakes, swamps, etc.) and impervious surfaces (roads, roofs, etc.). Land treatment applies mainly to agricultural land uses and it includes mechanical practices such as contouring or terracing and management practices such as grazing control or rotation of crops.

The NRCS has developed ground cover maps for San Diego County outlining vegetative and man-made cover. Ground cover maps can be obtained from local NRCS offices. GIS land cover data layer can also be obtained from [SanGIS](#). These maps may be used to determine the ground cover in the study area. Information on land use and treatment may also be obtained either by observation or by measurement of plant and litter density on sample areas. The CN values to be used for engineering design should correspond to the land use during the season for which maximum runoff is expected.

4.1.2.3 Urban Modifications

The urbanization of an area influences the relationship between rainfall and runoff by changing the percentage of impervious ground cover and changing the travel path of the runoff by concentrating flow in ditches, swales, gutters, channels, or pipes. The percentage of impervious area and the means of conveying runoff from impervious areas to the drainage system must be considered in computing CN for urban areas. Specifically, whether the impervious areas connect directly to the drainage system or outlet onto lawns or other pervious areas where infiltration can occur prior to entering the storm drain system must be considered.

The CN values given in Table 4-2 for urban land uses are based on directly connected impervious areas and specific assumed percentages of impervious area. An impervious area is considered directly connected if runoff from it flows directly into the drainage system. It is also considered directly connected if runoff from it occurs as concentrated shallow flow that runs over pervious areas (such as flow in a swale) and then into a drainage system. The CN values given in Table 4-2 were developed on the assumptions that:

- (a) pervious urban areas are equivalent to pasture in good hydrologic condition, and
- (b) impervious areas have a CN of 98 and are directly connected to the drainage system.

It is possible that CN values from urban areas could be reduced by not directly connecting impervious surfaces to the drainage system, but allowing runoff to flow as sheet flow over significant pervious areas. Sections 4.2.3.1 and 4.2.3.2 describe the method for adjusting CN values for land uses where impervious areas are not directly connected.

4.1.2.4 PZN Condition

The precipitation zone number (PZN) map provided in Appendix C was developed to reflect orographic effects to the distribution of rainfall in San Diego County. The basic categories of coast, foothill, mountain, and desert were selected as precipitation zones 1.0, 2.0, 3.0, and 4.0, respectively, because the NWS used these terms in forecasting rainfall amounts and because the county is divided into these climatic zones for agricultural purposes. The lines between precipitation zones were adjusted to conform more closely to isopluvial lines of equal precipitation that were developed to create the isopluvial maps that were provided in previous versions of this manual. PZNs are read from the PZN map provided in Appendix C.

The hydrograph of storm runoff from a drainage area is based in part on the degree of watershed wetness at the beginning of the storm. The index of watershed wetness used

with the runoff estimation method is PZN Condition. The PZN adjustment factor is a factor used to adjust the runoff curve number (Section 4.1.2) for the watershed to the appropriate PZN Condition. Three levels of PZN Condition are used (the PZN adjustment factors correspond to these PZN Conditions):

PZN Condition = 1.0. Lowest runoff potential. The watershed soils are dry enough for satisfactory plowing or cultivation to take place.

PZN Condition = 2.0. The average condition.

PZN Condition = 3.0. Highest runoff potential. The watershed is practically saturated from antecedent rains.

The CNs given in Table 4-2 are for PZN Condition = 2.0 (PZN adjustment factor = 2.0). CNs must be adjusted for PZN Condition based on the storm frequency and the location of the watershed within the county. The location of the watershed within the county is represented by the PZN from the map provided in Appendix C. The PZN adjustment factors for combinations of PZN and storm frequency are given in Table 4-6. The PZN adjustment factor can also be approximated by multiplying the previous 5 days of accumulated rainfall by 1.4. The maximum PZN adjustment factor is 3.0, representing PZN Condition 3.0, a saturated condition. Since the PZN adjustment factor is based on the previous 5-day precipitation amounts before major storms, a statistical analysis of these data provided the values listed in Table 4-6. Adjustment from PZN Condition 2.0 to other PZN Conditions can be accomplished by using the information shown in Tables 4-6 and 4-10 (see Section 4.2.4 for Table 4-10).

The adjustment for PZN Condition may be made to the composite CN for the watershed. It is not necessary to make the PZN Condition adjustment to each of the CNs for the different combinations of ground cover and soil group within the watershed before calculating the composite CN.

Table 4-6

**PZN ADJUSTMENT FACTORS FOR FLOW COMPUTATIONS
 (San Diego County)**

Storm Frequency	Coast (PZN = 1.0)	Foothills (PZN = 2.0)	Mountains (PZN = 3.0)	Desert (PZN = 4.0)
Less than 35-year return period	1.5	2.5	2.0	1.5
Greater than or equal to 35-year return period	2.0	3.0	3.0	2.0

Notes: PZN is the precipitation zone number (see Map, Appendix C). The PZN adjustment factor represents the PZN Condition that the CN for the watershed should be adjusted to.

4.1.3 Rainfall-Runoff Relationship

A relationship between accumulated rainfall and accumulated runoff was derived by NRCS from experimental plots for numerous soils and vegetative cover conditions. The following NRCS runoff equation is used to estimate direct runoff from 24-hour or 6-hour storm rainfall. The equation is:

$$Q_a = \frac{(P - I_a)^2}{(P - I_a) + S} \tag{Eq. 4-1}$$

- where: Q_a = accumulated direct runoff (in)
- P = accumulated rainfall (potential maximum runoff) (in)
- I_a = initial abstraction including surface storage, interception, evaporation, and infiltration prior to runoff (in)
- S = potential maximum soil retention (in)

S is based on the CN for the drainage area. The equation is:

$$S = 1000/CN - 10 \tag{Eq. 4-2}$$

An empirical relationship used in the NRCS method for estimating I_a is:

$$I_a = 0.2S \quad (\text{Eq. 4-3})$$

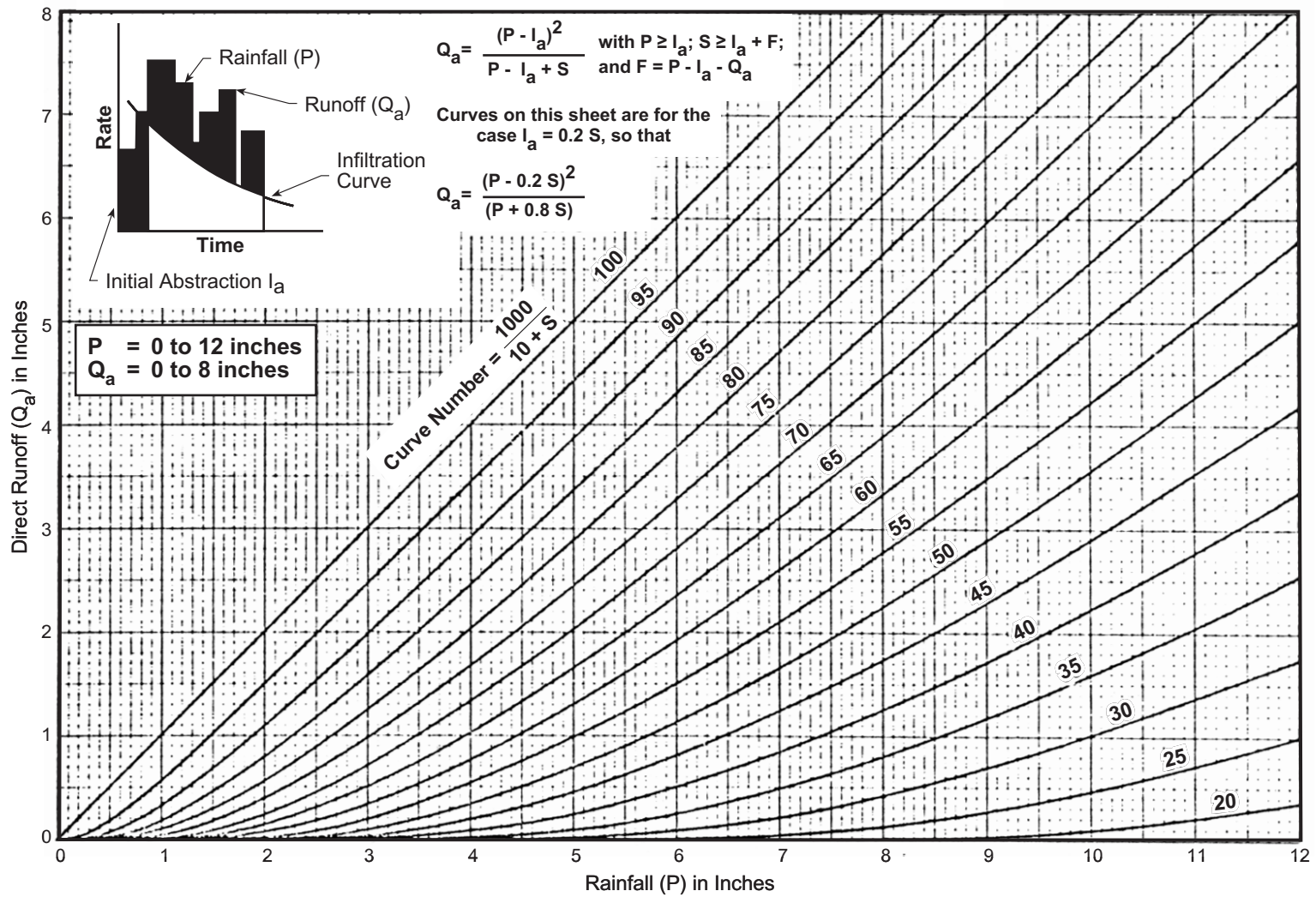
This is an average value that may be adjusted for flatter areas with more depressions if calibration data exists to substantiate the adjustment.

Substituting 0.2S for I_a in equation 4-1, the equation becomes:

$$Q_a = \frac{(P - 0.2S)^2}{(P + 0.8S)} \quad (\text{Eq. 4-4})$$

Equation 4-4 is subject to the limitation $P \geq 0.2S$.

Figure 4-3 shows a graphical solution of this equation. For example, 4.1 inches of direct runoff would result if 5.8 inches of rainfall occurred on a watershed with a CN of 85.



SOURCE: TR-55, Second Ed., June 1986

NRCS Solution of the Runoff Equation

FIGURE

4-3

4.1.4 Unit Hydrograph

The hydrograph of outflow from a drainage area is the sum of the elemental hydrographs from all the subareas, modified by the effects of transit time through the drainage area and storage in the stream channels. The NRCS method for estimating peak discharge and hydrographs is based on the unit hydrograph concept. The following discussion is taken from Chapter 16 of NEH-4.

To generate the peak discharge estimates and runoff hydrograph using the NRCS method, a rainfall distribution is used to estimate the variation in rainfall during the design storm event. Rainfall is divided into small time increments. The runoff hydrograph is then generated by developing incremental unit hydrographs for the different incremental rainfall time periods. The general procedure is to calculate the accumulated runoff (Q_a) using the NRCS derived equations for estimating Q_a presented below. Incremental values of Q_a are then obtained for each successive time period. These values are multiplied by the unit hydrograph peak to produce a peak value for the incremental hydrograph. The peak discharge values for incremental hydrographs provide the value for the ordinate. If the hand calculation method is used, the base of each triangle and the point of time at which the peak occurs are obtained by calculating the time from the beginning of runoff to the peak of the runoff, and the time from the peak of the runoff to the point where there is zero runoff. The result is a series of storm increments that when added together provide the runoff hydrograph.

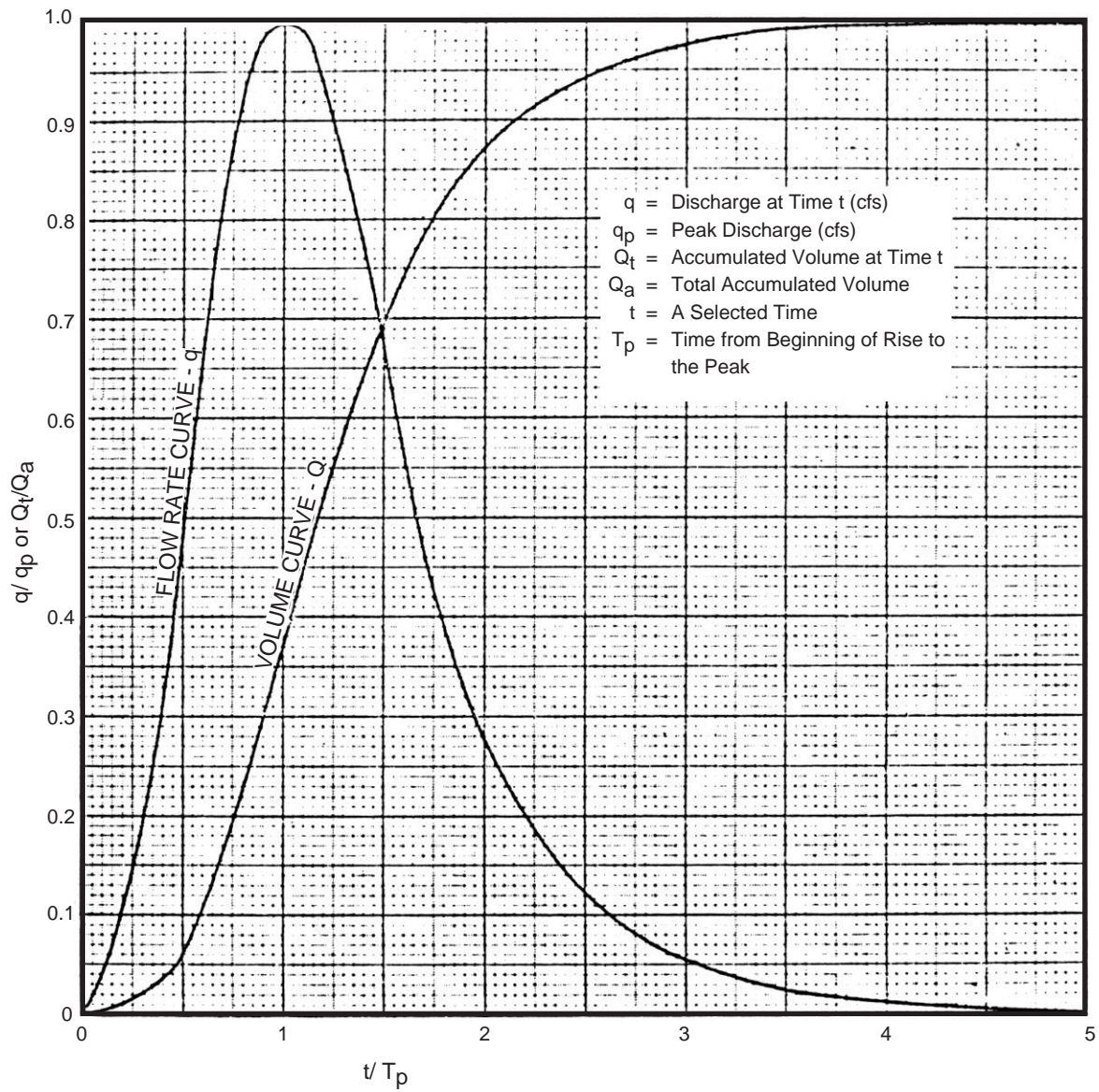
Since the physical characteristics of the drainage area, including shape, size, and slope, are constant from one storm to the next, the unit hydrograph approach assumes that there is considerable similarity in the shape of hydrographs from storms of similar rainfall characteristics (duration and intensity). Thus, the unit hydrograph is a typical hydrograph for the drainage area with a runoff volume under the hydrograph equal to 1 inch from a storm of specified duration. For a storm of the same duration but with a different amount of runoff, the hydrograph of direct runoff can be expected to have the same time base as the unit hydrograph and ordinates of flow proportional to the incremental runoff volume. Thus, a storm that produces 2 inches of runoff would have a hydrograph with ordinates of

flow equal to twice the ordinates of flow of the unit hydrograph. With 0.5 inches of runoff, the ordinates of flow of the hydrograph would be one-half of the ordinates of flow of the unit hydrograph.

The fundamental principles of invariance and superposition make the unit hydrograph an extremely flexible tool for developing synthetic hydrographs. The hydrograph of surface runoff from a watershed due to a given pattern of rainfall is invariable, and the hydrograph resulting from a given pattern of excess rainfall can be built up by superimposing the unit hydrograph due to the separate amounts of excess rainfall occurring in each unit period. This includes the principle of proportionality by which the ordinates of the hydrograph are proportional to the volume of excess rainfall.

The unit time or “unit hydrograph duration” is the optimum duration for occurrence of excess rainfall. In general, this unit time is approximately 20% of the time interval between the beginning of runoff from a short high-intensity storm and the peak discharge of the corresponding runoff.

The “storm duration” is the actual duration of the excess rainfall. The duration varies with actual storms. The dimensionless unit hydrograph used by NRCS (Figure 4-4) was developed by Victor Mockus. It was derived from a large number of natural unit hydrographs from watersheds varying widely in size and geographical locations. This dimensionless curvilinear hydrograph, also shown in Table 4-7, has its ordinate values expressed in a dimensionless ratio q/q_p or Q_t/Q_a and its abscissa values as t/T_p . This unit hydrograph has a point of inflection approximately 1.70 times the time to peak and the time-to-peak 0.2 of the time of base (T_b).



Dimensionless Unit Hydrograph and Volume Curve

FIGURE

4-4

Table 4-7

**RATIOS FOR THE NATURAL RESOURCES CONSERVATION SERVICE
 DIMENSIONLESS UNIT HYDROGRAPH***

Values for (t/T _p) Increments ≈ 0.1			Values for (t/T _p) Increments = 0.2	
Time Ratios (t/T _p)	Discharge Ratios (q/q _p)	Mass Curve Ratios (Q/Q _a)	Time Ratios (t/T _p)	Discharge Ratios (q/q _p)
0	0.000	0.000	0	0.000
0.1	0.030	0.001	0.2	0.100
0.2	0.100	0.006	0.4	0.310
0.3	0.190	0.017	0.6	0.660
0.4	0.310	0.035	0.8	0.930
0.5	0.470	0.065	1.0	1.000
0.6	0.660	0.107	1.2	0.930
0.7	0.820	0.163	1.4	0.780
0.8	0.930	0.228	1.6	0.560
0.9	0.990	0.300	1.8	0.390
1.0	1.000	0.375	2.0	0.280
1.1	0.990	0.450	2.2	0.207
1.2	0.930	0.522	2.4	0.147
1.3	0.860	0.589	2.6	0.107
1.4	0.780	0.650	2.8	0.077
1.5	0.680	0.705	3.0	0.055
1.6	0.560	0.751	3.2	0.040
1.7	0.460	0.790	3.4	0.029
1.8	0.390	0.822	3.6	0.021
1.9	0.330	0.849	3.8	0.015
2.0	0.280	0.871	4.0	0.011
2.2	0.207	0.908	4.2	0.010
2.4	0.147	0.934	4.4	0.007
2.6	0.107	0.953	4.6	0.003
2.8	0.077	0.967	4.8	0.001
3.0	0.055	0.977	5.0	<u>0.000</u>
3.2	0.040	0.984		
3.4	0.029	0.989		Total = 6.67
3.6	0.021	0.993		
3.8	0.015	0.995		
4.0	0.011	0.997		
4.5	0.005	0.999		
5.0	0.000	1.000		

*Runoff = 1 inch

Source: U.S. Department of Commerce, NEH4, 1985.

Elements of a Unit Hydrograph

The dimensionless curvilinear unit hydrograph (Figure 4-4) has 37.5% of the total volume in the rising side, which is represented by one unit of time and one unit of discharge. The dimensionless unit hydrograph also can be represented by an equivalent triangular hydrograph having the same units of time and discharge, thus having the same percent of volume in the rising side of the triangle (Figure 4-5).

This allows the base of the triangle to be solved in relation to the T_p using the geometry of triangles. Solving for the base length of the triangle, if one unit of time T_p equals 0.375 of volume:

$$T_b = \frac{1.00}{0.375} = 2.67 \text{ units of time,}$$

$$T_r = T_b - T_p = 1.67 \text{ units of time or } 1.67 T_p$$

where: T_b = time of base
 T_p = time to peak
 T_r = recession time

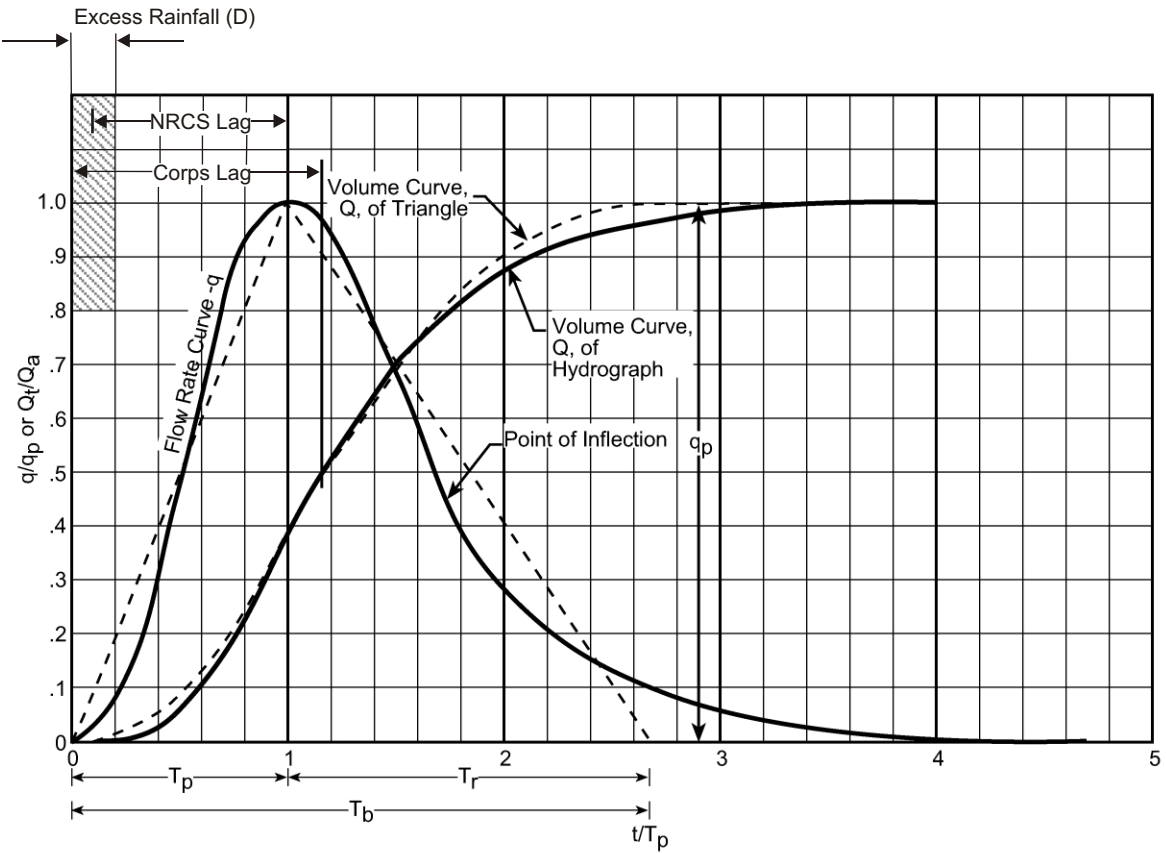
These relationships are useful in developing the peak rate equation for use with the dimensionless unit hydrograph.

Peak Rate Equation

From Figure 4-5 the total volume under the triangular unit hydrograph is:

$$Q_a = \frac{q_p T_p}{2} + \frac{q_p T_r}{2} = \frac{q_p}{2} (T_p + T_r) \quad (\text{Eq. 4-5})$$

or, $2Q_a = q_p (T_p + T_r)$



Ref: NEH4, March 1985

Dimensionless Curvilinear Unit Hydrograph
and Equivalent Triangular Hydrograph

FIGURE

4-5

With Q_a in inches, T in hours, and area set at unity, solve for peak rate q_p .

$$q_p = \frac{2Q_a}{T_p + T_r} = \frac{2Q_a}{T_p (1.0 + T_r/T_p)} \quad (\text{inches per hour}) \quad (\text{Eq. 4-6})$$

Let

$$K = \frac{2}{1 + \frac{T_r}{T_p}} \quad (\text{Eq. 4-7})$$

Therefore,

$$q_p = \frac{KQ_a A}{T_p} \quad \text{for a unit Drainage Area} \quad (\text{Eq. 4-8})$$

$$(A = 1 \text{ square mile})$$

In making the conversion from inches per hour to cubic feet per second and putting the equation in terms ordinarily used, including drainage area, A , in square miles, and the time, T , in hours, equation 4-12 becomes the general equation:

$$q_p = \frac{645.33 K A Q_a}{T_p} \quad (\text{Eq. 4-9})$$

Where q_p is peak discharge in cubic feet per second and the conversion factor 645.33 changes square mile inches per hour to cubic feet per second.

The relationship of the triangular unit hydrograph, $T_r = 1.67 T_p$, gives $K = 0.75$. Then substituting into equation 4-13 gives:

$$q_p = \frac{K_s A Q_a}{T_p} \quad (\text{Eq. 4-10})$$

$$\text{with } K_s = 484$$

K_s is a constant reflecting both the conversion of units and the shape of the hydrograph.

Any change in the dimensionless unit hydrograph reflecting a change in the percentage of volume under the rising side would cause a corresponding change in the shape factor associated with the triangular hydrograph and therefore a change in the constant 484.

4.1.4.1 Curvilinear Unit Hydrographs

Obtaining constants for curvilinear hydrographs is similar to the procedure used for the triangular hydrograph. The total runoff volume (above the base flow) from the drainage area must be included in the hydrograph. Figure 4-6 shows the generalized curvilinear hydrograph.

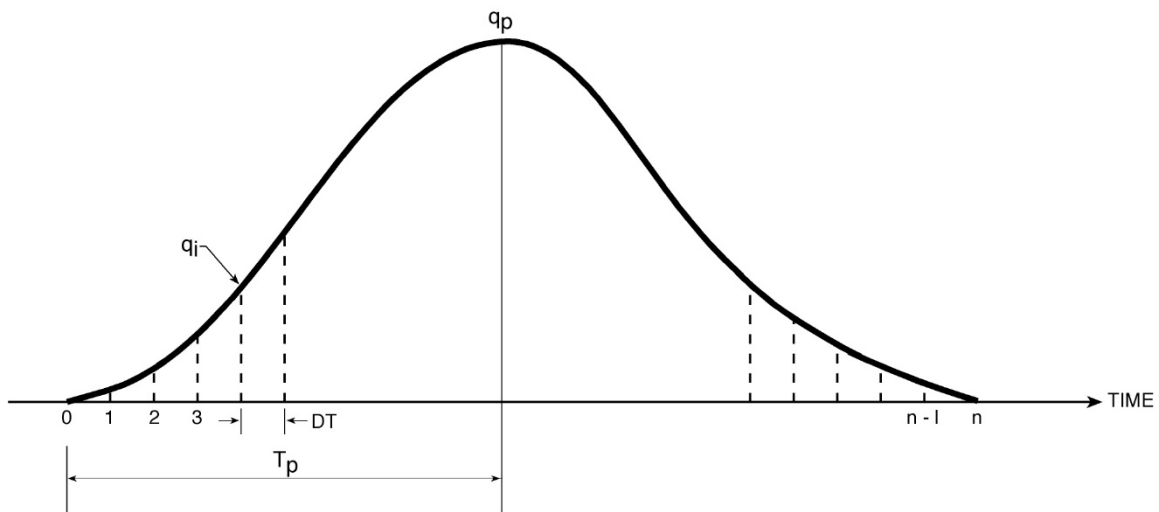


Figure 4-6. Trapezoidal Rule Illustration

- q_i = the discharge in cubic feet per second at the end of time interval, i
- DT = the length of the time interval in hours
- q_p = the peak discharge that occurs at time T_p , as previously defined

Using the Trapezoidal Rule, the area under a curve is:

$$A = DX \left(\frac{1}{2} y_0 + y_1 + y_2 + \dots + y_{n-1} + \frac{1}{2} y_n \right)$$

If $y_0 = y_n = 0$, then $A = DX \left(\sum_{i=1}^{n-1} y_i \right)$.

Where A is the area, the y_i 's are the ordinates to the curve, DX is the distance between ordinates, and n is the total number of ordinates.

To calculate the volume under the hydrograph curve using the trapezoidal rule:

$$\begin{aligned} y_i &= q_i \\ DX &= DT \\ V &= DT \left(\sum_{i=1}^{n-1} q_i \right) \end{aligned} \quad (\text{Eq. 4-11})$$

where V is the volume in (cfs x hr)

If the following dimensionless ratios are used:

$$\begin{aligned} r_i &= q_i/q_p \\ r_t &= DT/T_p \end{aligned}$$

Then the expression for V becomes

$$V = r_t T_p \sum_{i=1}^n (r_i) q_p \quad (\text{Eq. 4-12})$$

This volume of the hydrograph is equal to the runoff volume from the drainage area:

$$V = Q_a A C \quad (\text{Eq. 4-13})$$

where: A = the drainage area in square miles
Q_a = accumulated direct runoff
C = a constant to convert from square mile inches per hour to cubic feet per second
 $C = (5280^2 \text{ ft}^2 \text{ per mile}^2) / (12 \text{ inches per foot} \times 3600 \text{ second per hour})$
C = 645.33

therefore:

$$Q_a A C = r_t T_p \sum_{i=1}^n (r_i) q_p \quad (\text{Eq. 4-14})$$

Solving for q_p :

$$q_p = Q_a A C / (r_t T_p \Sigma(r_i))$$

$$q_p = (C / (r_t \Sigma r_i)) (A Q_a / T_p)$$

$$q_p = K_s (A Q_a / T_p) \quad (\text{Eq. 4-15})$$

where: $K_s = 645 / (r_t \Sigma r_i) \quad (\text{Eq. 4-16})$

K_s is a constant reflecting the units conversion and the shape of the hydrograph as previously defined in equation 4-14.

The summation of the ordinates (with $r_t = 0.2$) of the NRCS dimensionless unit hydrograph is 6.67 (Table 4-7). This gives a value of:

$$K_s = \frac{645}{(0.2/1) 6.67} = 484$$

Note that this is the same value computed for the NRCS triangular unit hydrograph (Figure 4-5).

4.1.4.2 Unit Hydrograph Rain Duration

Figure 4-5 shows the relationship between the period of excess rainfall (D) and the resulting unit hydrograph. The ratio D/T_p is usually taken as about 0.2, but it can vary. Large values of D/T_p may result in irregularly shaped hydrographs.

4.1.5 Time to Peak and Lag Time

Time to peak and Lag time are shown on Figure 4-5. Note that there are two definitions of lag time shown on Figure 4-5. Lag time as defined by the United States Army Corps of Engineers (Corps lag) differs from lag time as defined by the NRCS (NRCS lag). The relationships between time to peak, Corps lag, NRCS lag, duration of effective rainfall, and time of concentration are discussed in Sections 4.1.5.1 through 4.1.5.5, below. Depending on the method that will be used for NRCS hydrologic method calculations (i.e., whether the engineer will use the SDHydroTools computer program, HEC-HMS or hand computation); either time to peak, Corps lag, or NRCS lag will be required. Additionally depending on the data available for the study (i.e., whether a time of concentration for the study watershed has been calculated based on a rational method study, or whether watershed physical characteristics will be used to calculate Corps lag), it may be necessary to convert one parameter to another. The following discussions will define each parameter and relationships between the parameters. Section 4.3 will identify when each parameter should be used and the relationships to be used to convert one parameter to another.

4.1.5.1 Time to Peak

Time to peak is defined as the elapsed time from the beginning of unit effective rainfall to the peak flow for the point of concentration. Time to peak is used when NRCS hydrologic method calculations are performed by the hand computation method described in Section 4.3. Time to peak must be determined in order to calculate the Unit Hydrograph ordinates, which are at intervals of t/T_p . T_p may be calculated for a watershed based on Corps lag or time of concentration (see Sections 4.1.5.2 and 4.1.5.5, respectively).

4.1.5.2 Corps Lag

The lag relationship given in this section (herein referred to as Corps lag or Corps T_l) is based upon criteria developed by the United States Army Corps of Engineers (United States Army Engineer District Los Angeles Corps of Engineers, 1976). Corps lag for a drainage area can be defined as the elapsed time (in hours) from the beginning of unit effective

rainfall to the instant that the summation hydrograph for the point of concentration reaches 50% of ultimate discharge. Corps lag is an empirical expression of the physical characteristics of a drainage area in terms of time. Corps lag can be expressed by the empirical formula:

$$\text{Corps } T_1 \text{ (hours)} = 24 \bar{n} ((L \times L_c) / s^{0.5})^m \quad (\text{Eq. 4-17})$$

- where:
- L = length to longest watercourse (miles)
 - L_c = length along longest watercourse, measured upstream to a point opposite the watershed centroid (miles)
 - s = overall slope of drainage area between the headwaters and the collection point (feet per mile)
 - m = a constant determined by regional flood reconstitution studies (0.38 for San Diego County)
 - \bar{n} = the average of the Manning's n values of the watercourse and its tributaries (see Section 4.3.5)

Figure 4-4 shows that 50% of ultimate discharge ($Q_t/Q_a = 0.5$) occurs at $t/T_p = 1.16$. Since Corps lag is defined as the time at which 50% of ultimate discharge occurs, Corps lag is related to T_p by the following equations:

$$\text{Corps } T_1 = 1.16 T_p \quad (\text{Eq. 4-18})$$

Or:

$$T_p = 0.862 \text{ Corps } T_1 \quad (\text{Eq. 4-19})$$

for the typical NRCS dimensionless unit hydrograph.

4.1.5.3 NRCS Lag

Lag as defined by the NRCS for use with the NRCS dimensionless unit hydrograph (herein referred to as NRCS lag or NRCS T_l) differs from Corps lag. The NRCS lag of a watershed is defined as the time from the center of mass of excess rainfall to the time to peak of the unit hydrograph. NRCS lag is dependent on the period of effective rainfall (D) selected for the analysis. A small amount of variation is allowable in D , however D should be approximately $0.2T_p$. The center of mass of effective rainfall is found as $(D/2)$.

NRCS lag is found by first determining T_p using one of the equations given above in Section 4.1.5.2 or below in Section 4.1.5.5, and then selecting an appropriate D for the study based on T_p . NRCS lag is then determined by the following equation:

$$\text{NRCS } T_l = T_p - D/2 \quad (\text{Eq. 4-20})$$

4.1.5.4 Relationship between Corps Lag and NRCS Lag

By combining equations 4-19 and 4-20 above, NRCS lag can also be calculated from Corps lag using the following relationship:

$$\text{NRCS } T_l = 0.862 \text{ Corps } T_l - D/2 \quad (\text{Eq. 4-21})$$

4.1.5.5 Relationships between T_p , T_c , and Corps Lag

When the lags determined from summation hydrographs for several gauged drainage areas are correlated to the hydrologic characteristics of the drainage areas, an empirical relationship is usually apparent. This relationship can then be used to determine the lags for comparable drainage areas for which the hydrologic characteristics can be determined, but for which the distribution graphs for concentration points cannot be determined because of inadequate hydrologic data. By comparing lag values (obtained from the analysis of rainfall-runoff data) to catchment T_c values estimated from a detailed RM analysis, a relationship is readily determined.

In McCuen (1982), NRCS T_p is related to T_c by:

$$T_p = 0.67 T_c \quad (\text{Eq. 4-22})$$

Where T_c is defined in chapter 15 of NEH-4 as: 1) the time for runoff to travel from the furthestmost point in the watershed to one point in question, and 2) the time from the end of excess rainfall to the point of inflection of the unit hydrograph.

From equation 4-18, Corp lag can be related to T_p :

$$\text{Corps } T_1 = 1.16 T_p$$

When equations 4-18 and 4-22 are combined, the result is:

$$\text{Corps } T_1 = 1.16 (0.67) T_c = 0.77 T_c \quad (\text{Eq. 4-23})$$

The following relationship relating Corps lag to T_c , based on equation 4-23, was adopted for this manual:

$$\text{Corps } T_1 = 0.8 T_c \quad (\text{Eq. 4-24})$$

4.2 DEVELOPING INPUT DATA FOR NRCS HYDROLOGIC METHOD COMPUTATIONS

The following data is required for NRCS hydrologic method calculations: watershed area and physical characteristics, rainfall amounts for durations up to 24 hours from NOAA Atlas 14, precipitation zone number, and runoff curve number. Sections 4.2.1 through 4.2.4 describe the development of input data. Section 4-3 describes the procedure for NRCS hydrologic method calculations.

4.2.1 Watershed, Geographic Location, Area, and Physical Characteristics

The watershed area and geographic location are determined from the best available data from topographic maps, field surveys, reliable GIS programs, digital elevation models, or other industry accepted methods, subject to agency approval. For large drainage areas it might be necessary to divide the area into subareas to account for major land use changes, obtain analysis results at different points within the watershed area, combine hydrographs from different subareas as applicable, and/or route flows to points of interest. The highest elevation in the watershed (high point) and the elevation of the point of interest (low point) are determined from the map. The watershed length along the longest watercourse and the length to centroid are also read from the map. The centroid is the point in the watershed where approximately 50% of the watershed area is contributing to the watercourse. The length to centroid is measured from the low point of the watershed to the centroid.

4.2.1.1 Basin Factor (\bar{n})

The basin \bar{n} factor is the visually estimated mean of the n values (roughness values from Manning's formula) of all the channels within the basin area. A basin \bar{n} factor can be estimated by comparing characteristics of drainage areas being studied with the characteristics of the drainage areas for which basin \bar{n} factors have been estimated. Typical values of \bar{n} range from 0.015 for areas that are mainly developed and have a large percentage of impervious areas, to 0.100 for areas with extensive vegetation including vegetation in watercourses that slows water velocity.

The following descriptions are a guide for estimating the basin \bar{n} factor, based on Plate 21, *Lag Relationships* from *Antelope Valley Streams Los Angeles County, California, Draft Survey Report, Hydrology Part I*, prepared by the U.S. Army Engineer District, Los Angeles Corps of Engineers (USACE, 1976):

$\bar{n} = 0.100$: The drainage area has extensive vegetation, including grass, or is farmed with contoured plowing, and streams that contain a large amount of brush, grass or other vegetation that slows water velocity.

$\bar{n} = 0.050$: Drainage area is quite rugged, with sharp ridges and narrow, steep canyons through which watercourses meander around sharp bends, over large boulders and considerable debris obstruction. The ground cover, excluding small areas of rock outcrops, includes many trees and considerable underbrush. No drainage improvements exist in the area.

$\bar{n} = 0.030$: Drainage area is generally rolling, with rounded ridges and moderate side slopes. Watercourses meander in fairly straight, unimproved channels with some boulders and lodged debris. Ground cover includes scattered brush and grasses. No drainage improvements exist in the area.

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

The following additional references may also be helpful for determining an appropriate basin \bar{n} factor: *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains* (USGS Water Supply Paper 2339) presents procedures for assigning reliable n values for channels and floodplains. This paper contains some photos, examples, and step-by-step procedures. The procedures can be used in the field. *Open Channel Hydraulics* by Ven Te Chow also provides guidance with tables of n values and photographs.

4.2.2 Rainfall and Precipitation Zone Number

Rainfall amounts for durations up to 24 hours are taken from NOAA Atlas 14. Procedures to obtain data from NOAA Atlas 14 are provided in Appendix B of this manual. For relatively small watersheds the point rainfall may be taken at the watershed centroid. For larger watersheds in which rainfall varies throughout the watershed, the engineer may

estimate a weighted average rainfall amount for the entire watershed based on points taken within the watershed.

The PZN is taken from the map in Appendix C. As with the rainfall amount, the engineer may estimate a weighted average PZN for large watersheds.

4.2.3 Runoff Curve Number

The step-by-step procedure for calculating a composite CN is described in Table 4-8. A 2000-scale topographic map, and an NRCS hydrologic ground cover map and NRCS soil group map at the same scale are needed. GIS ground cover layer can also be obtained from [SanGIS](#). A sheet of translucent vellum is also needed. All of the above steps may be completed using more modern tools, such as reliable GIS, CADD, or spreadsheet programs, or other industry accepted methods subject to agency approval

Step 5 of the procedure described in Table 4-8 involves tabulating data for different combinations of land use and soil group within the watershed. Data may be recorded on Worksheet 4-1 (provided in Appendix D). Composite CNs for the drainage area can be calculated by entering the data collected into a table such as the one provided as Worksheet 4-2 (provided in Appendix D) or a spreadsheet set up in a similar format (Worksheet headers presented as Table 4-9).

The composite CN for the total drainage area is then the sum of the composite CNs from column 6 of Worksheet 4-2 or Table 4-9.

Table 4-8

PROCEDURE FOR CALCULATION OF CURVE NUMBERS

Reference in Hydrology Manual	Procedure Step No.:	Refer to Example on Page:
Workbook Figure WB.3-1	1. Locate drainage area on 1":2000' scale USGS topographic map(s).	p. WB-31
Workbook Figure WB.3-1	2. Using a ½-inch or 1-inch grid (½ inch for areas less than 5 square miles) on a translucent overlay sheet, trace the drainage area boundary and other significant information from the topographic maps.	p. WB-31
Workbook Figures WB.3-2 and WB.3-3	3. Locate the drainage area on 1":2000' scale. GIS ground cover and soil group maps are available at SanGIS .	p. WB-32 p. WB-33
Workbook Figure WB.3-4	4. Overlay the grid sheet onto the ground cover and soil group maps; for each map, record the appropriate group cover (OB, NC, DL) and soil group (A, B, C, or D) at each grid intersection within the drainage area.	p. WB-34
Workbook Figure WB.3-4 Appendix D, Worksheet 4-1	5. For each combination of ground cover/soil group, count and record the number of grid intersections where that combination occurs.	p. WB-31, Appendix WB.A, Worksheet 4-1
Appendix D, Worksheet 4-1	6. Compute the total number of grid intersections within the drainage area. For a 1-inch grid, each intersection represents 1 square inch on the maps, and the total area of the drainage area is found by scale conversion. For the ½-inch grid, each intersection is ¼ square inch. Compute the total area of the drainage area.	p. WB-31, Appendix WB.A, Worksheet 4-1
Tables 4-3, 4-4, 4-5	7. By field inspection, determine the hydrologic conditions that exist in the drainage area for each type of ground cover.	p. 4-15
Appendix D, Worksheet 4-2, Column 5	8. For each ground cover/soil group combination, compute the fraction of the total area represented by that combination by the ratio of the number of grid intersections counted in step 5 to the total grid intersections (step 6).	p. WB-35, Appendix WB.A, Worksheet 4-2

Table 4-8 (Continued)

PROCEDURE FOR CALCULATION OF CURVE NUMBERS

Reference in Hydrology Manual	Procedure Step No.:	Refer to Example on Page:
Table 4-2 Appendix D, Worksheet 4-2, Column 4	9. For each ground cover/soil group/hydrologic condition combination, select the appropriate runoff CN for PZN Condition = 2.0, the CN ₂ .	p. 4-10, Appendix WB.A, Worksheet 4-2
Appendix D, Worksheet 4-2, Column 6	10. Compute the partial CN ₂ for each combination by the product of area fraction of each combination from step 8 and the selected CNs from step 9.	Appendix WB.A, Worksheet 4-2
Appendix D, Worksheet 4-2, Column 6	11. Sum the partial CN ₂ 's to obtain the CN ₂ for the entire drainage area.	Appendix WB.A, Worksheet 4-2
	12. If applicable to the study, determine the CN for future land uses, modify existing ground cover designations and use the same procedures.	

Table 4-9

**Worksheet Headers for
 Composite Curve Number Calculations**

column 1	column 2	column 3	column 4	column 5	column 6
GROUND COVER/ LAND USE	HYDROLOGIC CONDITION (field inspection)	SOIL GROUP	CN ₂ (Table 4-2)	FRACTION OF AREA A _i /A	PARTIAL CN ₂ CN ₂ x A _i /A

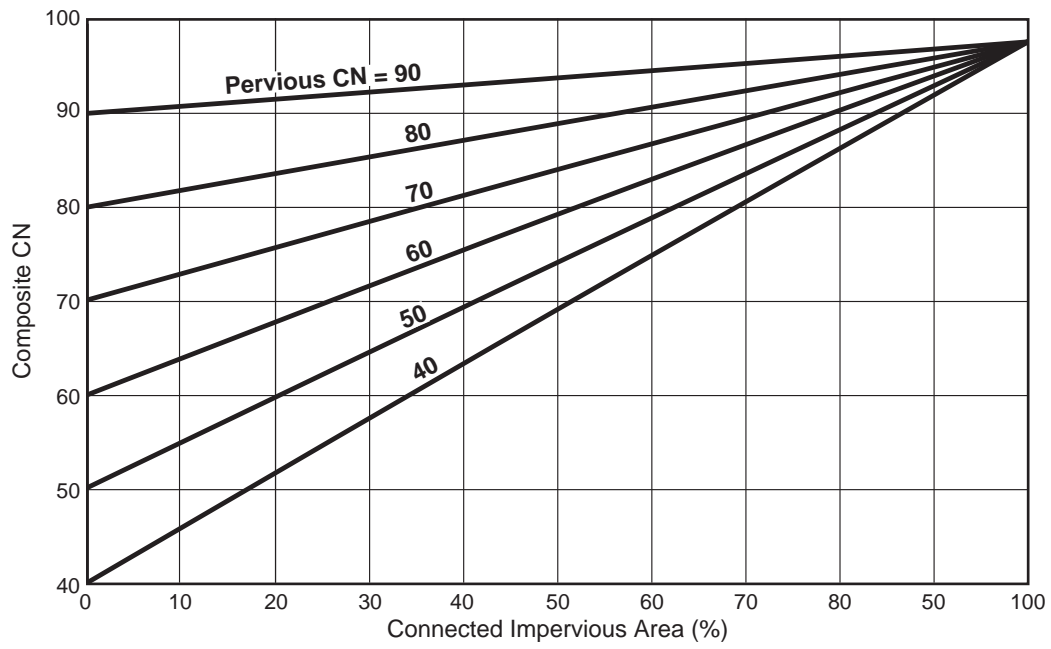
The following discussion gives some guidance based on TR-55 for adjusting CNs for different types of impervious areas. These adjustments may be used where relevant for urbanized areas that have unconnected impervious areas or where the values of percentage of impervious area are not applicable. Section 4.2.4 describes adjustment of the CN for PZN Condition (required for all NRCS hydrologic method studies).

4.2.3.1 Connected Impervious Areas

The CN values given in Table 4-2 for urban land uses are based on directly connected impervious areas and specific assumed percentages of impervious area. An impervious area is considered directly connected if runoff from it flows directly into the drainage system. It is also considered directly connected if runoff from it occurs as concentrated shallow flow that runs over pervious areas (such as flow in a swale) and then into a drainage system. The CN values given in Table 4-2 were developed on the assumptions that:

- (a) pervious urban areas are equivalent to pasture in good hydrologic condition, and
- (b) impervious areas have a CN of 98 and are directly connected to the drainage system.

If all the impervious area is directly connected to the drainage system, but the impervious area percentages or the pervious land use assumptions in Table 4-2 are not applicable, use Figure 4-7 to compute a composite CN. For example, Table 4-2 gives a CN of 70 for a 0.5-acre lot in hydrologic soil group B, with an assumed impervious area of 25%. However, if the lot has 20% impervious area and a pervious area CN of 61, the composite CN obtained from Figure 4-7 is 68. The CN difference between 70 and 68 reflects the difference in percent impervious area.



SOURCE: TR-55, Second Ed. June 1986

Composite CN with Connected Impervious Areas

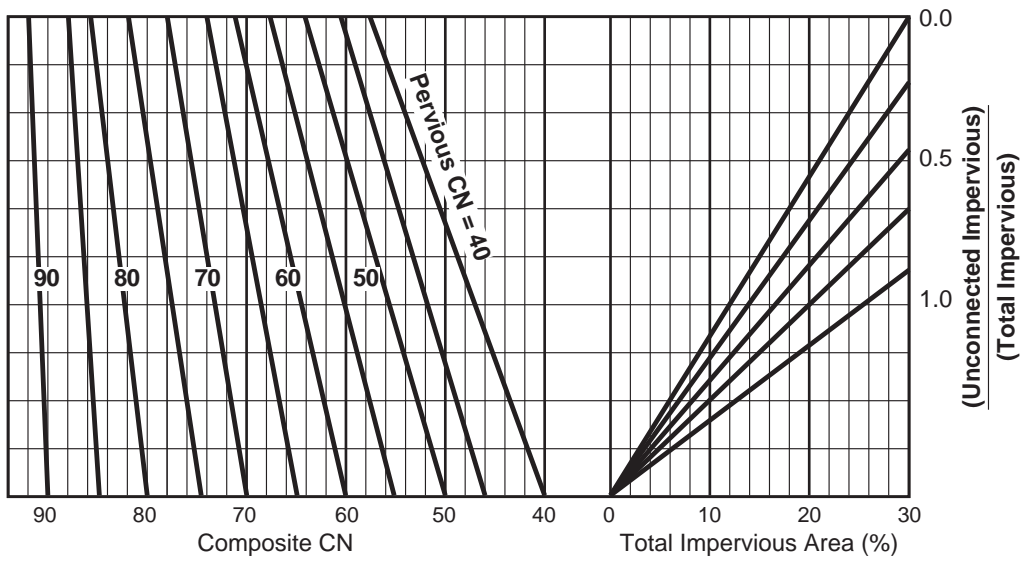
FIGURE

4-7

4.2.3.2 Unconnected Impervious Areas

Runoff from these areas is spread over a pervious area as sheet flow. To determine CN when all or part of the impervious area is not directly connected to the drainage system, (1) use Figure 4-8 if total impervious area is less than 30% or (2) use Figure 4-7 if the total impervious area is equal to or greater than 30%, because the absorptive capacity of the remaining pervious areas will not significantly affect runoff.

When the impervious area is less than 30%, obtain the composite CN by entering the right half of Figure 4-8 with the percentage of total impervious area and the ratio of total unconnected impervious area to total impervious area. Then move left to the appropriate pervious CN and read down to find the composite CN. For example, for a 0.5-acre lot with 20% total impervious area (75% of which is unconnected) and pervious CN of 61, the composite CN from Figure 4-8 is 66. If all the impervious area is connected, the resulting CN (from Figure 4-7) would be 68.



SOURCE: TR-55, Second Ed. June 1986

**Composite CN with Unconnected Impervious Areas
(Total Impervious Area Less Than 30%)**

F I G U R E

4-8

4.2.4 PZN Condition

The CNs provided in Table 4-2 are for PZN Condition 2.0 (PZN adjustment factor = 2.0). After the CN has been calculated for the study area, it must be adjusted for PZN Condition. This adjustment is required for NRCS hydrologic method studies. The PZN adjustment factors (described in Section 4.1.3 and provided in Table 4-6) are based on the storm frequency and the precipitation zone that the watershed is located in. To adjust the CN for PZN Condition, first determine the appropriate PZN adjustment factor for the combination of storm duration and precipitation zone for the study. For precipitation zone numbers not equal to 1.0, 2.0, 3.0, and 4.0 (Coast, Foothills, Mountains, and Desert), interpolate the PZN adjustment factor between the zones. Interpolation, if necessary, is linear. For example, for a 100-year storm duration for a study area with a PZN of 1.5, the PZN adjustment factor interpolated from the values in Table 4-6 is 2.5. After determining the appropriate PZN adjustment factor, use Table 4-10 to determine the adjusted CN for the study area for the appropriate PZN Condition. If the appropriate PZN Condition for the study area based on the storm duration and PZN is 2.0 (PZN adjustment factor = 2.0), no adjustment is necessary because the CNs provided in Table 4-2 are for PZN Condition 2.0. For PZN adjustment factor equal to 1.0 or 3.0, locate the CN value for PZN Condition 2.0 and read the adjusted CN value for PZN Condition 1.0 or 3.0 from the same row of the table. For PZN adjustment factor not equal to 1.0, 2.0, or 3.0, interpolate the CN between the value for PZN Condition 2.0 and the value for the appropriate PZN Condition in the same row of the table. Interpolation, if necessary, is linear.

Table 4-10

RUNOFF CURVE NUMBERS FOR PZN CONDITIONS 1.0, 2.0, AND 3.0

CN For:			CN For:		
PZN Condition = 1.0	PZN Condition = 2.0	PZN Condition = 3.0	PZN Condition = 1.0	PZN Condition = 2.0	PZN Condition = 3.0
100	100	100	40	60	78
97	99	100	39	59	77
94	98	99	38	58	76
91	97	99	37	57	75
89	96	99	37	56	75
87	95	98	34	55	73
85	94	98	34	54	73
83	93	98	33	53	72
81	92	97	32	52	71
80	91	97	31	51	70
78	90	96	31	50	70
76	89	96	30	49	69
75	88	95	29	48	68
73	87	95	28	47	67
72	86	94	27	46	66
70	85	94	26	45	65
68	84	93	25	44	64
67	83	93	25	43	63
66	82	92	24	42	62
64	81	92	23	41	61
63	80	91	22	40	60
62	79	91	21	39	59
60	78	90	21	38	58
59	77	89	20	37	57
58	76	89	19	36	56
57	75	88	18	35	55
55	74	88	18	34	54
54	73	87	17	33	53
53	72	86	16	32	52
52	71	86	16	31	51
51	70	85	15	30	50
50	69	84			
48	68	84	12	25	43
47	67	83	9	20	37
46	66	82	6	15	30
45	65	82	4	10	22
44	64	81	2	5	13
43	63	80	0	0	0
42	62	79			
41	61	78			

4.3 PROCEDURE FOR NRCS HYDROLOGIC METHOD COMPUTATIONS

NRCS hydrologic method computations are based on the concepts and equations presented in Section 4.1. Several computer programs, including the public domain computer programs and HEC-HMS model these equations. Additionally, the County of San Diego provides the SDHydroTools computer program for use with this manual. The purpose of the San Diego Unit Hydrograph (SDUH) module within the SDHydroTools computer program is the same as the peak flow charts that were provided in the 1993 edition of this manual and the SDUH Peak Discharge Program that was provided with the 2003 edition of this manual, to provide engineers with the option of determining the peak flow without the need for hand calculations, thus reducing the time and effort needed in watershed studies and improving consistency between studies. As with the peak flow charts and the SDUH Peak Discharge Program, the SDUH module in the SDHydroTools computer program is for single-basin analysis. The SDHydroTools computer program will provide a hydrograph and a peak flow rate for a single basin but does not combine hydrographs or perform hydrograph routing calculations. For those functions, HEC-HMS or hand calculations may be used. Other computer programs may be used provided a detailed output and description is provided, and it is allowed by the reviewing agency.

The following steps describe the procedure for performing NRCS hydrologic method calculations:

1. Determine lag time and/or time to peak, and computation interval
2. Prepare incremental rainfall distribution
3. Calculate excess rainfall
4. Develop hydrograph of direct runoff from the drainage area

These steps are discussed in Sections 4.3.1 through 4.3.4 below.

4.3.1 Step 1: Determine Lag Time and/or Time to peak, and Computation Interval

To perform NRCS hydrologic method calculations, the lag time and/or T_p for the watershed must be calculated first. Concepts and equations for lag time and T_p are presented in Section 4.1.5. If a hand calculation will be used to determine the peak flow for the study, T_p is needed. If the SDHydroTools computer program will be used to determine the peak flow for the study, Corps lag is needed. HEC-HMS requires NRCS lag.

Lag time (Corps lag or NRCS lag) and T_p may be calculated based on the T_c to the point of interest or based on watershed characteristics, depending on the type of study and data available. Section 4.3.1.1 describes the method for calculating T_p and/or NRCS lag based on Corps lag, which can be calculated based on the watershed physical characteristics. Section 4.3.1.2 describes the method for calculating T_p based on T_c for studies that are transitioned from the RM or MRM to the NRCS hydrologic method. Corps lag or NRCS lag for a basin may also be calculated based on T_c if needed.

4.3.1.1 Calculation of Time to Peak Using Corps Lag

To calculate T_p using Corps lag, with Corps lag calculated based on watershed physical characteristics, use the empirical formula given in equation 4-17 for Corps lag, where watershed length (L) and length to centroid (L_c) are in miles and watershed slope (s) is in feet per mile (see Section 4.1.5.2).

$$\text{Corps } T_1 \text{ (hours)} = 24 \bar{n} ((L \times L_c) / s^{0.5})^m$$

Once Corps lag has been determined, T_p is calculated using equation 4-19:

$$T_p = 0.862 \times \text{Corps } T_1$$

If NRCS lag is needed, it is calculated using equation 4-20:

$$\text{NRCS } T_1 = T_p - D/2$$

4.3.1.2 Calculation of Time to Peak Using Time of Concentration

When a RM study is transitioned to an NRCS hydrologic method study, it may be more convenient to continue calculating T_c for each reach of the study and relate T_p to T_c . To extend T_c along a reach of a study, estimate the velocity for the reach using Manning's formula (Figure 3-7) and estimated channel geometry for the reach. Calculate T_t in the reach (length divided by velocity), and add T_t to the previous T_c to obtain the T_c for the point at the end of the reach.

Once T_c has been determined, T_p is calculated using equation 4-22:

$$T_p = 0.67 T_c$$

It is noted that the RM T_c , used for the estimation of T_p , is a critical parameter in the unit hydrograph method. Extreme care must be taken in the evaluation of the watershed T_c in order to reduce uncertainty and enable "reproducibility" of this parameter.

If needed, NRCS lag can then be calculated using equation 4-20:

$$\text{NRCS } T_1 = T_p - D/2$$

If needed, the Corps lag may also be calculated based on T_c . As discussed above in Section 4.1.5, the relationship of Corps $T_1 = 0.8 T_c$ (equation 4-24) was adopted for this manual.

4.3.2 Step 2: Prepare Incremental Rainfall Distribution

Creation of the 24-hour nested storm rainfall distribution requires rainfall depths for increments of storm duration from the selected computation interval through 24 hours (e.g., to create the nested storm using a 15-minute computation interval, rainfall depths are required for durations equal to 15 minutes, 30 minutes, 45 minutes, 1 hour, 1.25 hours, and so on through 24 hours). The computation interval is the period of excess rainfall (D) and should be approximately $\leq 0.2T_p$.

Rainfall depths can be determined from precipitation depth data pairs provided in NOAA Atlas 14. Discussion of NOAA Atlas 14 and procedures to obtain data from NOAA Atlas 14 are provided in Appendix B of this manual.

NOAA Atlas 14 provides precipitation depth data and precipitation intensity data for selected durations (e.g., 5 minutes, 10 minutes, etc.) ranging from 5 minutes to 60 days. For a duration that is between given data points from NOAA Atlas 14 (e.g., 45 minutes), use log-log interpolation to interpolate the depth for the duration, using the following equation:

$$P = \text{Log}^{-1}(\text{Log}P_1 + (\text{Log}(T / T_1)) (\text{Log}(P_2 / P_1)/(\text{Log}(T_2 / T_1))))$$

Where:

P = Rainfall depth at the desired duration (inches)

T = Desired duration (minutes)

T₁ and T₂ are the bounding times (minutes) of given data points from NOAA Atlas 14
and T₁ < T < T₂

P₁ = Rainfall depth at T₁ (inches per hour) from NOAA Atlas 14

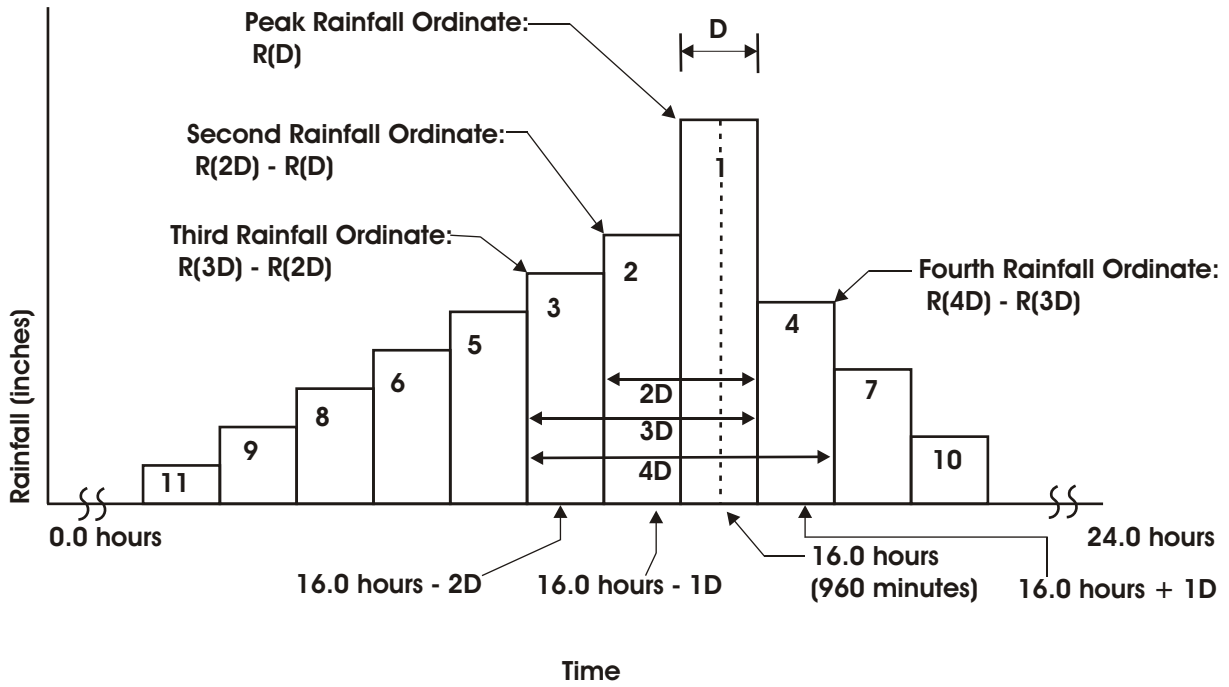
P₂ = Rainfall depth at T₂ (inches per hour) from NOAA Atlas 14

Record the total rainfall depth for each duration in order of increasing duration.

Next, adjust the total rainfall depth for each duration using the appropriate depth-area adjustment values based on the watershed area (multiply the rainfall amount by the depth-area adjustment factor). For durations less than 30 minutes, use the 30-minute depth area adjustment value. For durations greater than 30 minutes and not equal to durations with data available on Figure 4-2 and Table 4-1, interpolate the depth area adjustment between the surrounding data points on Table 4-1. Interpolation, if necessary, is linear.

Next, create the ordinates of the hyetograph using the depth-area adjusted total rainfall amounts. Figure 4-9 shows the construction of the hyetograph. The first ordinate “ $R(D)$ ” is the depth-area adjusted total rainfall amount for the first time increment. The second ordinate “ $R(2D) - R(D)$ ” is the depth-area adjusted total rainfall amount for the second time increment minus the depth-area adjusted total rainfall amount for the first time increment. The third ordinate “ $R(3D) - R(2D)$ ” is the depth-area adjusted total rainfall amount for the third time increment minus depth-area adjusted total rainfall amount for the second time increment, and so on. Note: the sum of the ordinates of the hyetograph should be equal to the depth-area adjusted total rainfall amount for duration = 24 hours.

Finally, sort the ordinates of the hyetograph into the order of the (2/3, 1/3) distribution. The first ordinate (calculated above, the depth-area adjusted incremental rainfall amount for the first time increment) is the peak rainfall ordinate. This peak rainfall ordinate occurs at hour 16.0 of the 24-hour storm. The second rainfall ordinate (calculated above) occurs at 16.0 hours – 1D, the third rainfall ordinate (calculated above) occurs at 16.0 hours – 2D, and the fourth rainfall ordinate (calculated above) occurs at 16.0 hours + 1D. The sequence continues alternating two ordinates to the left and one ordinate to the right (see Figure 4-9).



Construction of Hyetograph

FIGURE

4-9

4.3.3 Step 3: Calculate Excess Rainfall

Excess rainfall is calculated using equation 4-4. Excess rainfall must be calculated for a cumulative rainfall series. Because equation 4-4 is subject to the limitation, $P \geq 0.2S$, calculation of excess rainfall based on the ordinates of the hyetograph (which are incremental rainfall amounts) will result in underestimation of excess rainfall because the incremental rainfall amounts are small. To calculate excess rainfall, create a cumulative rainfall series by summing the ordinates of the hyetograph. This must be performed after the ordinates have been sorted into the (2/3, 1/3) distribution (see above). Calculate each excess rainfall ordinate from the cumulative rainfall series using equation 4-4.

$$Q_a = \frac{(P - 0.2S)^2}{(P + 0.8S)}$$

Note: the last ordinate of the series should be equal to the excess runoff from the depth-area adjusted incremental rainfall amount for duration = 24 hours.

Finally, create incremental amounts of excess rainfall from the cumulative series.

Table 4-11 gives values of S and $P \geq 0.2S$ for the curve numbers.

Table 4-11

RUNOFF CURVE NUMBERS AND CONSTANTS FOR THE CASE $I_A=0.2S$

CN	S (inches)	Curve Starts Where P = (inches)	CN	S (inches)	Curve Starts Where P = (inches)
100	0	0	60	6.67	1.33
99	.101	.02	59	9.95	1.39
98	.204	.04	58	7.24	1.45
97	.309	.06	57	7.54	1.51
96	.417	.08	56	7.86	1.57
95	.526	.11	55	8.18	1.64
94	.638	.13	54	8.52	1.70
93	.753	.15	53	8.87	1.77
92	.870	.17	52	9.23	1.85
91	.989	.20	51	9.61	1.92
90	1.11	.22	50	10.0	2.00
89	1.24	.25	49	10.4	2.08
88	1.36	.27	48	10.8	2.16
87	1.49	.30	47	11.3	2.26
86	1.63	.33	46	11.7	2.34
85	1.76	.35	45	12.2	2.44
84	1.90	.38	44	12.7	2.54
83	2.05	.41	43	13.2	2.64
82	2.20	.44	42	13.8	2.76
81	2.34	.47	41	14.4	2.88
80	2.50	.50	40	15.0	3.00
79	2.66	.53	39	15.6	3.12
78	2.82	.56	38	16.3	3.26
77	2.99	.60	37	17.0	3.40
76	3.16	.63	36	17.8	3.56
75	3.33	.67	35	18.6	3.72
74	3.51	.70	34	19.4	3.88
73	3.70	.74	33	20.3	4.06
72	3.89	.78	32	21.2	4.24
71	4.08	.82	31	22.2	4.44
70	4.28	.86	30	23.3	4.66
69	4.49	.90			
68	4.70	.94	25	30.0	6.00
67	4.92	.98	20	40.0	8.00
66	5.15	1.03	15	56.7	11.34
65	5.38	1.08	10	90.0	18.00
64	5.62	1.12	5	190.0	38.0
63	5.87	1.17	0	infinity	infinity
62	6.13	1.23			
61	6.39	1.28			

4.3.4 Step 4: Develop Hydrograph of Direct Runoff from the Drainage Area

The hydrograph of direct runoff from the drainage area is developed as follows:

First, the unit hydrograph ordinates are created based on the T_p and the unit hydrograph q_p for the study area. T_p was calculated in step 1 (see Section 4.3.1). The unit hydrograph q_p is then calculated using equation 4-10:

$$q_p = \frac{K_s A Q_a}{T_p}$$

where:

$K_s = 484$, a constant reflecting both the conversion of units and the shape of the hydrograph

$Q_a = 1$ inch of effective runoff

$A =$ watershed area (square miles)

Next, set up the unit hydrograph ordinates t/T_p and q/q_p . The time increment (t) for unit hydrograph ordinates must be the same duration as the period of effective rainfall selected for the rainfall ordinates (D). For multiples of t , compute t/T_p until $t/T_p = 5$. For each t/T_p , find the corresponding q/q_p from Table 4-7. For values of t/T_p that are not given on Table 4-7, read the corresponding values of q/q_p from Figure 4-4 or interpolate from the nearest values from Table 4-7. Next, compute the unit hydrograph q for each ordinate.

Set up a table with the unit hydrograph q ordinates in rows and incremental excess rainfall ordinates in columns. Table 4-12 provides an abbreviated sample table with letters for column identification and numbers for row identification that correspond with the discussion below describing convolution of the unit hydrograph. A complete example is provided in the Workbook of Sample Problems provided at the end of this manual. Convolution of the unit hydrograph is performed as follows:

1. Multiply the effective rainfall depth for the first unit time period (column C, row 2) by each unit hydrograph ordinate q (column B, rows 4 through 9) to determine the flood hydrograph which would result from that increment of effective rainfall.
2. Repeat the above process for each succeeding effective rainfall depth (columns D through H) advancing the resultant flood hydrographs one unit time period for each cycle.
3. Sum the flow ordinates found in the steps above across the rows to determine the average flow ordinates per unit time period for the design storm flood hydrograph (column I).

Table 4-12
SAMPLE TABLE FOR CONVOLUTION OF UNIT HYDROGRAPH

Row/Column Identification	A	B	C	D	E	F	G	H	I
1		Time (minutes)	15	30	45	60	75	90	
2		Effective Rainfall Ordinate (inches)	0.00	0.01	0.02	0.04	0.05	0.07	
3	Time, t (minutes)	Unit Hydrograph Ordinate, q							Flood Hydrograph Ordinate (cfs)
4	15	119	0.00						0.00
5	30	339	0.00	1.19					1.19
6	45	705	0.00	3.39	2.38				5.77
7	60	1158	0.00	7.05	6.78	4.76			18.59
8	75	1422	0.00	11.58	14.10	13.56	5.95		45.19
9	90	1500	0.00	14.22	23.16	28.20	16.95	8.33	90.86

Note: This Table represents an abbreviated example. A complete example is included in the Workbook Section of this manual.

4.4 TRANSITION FROM RATIONAL METHOD TO NRCS HYDROLOGIC METHOD

As discussed in Section 3, the engineer should only use the RM or MRM for drainage areas up to approximately 1 square mile. The NRCS hydrologic method should be used for study areas approximately 1 square mile and greater in size. For study areas greater than approximately 1 square mile, the NRCS hydrologic method may be used for the entire study area, or the RM or MRM may be used for approximately 1 square mile of the study area and then transitioned to the NRCS hydrologic method using the procedure described below:

- Stop RM calculations at approximately 1 square mile.
- Freeze RM peak discharge, Q_p , at approximately 1 square mile.
- Begin NRCS hydrograph calculations at the next point of interest. Estimate the travel time, T_t , from the MRM calculations along the reach to the point of interest and increase the T_c from the MRM calculations by T_t . Determine T_p based on T_c using equation 4-22. Perform NRCS calculations using T_p and the total watershed area to the point of interest (Note: if the SDUH Peak Discharge Program will be used for the NRCS calculations, convert T_p to Corps lag using equation 4-18).

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

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

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SECTION 5 EROSION AND SEDIMENTATION

5.1 LIMITATION AND APPLICATION

This section should be useful to the engineer analyzing the erosion potential of a site and sizing sediment basins, or in applying other erosion protection devices to a project. Municipal agency staff members may also use this section when reviewing project plans and studies. The focus of this section is to provide an outline for the prediction of sediment yield that occurs during rainfall events in a study area. The County may require implementation of Best Management Practices (BMPs) to limit the production and transport of sediment.

5.2 INTRODUCTION AND DEFINITIONS

Erosion and sedimentation are generally natural processes but may be accelerated by human activities. During construction of development projects, scarification and grading of topsoil is performed by machinery. These actions remove natural vegetation that would normally absorb some stormwater and assist in holding soil together with root structures. The entire area is subject to severe erosion and sedimentation after a rainstorm once vegetation is removed or soil is disturbed. Generally, consideration should be taken by projects to mimic pre-development conditions to minimize erosion and sediment deposition.

The ability to predict sediment yield and determine methods required to control sediment has improved over the last 50 years. Many erosion control devices stem from historical ideas of farmers whose soils and crops were being degraded by erosion. The importance of preserving soil on development projects has become just as important as on croplands. Examples of erosion control devices can be found in Section 5.5.

5.2.1 Erosion

Soil erosion is defined by the CASQA Construction BMP Handbook as the process by which soil particles are removed from the land surface by wind, water, or gravity. Erosion begins with the first splash of a raindrop onto the soil. The impact of the raindrop may displace a soil particle on the ground, depending on the cohesion or stability of the material. Erosion intensifies during significant storm events lasting for long periods of time and/or that experience intense rainfall. Soil may become saturated after long or intense rainfall, allowing rainfall-runoff to sheet-flow over it. Raindrops then follow the natural terrain and are combined with other raindrops carrying suspended particles as well. The runoff becomes a trickle, gully, creek, and eventually a river. This process can lead to the transport of large amounts of soil particles downstream. As seen in Figure 5-1, erosion can, has, and will likely change the terrain, undermine structures, alter the capacity of storm drain infrastructure, cause damage to property, and may cause loss of life.

5.2.2 Sedimentation

Sedimentation is defined by the California Stormwater Quality Association (CASQA) Construction BMP Handbook as the settling out of particles transported by water. After a soil particle is displaced by the impact of a raindrop, the particle may remain suspended in the raindrop for a period of time and be transported downstream until it is deposited in a new location (sedimentation). In general, the soil particle will remain in suspension until the raindrop (1) slows down to a sufficient velocity to allow the particle to fall out of suspension, (2) is absorbed into the ground, leaving the soil particle behind on the ground, or (3) evaporates, leaving the soil particle behind on the ground.



Example of Urban Erosion

FIGURE

5-1

5.2.3 Supplemental References

The following references are suggested reading material. The references listed are not meant to be all-inclusive but are a sample of available material to supplement the engineer's experience when making decisions about sediment yield.

- (a) *Erosion and Sedimentation Manual*, U.S. Department of Interior Bureau of Reclamation, 2006.
- (b) *Application of Methods and Models for Prediction of Land Surface Erosion and Yield*, Technical Document (TD) 36, U.S. Army Corps of Engineers Hydrologic Engineering Center, 1995.
- (c) *Sedimentation Engineering: Processes, Measurements, Modeling, and Practice*, Manual of Practice No. 110, ASCE, 2008.
- (d) *Debris Method, Los Angeles District Method for Prediction of Debris Yield*, U.S. Army Corps of Engineers, Los Angeles District, 2000.
- (e) *Construction BMP Handbook*, California Stormwater Quality Association, 2024

Attending seminars, webinars, or other training that focuses on sedimentation, such as those presented by the International Erosion Control Association (IECA), may also prove useful. More information can be found at www.ieca.org.

5.3 NEAR-TERM AND LONG-TERM SOIL LOSS CALCULATIONS

When an engineer is developing a hydrology study for a project, the study may include two calculations. The first calculation addresses “near-term” soil loss. Near-term refers to the construction phase of the project. During construction, significant erosion potential exists. Near-term erosion is generally controlled by installing devices such as gravel bags, straw wattles, silt fences, hay bales, trenches, temporary sediment basins, fiber blankets, and hydroseed.

The second calculation addresses “long-term” soil loss. After construction, erosion may still be an issue. Long-term erosion potential is dependent on the terrain, slope gradients, vegetative cover, root type, type of exposed soil, and intensity of rainfall experienced at the site. If an engineer finds that a site has a potential for long-term erosion, permanent sediment basins (or other devices) that will prevent the transport of sediment through storm drain infrastructure should typically be installed.

5.3.1 Methods of Soil Loss and Sediment Yield Prediction

Predicting soil loss or sediment yield is a science based on methods using interpolation of historic data, laboratory results, and derivation of formulas. There are a number of existing methods used to quantify soil losses or sediment yield.

5.3.1.1. Soil Loss Equations

Soil loss refers to the amount of soil eroded from a hillslope or other small plot. Soil loss equations include:

- Universal Soil Loss Equation (USLE)
- Revised Universal Soil Loss Equation, Version 2 (RUSLE 2)
- Water Erosion Prediction Project (WEPP) model;
GeoWEPP (<http://geowepp.geog.buffalo.edu>)

The Caltrans RUSLE 2 software predicts long-term, average annual erosion by rainfall. The software and training materials can be downloaded from the Caltrans website (<https://dot.ca.gov/programs/design/hydraulics-stormwater/bsddd-erosion-prediction->

[with-rusle2](#)). RUSLE 2 computes soil loss but can also be used for computing sediment yield depending on the project site and input parameters.

In general, the County of San Diego supports the updated version of the USLE method—RUSLE 2—in predicting soil losses. USLE-based equations are widely used throughout the U.S. and can produce accurate volumes of soil losses if the equation, graphs, and tables are used appropriately. It is also important that the engineer performs research to find the information required to use the nomographs and tables that produce the factors used in the equation.

The engineer should meet with the agency for approval before preparing soil loss calculations using a method other than RUSLE 2. The engineer should request an alternate method if it is the engineer’s opinion that the alternate method would result in calculation of more accurate quantities. If the alternate method is approved by the agency, the engineer shall describe the alternate method in the study/report. The engineer shall explain the steps used, including applicable portions of reference material used, and reasons for choosing the alternate method.

It is important to note that USLE and RUSLE 2 are primarily hillslope models that do not explicitly factor in gully or instream erosion. A study by the USDA estimated that approximately 60 percent of the sediment yield in the San Diego region is attributed to gully/instream processes.

5.3.1.2. Sediment Yield Equations

Sediment yield refers to the amount of sediment produced by a watershed, and may include a variety of sediment sources, including hillslope erosion, gully erosion, in-stream erosion, and mass wasting/movement. Sediment yield equations include (1) single storm-event equations, which compute watershed sediment yield for a single design discharge, and (2) average-annual equations, which compute sediment yield expected in an average year. It is important to note that single-event sediment yields in flashy ephemeral watersheds can often be several orders of magnitude greater than average annual values.

Single Storm-Event sediment yield equations include:

- Modified Universal Soil Loss Equation (MUSLE)
- US Army Corps of Engineers' Los Angeles District Method

Average-Annual sediment yield equations include:

- Flaxman's Method
- Renard's Equation
- Dendy and Bolton Equation

The appropriate sediment yield equation(s) will depend on the application (e.g., reservoir sedimentation, sediment/debris basin design, etc.) and the characteristics of the project watershed. The engineer should provide sound reasoning for the selected sediment yield method(s).

5.3.2 Basic Soil Loss Table

The County of San Diego accepts soil loss predictions using Table 5-1. Once the engineer identifies the average slopes and acreage of disturbed soil of a project, a resulting soil loss may be calculated. This table involves interpolation, but generally produces volumes more conservative than the USLE.

5.3.3 Universal Soil Loss Equation

The USLE was derived by Wischmeier and Smith in 1965 while working for the Agricultural Research Service (ARS). The ARS then performed significant research to estimate and fine-tune factors in the equation. The original purpose of the USLE was to predict soil losses in croplands east of the Rocky Mountains. However, the USLE was modified so that it could be used in different regions of the United States, including California.

Table 5-1

**BASIC SOIL LOSS TABLE
 (in cubic yards)***

TRACT AREA (acres)	AVERAGE SLOPES					
	2%	5%	8%	10%	12%	15%
10	270	350	370	400	450	500
15	400	420	460	600	675	750
20	540	700	740	800	900	1000
40	1080	1400	1480	1600	1800	2000
80	2160	2800	2960	3200	3600	4000
100	2700	3500	3700	4000	4500	5000
150	4000	4200	4600	6000	6750	7500
200	5400	7000	7400	8000	9000	10000

*Engineer shall interpolate the figures listed in the table as required.

The USLE accounts for all known factors affecting rainfall erosion and is calculated as:

$$A_s = RKLsCP$$

where:

- A_s = the computed soil loss in tons per acre per year (dry weight)
- R = the rainfall factor
- K = the soil erodibility factor
- L = the slope length factor
- s = the slope gradient factor
- C = crop management (vegetation) factor
- P = erosion control practice factor

5.3.3.1. Rainfall Factor (R)

The R factor is an index associated with the mean annual rainfall experienced at a particular location. An engineer should first obtain 2-year, 6-hour rainfall totals for a project using the NOAA Atlas 14 rainfall data (Appendix B). Once the rainfall depth is obtained, plot the value on Figure 5-2 to find the mean annual R Factor. Use line I as the pivot-point. This line applies to the San Diego Region. Lines IA and II apply to northern California and the eastern deserts, respectively. Refer to Figure 5-3.

5.3.3.2. Soil Erodibility Factor (K)

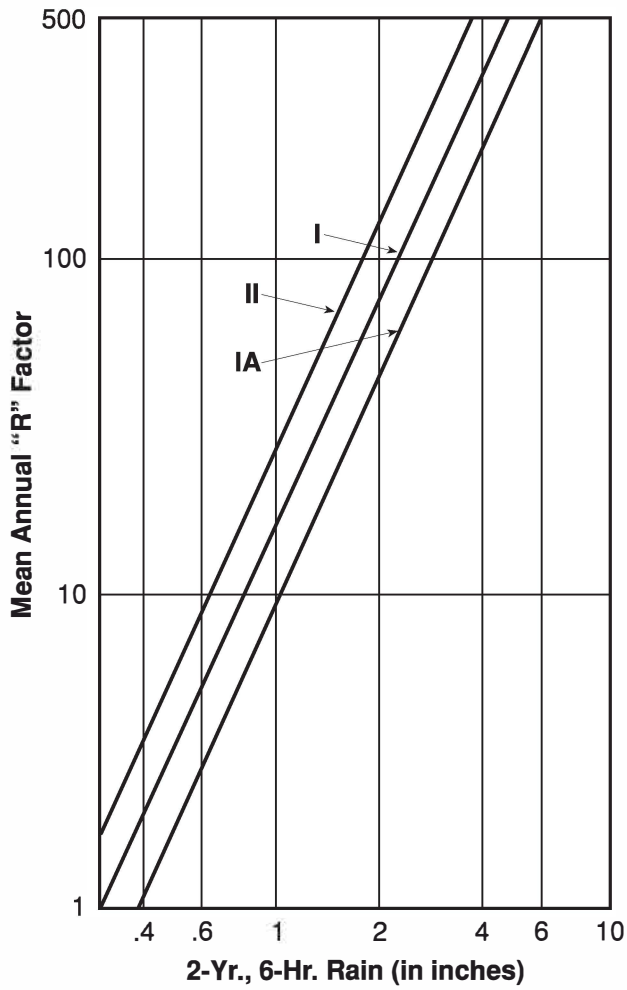
The K factor represents the potential erodibility a soil has based on its properties. The texture and gradation of the soil exposed during construction must be known or anticipated. In general, an engineer working on a new project may find soil properties from actual test results performed with the current project soils report. If a new soils report is not available, it is suggested the engineer review recent soils reports filed on the property or within the vicinity of the study area.

Table 5-2 provides a list of soil types, some of which are located in the San Diego Region. Once the soil classification is known, the engineer may use this list to find the K factor of the soil.

If specific soil characteristics of the soil are known or anticipated, there is another method to find the K factor by using the nomograph in Figure 5-4 (published by Wischmeier et al. 1965).

5.3.3.3. Slope Length and Steepness Factors (L,s)

The effect of length (L) and steepness (s) were established separately but are typically combined into a one single topographic factor. The combined Ls factor is graphed in Figure 5-5.

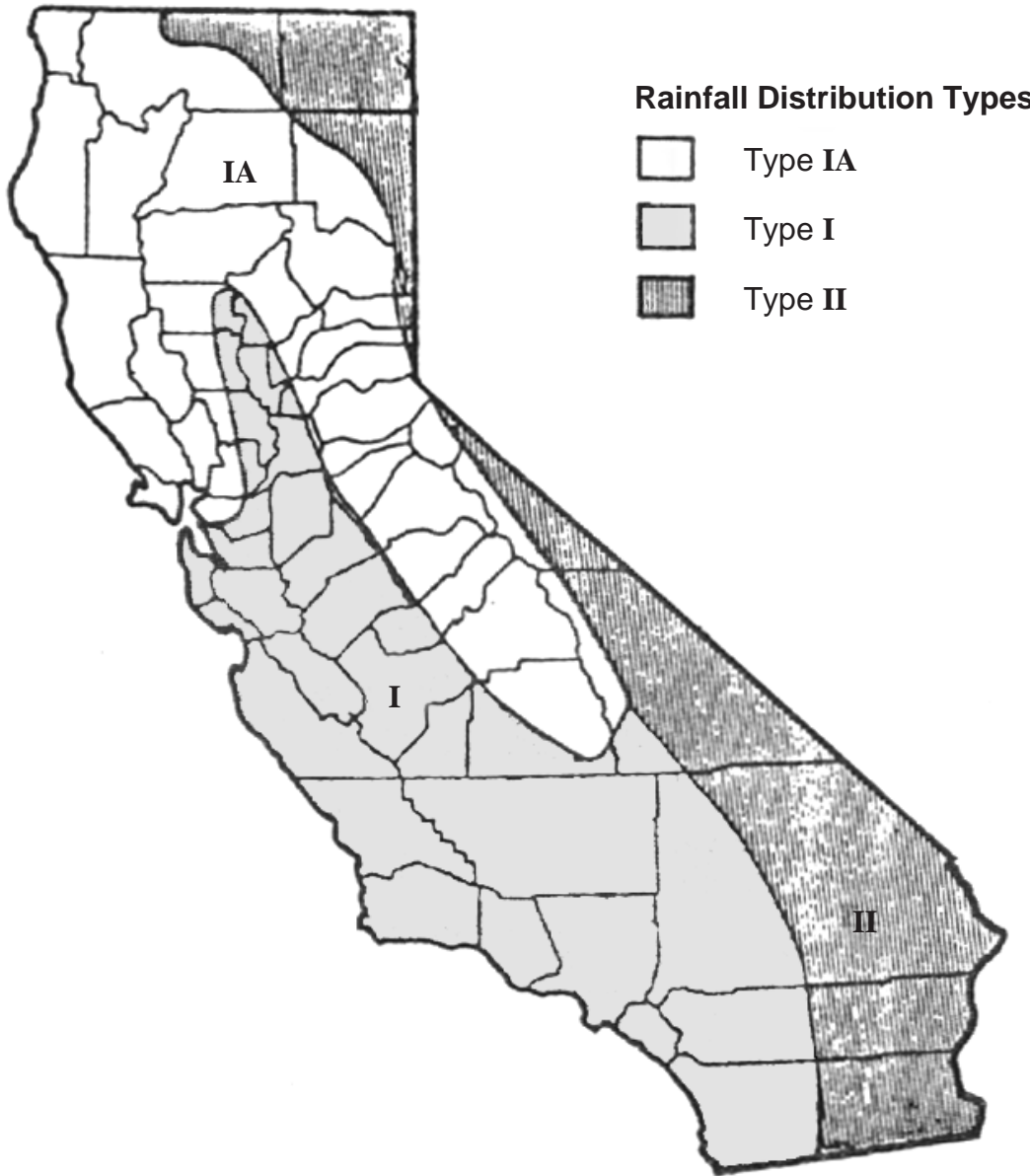


SOURCE: Wischmeier, 1977

R Factors based on 2-Year, 6-Hour Storm Event

FIGURE

5-2



SOURCE: Wischmeier, 1977

Rainfall Distribution Regions in California

FIGURE

5-3

Table 5-2

K FACTORS FOR SOILS IN THE SAN DIEGO REGION AND OTHERS

K factors for use within existing water erosion source areas
(using Universal Soil Loss Equation)

<u>MAPPING UNIT</u>	<u>K FACTOR</u>
1. Acid Igneous rockland (AcG)	*
2. AtD (Altamount clay 9-15% slopes)	.24
AtD2 (Altamount clay 9-15% slopes, eroded)	.24
AtE (Altamount clay 15-30%)	.24
AtE2 (Altamount clay 15-30%, eroded)	.24
AtF (Altamount clay 30-50%)	.24
3. AvC (Anderson very gravelly sandy loam 5-9% slopes)	0.15
AvF (Anderson very gravelly sandy loam 9-45% slopes)	0.15
4. AvC (Arlington coarse sandy loam 2-9% slopes)	0.17
5. AwC (Auld clay 5-9% slopes)	.28
AwD (Auld clay 9-15%)	.28
6. AyE (Auld stoney clay 9-30%)	.24
7. BaG (Badland)	*
8. BbE (Bancas stony loam 5-30%)	.28
BbE2 (Bancas stony loam 5-30% eroded)	.28
BbG (Bancas stony loam 30-65%)	.28
BbG2 (Bancas stony loam 30-65% eroded)	.28
9. BeE (Blasingame loam 9-30%)	.37
10. BgE (Blasingame stony loam 9-30%)	.32
BgF (Blasingame stony loam 30-50%)	.32
11. Bonsall (sandy loam 2-9% B1C)	.20
B1C2 (Bonsall sandy loam 2-9% eroded)	.20
B1D2 (Bonsall sandy loam 9-15% eroded)	.20
12. BmC (Bonsall sandy loam thick surface 2-9%)	.20
13. Bn9 (Bonsall-Fallbrook sandy loams 2-5%)	

Table 5-2 (Continued)

K FACTORS FOR SOILS IN THE SAN DIEGO REGION AND OTHERS

<u>MAPPING UNIT</u>	<u>K FACTOR</u>
14. BoC (Boomer loam 2-9% slopes)	.37
BoE (Boomer loam 9-30%)	.37
15. BrE (Boomer stony loam 9-30%)	.32
BrG (Boomer stony loam 30-65%)	.32
16. BsC (Bosanko clay 2-9%)	.28
BsD (Bosanko clay 9-15%)	.28
BsE (Bosanko clay 15-30%)	.28
17. BtC (Bosanko clay 5-9%)	N/A
18. BuB (Bull Trail sandy loam 2-5%)	.28
BuC (Bull Trail sandy loam 5-9%)	.28
BuD2 (Bull Trail sandy loam 9-15% eroded)	.28
BuE2 (Bull Trail 15-30% eroded)	.28
19. CaB (Calpine coarse sandy loam 2-5%)	0.15
CaC (Calpine coarse sandy loam 5-9%)	0.15
CaC2 (Calpine coarse sandy loam 5-9% eroded)	.20
CaD2 (Calpine coarse sandy loam 9-15% eroded)	.20
20. CbB (Carlsbad gravelly loamy sand 2-5%)	.17
CbC (Carlsbad gravelly loamy sand 5-9%)	.17
CbD (Carlsbad gravelly loamy sand 9-15%)	.17
21. CcC (Carlsbad-Urban land complex 2-9%)	*
CcE (Carlsbad-Urban land complex 9-30%)	*
22. CeC (Carrizo very gravelly sand 0-9%)	.10
23. CfB (Chesterton fine sandy loam 2-5%)	.24
CfC (Chesterton fine sandy loam 5-9%)	.24
CfD2 (Chesterton fine sandy loam 9-15% eroded)	.24
24. CgC (Chesterton-Urban land complex 2-9%)	*
25. ChA (Chino fine sandy loam 0-2% slopes)	.24
ChB (Chino fine sandy loam 2-5%)	.24

Table 5-2 (Continued)

K FACTORS FOR SOILS IN THE SAN DIEGO REGION AND OTHERS

MAPPING UNIT	K FACTOR
26. CkA (Chino silt loam saline 0-2%)	.43
27. CID2 (coarse sandy loam 5-15%)	.24
CIE2 (coarse sandy loam 15-30%)	.24
CIG2 (coarse sandy loam 30-65%)	.24
28. CmE2 (rocky coarse sandy loam 9-30%)	.24
29. CmrG (very rocky coarse sandy loam 30-75%)	.20
30. CnE2 (rocky sandy loam 9-30%)	*
CnG2 (rocky sandy loam 30-65%)	*
31. Co (clayey alluvial land)	.24
32. Cr (coastal beaches)	*
33. CsB (loamy sand 0-5%)	.20
CsC (loamy sand 5-9%)	.20
SsD (loamy sand 9-15%)	.20
34. CtE (coarse sandy loam 5-30%)	.17
CtF (coarse sandy loam 30-50%)	.17
35. CuE (rocky coarse sandy loam 5-30%)	.15
CuG (rocky coarse sandy loam 30-70%)	.15
36. CuG (stony fine sandy loam 30-75%)	.15
37. DaC (clay 2-9%)	.24
DaD (clay 9-15%)	.24
DaE (clay 15-30%)	.24
DaE2 (clay 15-30%)	.24
DaF (clay 30-50%)	.24
38. DcD (urban land complex 5-15%)	*
DcF (urban land complex 15-50%)	*
39. DoE (urban land complex 9-30%)	*
40. EdC (shaly fine sandy loam 2-9%)	.17

Table 5-2 (Continued)

K FACTORS FOR SOILS IN THE SAN DIEGO REGION AND OTHERS

<u>MAPPING UNIT</u>	<u>K FACTOR</u>
41. EsC (very fine sandy loam 5-9%)	.43
EsD2 (very fine sandy loam 9-15%)	.43
EsE2 (very fine sandy loam 15-30%)	.43
EvC (very fine sandy loam 5-9%)	.43
42. ExE (rocky silt loam 9-30%)	.43
ExG (rocky silt loam 30-70%)	.43
43. FaB (sandy loam 2-5%)	.28
FaC (sandy loam 5-9%)	.28
FaC2 (sandy loam 5-9%)	.28
FaD2 (sandy loam 9-15%)	.28
FaE2 (sandy loam 15-30%)	.28
FaE3 (sandy loam 9-30%)	.24
44. FeC (rocky sandy loam 5-9%)	.24
FeE (rocky sandy loam 9-30%)	.24
FeE2 (rocky sandy loam 9-30%)	.24
45. FvD (sandy loam 9-15%)	*
FvE (sandy loam 15-30%)	*
46. FwF (fine sandy loam 30-50%)	.32
47. FxE (rocky fine sandy loam 9-30%)	.32
FxG (rocky fine sandy loam 30-70%)	.32
48. GaE (fine sandy loam 9-30%)	.43
GaF (fine sandy loam 30-50%)	.43
49. GoA (fine sandy loam 0-2%)	.20
50. GrA (sandy loam 0-2%)	.24
GrB (sandy loam 2-5%)	.24
GrC (sandy loam 5-9%)	.24
GrD (sandy loam 9-15%)	.24
51. HaG (gravelly clay loam 30-75%)	.17

Table 5-2 (Continued)

K FACTORS FOR SOILS IN THE SAN DIEGO REGION AND OTHERS

MAPPING UNIT	K FACTOR
52. HmD (fine sandy loam 5-15%)	.24
HmE (fine sandy loam)	.24
53. HnE (stony fine sandy loam 5-30%)	.20
HnG (stony fine sandy loam 30-60%)	.20
54. HoC (fine sandy loam, deep 2-9%)	.24
55. HrC (loam 2-9%)	.37
Hrc2 (loam 5-9%)	.37
HrD (loam 9-15%)	.32
HrE2 (loam 15-30%)	.32
56. HuC (urban land complex 2-9%)	*
HuE (urban land complex 9-30%)	*
57. InA (silt loam 0-2%)	.55
InB (silt loam 2-5%)	.55
IoA (saline 0-2%)	.55
58. IsA (silt loam, dark variant)	.49
59. KcC (loamy coarse sand 5-9%)	.17
KcD2 (loamy coarse sand 9-15%)	.17
60. LaE2 (loamy coarse sand 5-30%)	.17
LaE3 (loamy coarse sand 5-30%)	.17
61. LcE (rocky loamy coarse sand 5-30%)	.15
LcE2 (rocky loamy coarse sand 5-30%)	.15
LcF2 (rocky loamy coarse sand 30-50%)	.15
62. LdE (complex 9-30%)	*
LdG (complex 30-65%)	*
63. LeC (2-(%)	.17
LeC2 (5-9%)	.17
LeD (9-15%)	.17
LeD2 (9-15%)	.17

Table 5-2 (Continued)

K FACTORS FOR SOILS IN THE SAN DIEGO REGION AND OTHERS

MAPPING UNIT	K FACTOR
LeE (15-30%)	.17
LeE2 (15-30%)	.15
LeE3 (9-30%)	.15
64. LfC (urban land complex 2-9%)	*
LfE (urban land complex 9-30%)	*
65. LpB (fine sandy loam 2-5%)	.28
LpC (fine sandy loam 5-9%)	.28
LpC2 (fine sandy loam 5-9%)	.28
LpD2 (fine sandy loam 9-15%)	.28
LpE2 (fine sandy loam 15-30%)	.28
66. LrE (stony fine sandy loam 9-30%)	.24
LrE2 (stony fine sandy loam 9-30%)	.24
LrG (fine sandy loam 30-65%)	.24
67. LsE (clay loam 9-30%)	.24
LsF (clay loam 30-50%)	.24
68. Lu (loamy alluvial land)	*
69. LuF3 (loamy alluvial land - Huerhuro complex 9-50%)	*
70. Md (made land)	*
71. MlC (loamy coarse sand 2-9%)	.10
MlE (loamy coarse sand 9-30%)	.10
72. MnA (coarse sandy loam 0-2%)	.17
MnB (coarse sandy loam 2-5%)	.17
73. MdA (sandy loam 0-2%)	.17
74. MpA2 (fine sandy loam 0-2%)	.20
75. MrG (metamorphic rock land)	*
76. MvA (loamy coarse sand 0-2%)	.24
MvC (loamy coarse sand 2-9%)	.24
MvD (loamy coarse sand 9-15%)	.24

Table 5-2 (Continued)

K FACTORS FOR SOILS IN THE SAN DIEGO REGION AND OTHERS

MAPPING UNIT	K FACTOR
MxA (loamy coarse sand wet 9-2%)	.24
77. OhC (cobbly loam 2-9%)	.28
OhE (cobbly loam 9-30%)	.28
OhF (cobbly loam 30-50%)	.28
78. OkC (urban land complex 2-9%)	*
OkE (urban land complex 9-30%)	*
79. PeA (sandy loam 0-2%)	.32
PeC (sandy loam 2-9%)	.32
PeC2 (sandy loam 5-9%)	.32
PeD2 (sandy loam 9-15%)	.32
PfA (thick surface 0-2%)	.32
PfC (thick surface 2-9%)	.32
80. Py	*
81. RaA (sandy loam 0-2%)	.32
RaB (sandy loam 2-5%)	.32
RaC (sandy loam 5-9%)	.32
RaC2 (sandy loam 5-9%)	.32
RaD2 (sandy loam 9-15%)	.32
82. RcD (gravelly sandy loam 9-15%)	.32
RcE (gravelly sandy loam 15-30%)	.32
83. RdC (gravelly loam 2-9%)	.32
84. ReE (cobbly loam 9-30%)	.28
RfF (cobbly loam 15-50%)	.28
85. RhC (urban land complex 2-9%)	*
RhE (urban land complex 9-30%)	*
86. RkA (fine sandy loam 0-2%)	.32
RkB (fine sandy loam 2-5%)	.32
RkC (fine sandy loam 5-9%)	.32

Table 5-2 (Continued)

K FACTORS FOR SOILS IN THE SAN DIEGO REGION AND OTHERS

MAPPING UNIT	K FACTOR
87. Rm (riverwash)	*
88. RoA (fine sand 0-2%)	.17
RrC (fine sand 5-9%)	.17
89. RsA (loamy coarse sand 0-2%)	.20
RsC (loamy coarse sand 2-9%)	.20
RsD (loamy coarse sand 9-15%)	.20
90. (rough broken land)	*
91. SbA (clay loam 0-2%)	.37
SbC (clay loam 2-9%)	.37
92. ScA (clay 0-2%)	.24
ScB (clay 2-5%)	.24
93. SmE (rocky silt loam 9-30%)	.43
94. SnG (rocky silt loam 9-70%)	.43
95. SpE2 (rocky fine sandy loam 9-30%)	.28
SpG2 (rocky fine sandy loam 30-65%)	.28
96. SrD (sloping gullied land)	*
97. SsE (stony loamy sand 9-30%)	.15
98. StG (steep gullied land)	*
99. SuA (gravelly clay loam 0-2%)	.24
SuB (gravelly clay loam 2-5%)	.24
100. SvE (stony land)	*
101. TeF (terrace escarpments)	*
102. Tf (tidal flats)	*
103. ToE2 (rocky coarse sandy loam 5-30%)	.24
ToG (rocky coarse sandy loam 30-65%)	.24
104. TuB (sand 0-5%)	.17
105. Urban land	*

Table 5-2 (Continued)

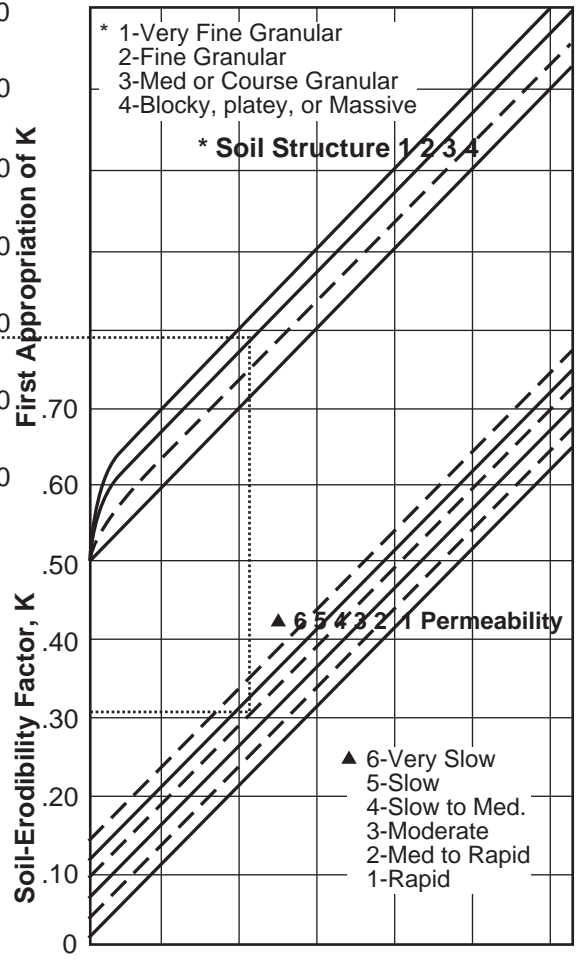
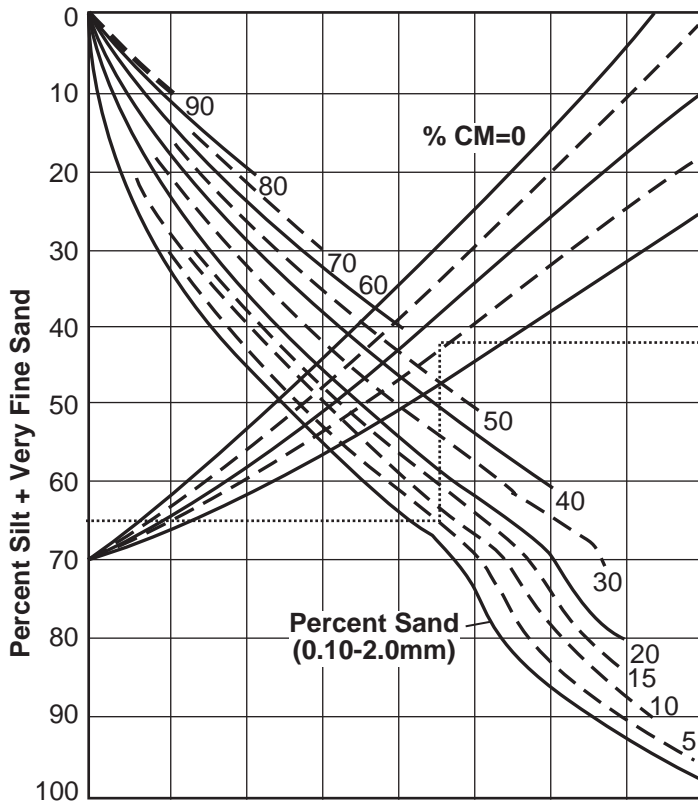
K FACTORS FOR SOILS IN THE SAN DIEGO REGION AND OTHERS

<u>MAPPING UNIT</u>	<u>K FACTOR</u>
106. VaA (sandy loam 2-5%)	.28
VaC (sandy loam 5-9%)	.28
VaD (sandy loam 9-15%)	.28
107. VbB (gravelly sandy loam 2-5%)	.28
VbC (gravelly sandy loam 5-9%)	.28
108. VsC (coarse sandy loam)	.28
VsD (coarse sandy loam)	.28
VsD2 (coarse sandy loam)	.28
VsE (coarse sandy loam)	.28
VsE2 (coarse sandy loam)	.28
VsG (coarse sandy loam)	.28
109. VvD (rocky coarse sandy loam 5-15%)	.28
VvE (rocky coarse sandy loam 15-30%)	.28
VvG (rocky coarse sandy loam 30-65%)	.28
110. WmB (loam 2-5%)	.43
WmC (loam 5-9%)	.43
WmD (loam 9-15%)	.43

Footnotes:

*Too variable to determine

For complexes, use the individual soils within the complex to determine K



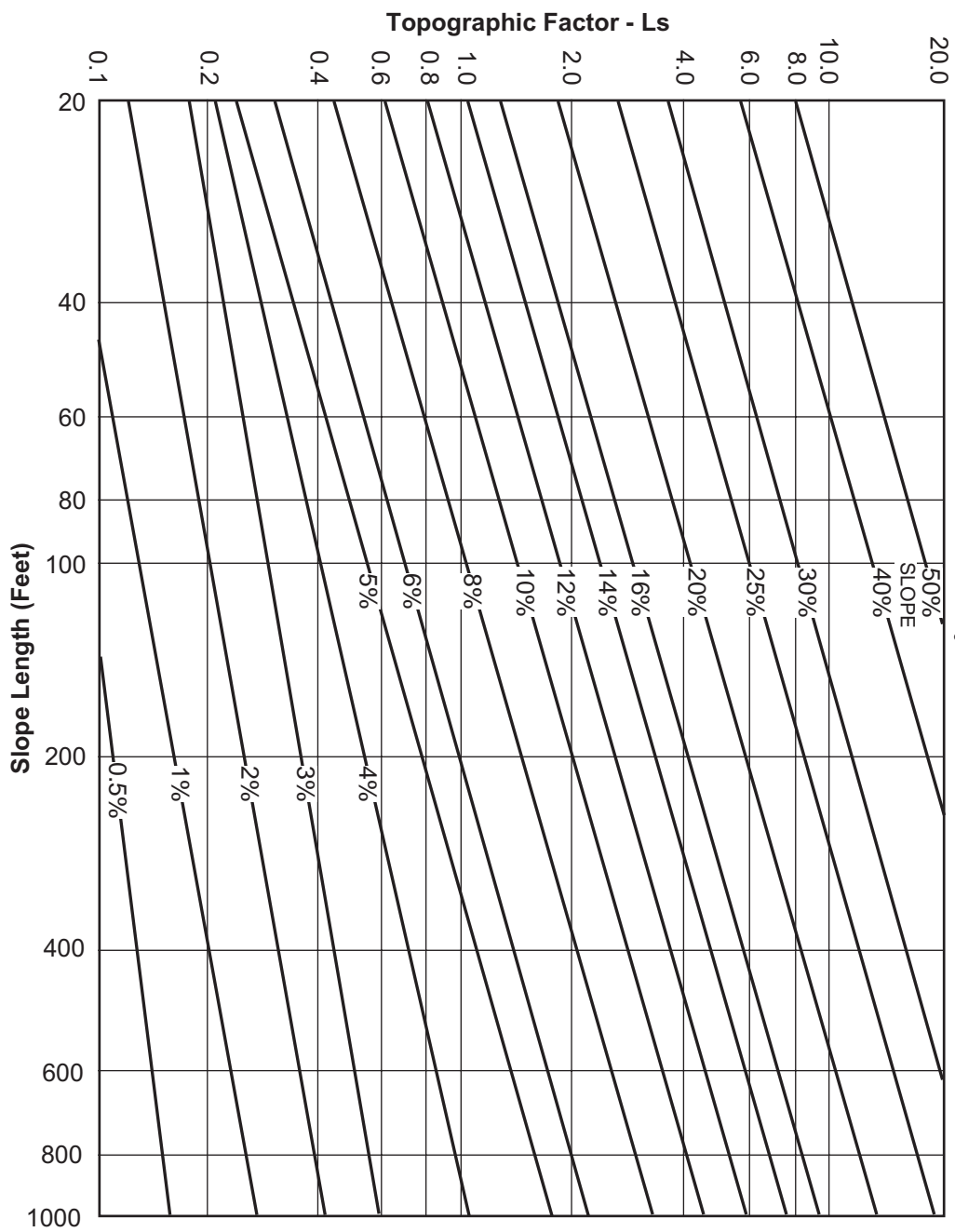
PROCEDURE: With appropriate data, enter scale at left and proceed to points representing the soils's S sand (0.10-2.0), S Organic Matter, Structure, and Permeability, in that sequence. Interpolate between plotted curves. The dotted line illustrates procedure for a soil having: sl+vfs 65%, sand 5%, OM 2.8%, structure 2, permeability 4, Solution: K = 0.31

Where the silt fraction does not exceed 70 percent, the equation is $100 K = 2.1 M^{1.14} (10^{-4}) (12-a) + 3.25 (b-2) + 2.5 (c-3)$ where M= (percent sl + vfs) (100-percent c), a = percent organic matter, b = structure code, and c = profile permeability class.

SOURCE: SEA, USDA, Agricultural Handbook Number 537, December 1978

Nomograph for Soil Erodibility Factors

FIGURE
5-4



SOURCE: SEA, USDA, Agricultural Handbook Number 537, December 1978

F I G U R E

5-5

Chart for determining Topographic Factors (L,s)

5.3.3.4. Cropping Management Factor (*C*)

The cropping management factor (*C*) represents the reduction in soil losses resulting from the effects of vegetation on a site. For complete, bare ground, the *C* factor is 1. As vegetation increases, the *C* factor value decreases to reflect the additional erosion protection. Please refer to the comprehensive list of *C* factors for permanent pasture, rangeland, and idle ground in Table 5-3. *C* factors for woodland are in Table 5-4. *C* factors for annual cover and various quantities of mulch are in Table 5-5. *C* factor and *P* factor values for rainfall erosion control measures are in Table 5-6 and *C* factors for established grass and ground cover are found in Figure 5-6.

5.3.3.5. Erosion Control Practice Factor (*P*)

This factor represents the reduction in soil losses resulting from the implementation of soil conservation measures. These measures may include but are not limited to contouring, terracing, or installing vegetation; mechanical devices; chemical devices; or combinations thereof. Standard *P* factor values can be found in Table 5-6.

5.3.3.6. Calculating Soil Loss (A_s)

Once all known factors of the USLE are identified, multiply them together and the result (A_s) will be the estimated soil loss per year for the site. The units of A_s will be in tons of soil (dry weight) per acre per year. To estimate volume of soil loss, convert A_s to weight, in pounds (use multiplier 2,000 pounds per 1 ton). Then calculate volume by dividing the weight (in pounds) by the density (pounds per cubic foot) of soil per the test results of the soils report. The final result will be the volume of estimated soil loss (in cubic feet).

Table 5-3

C FACTORS FOR PASTURE, RANGELAND, AND IDLE GROUND¹

Vegetal Canopy			Cover That Contacts the Surface					
Type and Height of Raised Canopy ²	Canopy Cover ³ %	Type ⁴	Percent Ground Cover					
			0	20	40	60	80	95-100
Column No.:	2	3	4	5	6	7	8	9
No appreciable canopy		G	.45	.20	.10	.042	.013	.003
		W	.45	.24	.15	.090	.043	.011
Canopy of tall weeds or short brush (0.5 m fall ht.)	25	G	.36	.17	.09	.038	.012	.003
		W	.36	.20	.13	.082	.041	.011
	50	G	.26	.13	.07	.035	.012	.003
		W	.26	.16	.11	.075	.039	.011
	75	G	.17	.10	.06	.031	.011	.003
		W	.17	.12	.09	.067	.038	.011
Appreciable brush or brush (2 m fall ht.)	25	G	.40	.18	.09	.040	.013	.003
		W	.40	.22	.14	.085	.042	.011
	50	G	.34	.16	.085	.038	.012	.003
		W	.34	.19	.13	.081	.041	.011
	75	G	.28	.14	.08	.036	.012	.003
		W	.28	.17	.12	.077	.041	.011
Trees but no appreciable low brush (4 m fall ht.)	25	G	.42	.19	.10	.041	.013	.003
		W	.42	.23	.14	.087	.042	.011
	50	G	.39	.18	.09	.040	.013	.003
		W	.39	.21	.14	.085	.042	.011
	75	G	.36	.17	.09	.039	.012	.003
		W	.36	.20	.13	.083	.041	.011

Source: Gray and Leiser 1982.

¹ All values shown assume (1) random distribution of mulch or vegetation, and (2) mulch of appreciable depth where it exists.

² Average fall height of waterdrops from canopy to soil surface: m = meters.

³ Portion of total-area surface that would be hidden from view by canopy in a vertical projection (a bird's-eye view).

⁴ G: Cover at surface is grass, grasslike plants, decaying compacted duff, or litter at least 2 inches deep.
W: Cover at surface is mostly broadleaf herbaceous plants (as weeds) with little lateral-root network near the surface, and/or undecayed residue.

Table 5-4

C FACTORS FOR WOODLAND

Stand Condition	Tree Canopy ¹ % or Area	Forest Litter % of Area ²	Undergrowth ³	C Factor
Well Stocked	100-75	100-90	Managed ⁴	.001
			Unmanaged ⁴	.003-.011
Medium Stocked	70-40	85-75	Managed	.002-.004
			Unmanaged	.01-.04
Poorly Stocked	35-20	70-40 ⁵	Managed	.003-.009
			Unmanaged	.02-.09 ⁵

Source: USDA Soil Service 1978; Gray and Leiser 1982.

¹ When tree canopy is less than 20%, the area will be considered as grassland or cropland for estimating soil loss. See Table 5-3.

² Forest litter is assumed to be at least 2 inches deep over the percent ground surface area covered.

³ Undergrowth is defined as shrubs, weeds, grasses, vines, etc., on the surface area not protected by forest litter. Usually found under canopy openings.

⁴ Managed - grazing and fires are controlled.

Unmanaged - stands that are overgrazed or subjected to repeated burning.

⁵ For unmanaged woodland with litter cover of less than 75%, C values should be derived by taking 0.7 of the appropriate values in Table 5-3. The factor of 0.7 adjusts for the much higher soil organic matter on permanent woodland.

Table 5-5

**C FACTORS FOR ANNUAL COVER AND
VARIOUS QUANTITIES OF MULCH¹**

Cover or Mulch	C Factor
bare areas	1.0
¼ ton straw mulch	0.52
½ ton straw mulch	0.35
¾ ton straw mulch	0.24
1 ton straw mulch	0.18
1½ ton straw mulch	0.10
2 tons straw mulch	0.06
3 tons straw mulch	0.03
4 tons straw mulch	0.02
annual cover	0.15

Source: USDA Soil Service 1978; Gray and Leiser 1982.

¹ When tree canopy is less than 20%, the area will be considered as grassland or cropland for estimating soil loss. See Table 5-3.

Table 5-6 (Page 1 of 2)

**C FACTOR AND P FACTOR VALUES FOR RAINFALL
 EROSION CONTROL MEASURES**

Treatment	C Factor	P Factor
BARE SOIL		
Packed and Smooth.....	1.00	1.00
Freshly Disked	1.00	0.90
Rough Irregular Surface.....	1.00	0.90
SEDIMENT BASIN/TRAP	1.00	0.50 ^A
STRAW BALE BARRIER, GRAVEL FILTER, SAND BAGS	1.00	0.80
SILT FENCE BARRIER	1.00	0.50
ASPHALT/CONCRETE PAVEMENT.....	1.00	1.00
GRAVEL (¼" to 1½") @ 135 TONS/ACRE	0.05	1.00
SOD GRASS.....	0.01	1.00
TEMPORARY VEGETATION/COVER CROP	0.45 ^B	1.00
HYDRAULIC MULCH @ 2 TONS/ACRE.....	0.10 ^C	1.00
SOIL SEALANT	0.01 - 0.60 ^D	1.00
EROSION CONTROL MATS/BLANKETS	0.10	1.00
HAY OR STRAW DRY MULCH @ 2 TONS/ACRE & ANCHORED		
Assumes planting of grass seed has occurred prior to application, otherwise C Factor = 1.00.		
<u>Slope (%)</u>		
1 to 10	0.06	1.00
11 to 15	0.07	1.00
16 to 20	0.11	1.00
21 to 25	0.14	1.00
25 to 33	0.17	1.00
> 33.....	0.20	1.00

Table 5-6 (Page 2 of 2)

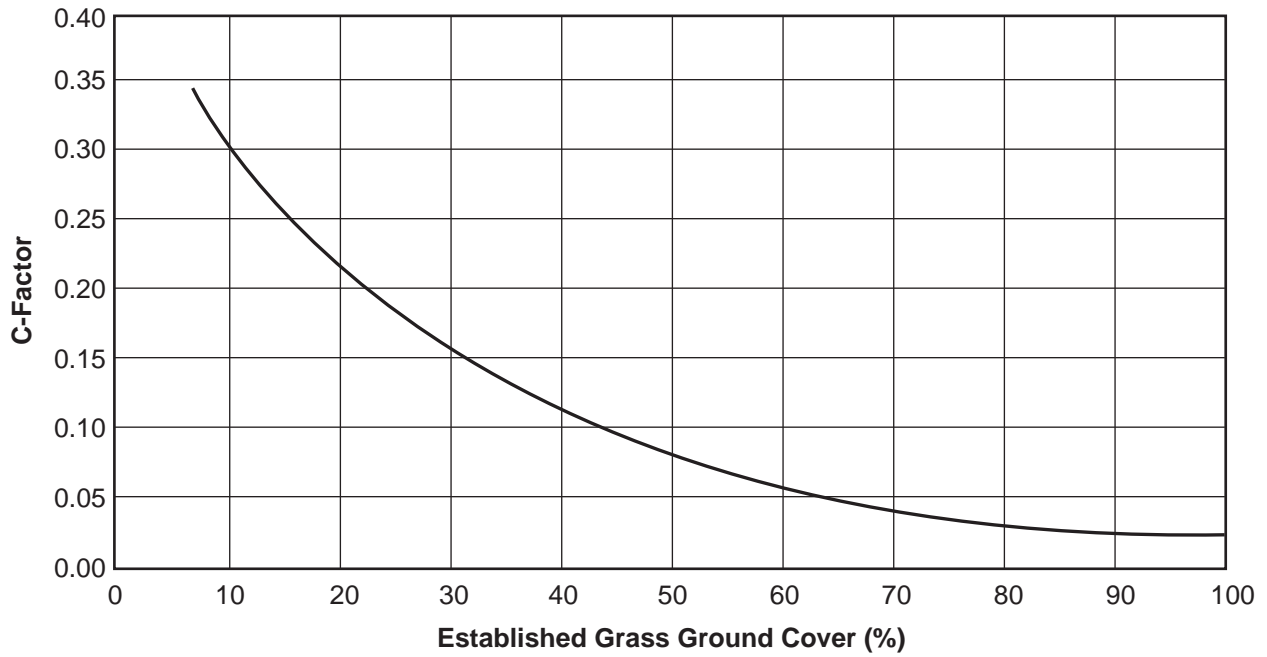
**C FACTOR AND P FACTOR VALUES FOR RAINFALL
 EROSION CONTROL MEASURES**

Treatment	C Factor	P Factor
CONTOUR FURROWED SURFACE		
Must be maintained throughout construction activities, otherwise P Factor = 1.00. Maximum length refers to down slope length.		
<u>Slope (%)</u>	<u>Max. Length (feet)</u>	
1 to 2	400.....	1.00 0.60
3 to 5	300.....	1.00 0.50
6 to 8	200.....	1.00 0.50
9 to 12	120.....	1.00 0.60
13 to 16	80.....	1.00 0.70
17 to 20	60.....	1.00 0.80
> 20	50.....	1.00 0.80
TERRACING		
Must contain 10-year runoff volumes without overflowing, otherwise P Factor = 1.00		
<u>Slope (%)</u>		
1 to 2	1.00 0.12
3 to 8	1.00 0.10
9 to 12	1.00 0.12
13 to 16	1.00 0.14
17 to 20	1.00 0.16
> 20	1.00 0.18
GRASS BUFFER STRIPS TO FILTER SEDIMENT LADEN SHEET FLOWS		
Strips must be at least 125 feet wide and have a ground cover value of 50% or greater, otherwise P Factor = 1.00.		
<u>Basin Slope</u>		
0% to 10%	1.00 0.60
10% to 24%	1.00 0.80

Source: IECA 1996.

NOTE: Use of C Factor or P Factor values other than reported in this table should be substantiated by documentation.

- ^A Should be constructed as the first step in over lot grading
- ^B Assumes planting occurs within optimal climatic conditions
- ^C Some limitation on use in arid and semi-arid climates
- ^D Value used must be substantiated by documentation



Ground cover represents the portion of total soil surface area occupied by grass when viewed from above.

SOURCE: IECA Conference, February 1996

C-Factors for Established Grass

F I G U R E

5-6

5.4 SEDIMENT TRANSPORT

5.4.1 Natural Streambed

Soil particles are typically displaced by wind or water through the erosion process as previously described. Sediment transport is defined as soil particles moved from one location to another. During a rainstorm event (or inadvertent flooding, such as a water main break) erosion may take place on any area of exposed soil subjected to the sufficient water velocity and momentum of a watercourse. Erosion will usually occur in a watercourse (e.g., riverbed, ravine, stream, channel, or gully) when the velocity of water exceeds the weight and cohesion of the soil particle being displaced. Once displaced, the soil particle becomes suspended in the water for a period of time. The soil particle travels with the water and is eventually deposited downstream. The distance the soil particle will travel depends on many things, including the weight of the soil particle, volume and velocity of the water displacing it, storm duration, peak storm flow characteristics, topographic features of the watershed, and character of the watercourse.

Sediment transport is an event that has recurred throughout history, evidenced by beach sand brought from the tops of mountains. As sediment is displaced and transported downstream, sediment transported from upstream typically fills the remaining void (see Figure 5.7).

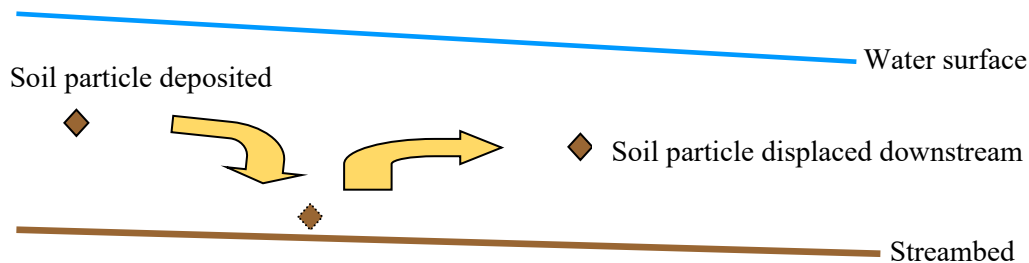


Figure 5-7
Sediment Transport

In general, an established watercourse is said to have reached a relative equilibrium if the displaced sediment is replaced equally by sediment displaced from upstream. If the upstream supply of sediment is not enough to replace the downstream sediment volume being transported, this will lead to lower channel bed elevations, better known as scour or degradation. Conversely, if the inflow of upstream sediment exceeds downstream sediment volumes, this will lead to an increase in the channel bed elevations, better known as deposition or aggradation. Thus, changes in sediment transport lead to changes in the channel bed elevations.

Channel bed elevations are important to any facility crossing a watercourse (e.g., bridge abutments, bridge piers, storm drain outlet, gas main, or sewer main). These facilities each require designs with assumed finished grade elevations. If a proposed facility crosses any major watercourse, an analysis must be prepared to predict anticipated changes in channel bed elevations/profiles. This information is critical to designing support structures of a bridge or verifying whether a buried facility will be exposed during severe storm events.

The combination of engineered infrastructure and increased development presents difficulties in forecasting how sediment is transported in natural channels. One example of how infrastructure influences sediment transport is the construction of a bridge in a riverbed. The bridge would act as a constriction, resulting in increased velocities and an increase in sediment transport. This condition is apparent in scoured bridge crossings where the foundations of bridge piers are exposed. A worst-case scenario would be the construction of a dam, which virtually eliminates the transport of sediment from areas upstream of the dam. An example of the influence of development upon sediment transport is the development of an existing 15-acre scrub brush land into an industrial complex with large buildings, parking lots, streets, and storm drains. This project would no doubt result in increased storm runoff quantities due to increased imperviousness of the buildings, parking lots, and streets, and shorter time of concentration. If an area is studied just downstream of a new development, this increased storm runoff has a potential to transport a higher volume of sediment than before the project was constructed. These are the issues that must be confronted and resolved prior to construction.

As an engineer, you should consider the effect sediment transport may have on any existing or proposed facilities within your project area. Your efforts should also expand to consider the effect your project will have on downstream projects (off-site) with respect to sediment transport. The process of sediment transport contains many complexities and variables and is typically forecasted by computer models. A channel bed study should describe the watershed area and characteristics, study limits of the channel, channel characteristics, soil types, model chosen, results of water surface profiles and channel bed, and technical appendices.

5.4.2 Development

After a project has been constructed in urban development, the potential for erosion may diminish as buildings are erected, surfaces are paved or improved, underground storm drains are installed, average slopes are reduced, and vegetation is established. However, while the soil loss at the project site decreases if it is paved, increased runoff may also occur, resulting in increased erosion in downstream receiving waters due to the “hungry water” effect. Regulations and guidance related to hydromodification is provided in the *County of San Diego BMP Design Manual*.

Some projects include the construction of storm drain infrastructure that collects stormwater from off-site or on-site tributary areas that are still undeveloped. These areas may be developed in the future, or they may remain undeveloped if they are designated as open space by an agency. Some areas are subjected to fires that obliterate all vegetation. In any case, the engineer should consider the potential soil loss generated from areas like these. Storm drain infrastructure should be self-cleaning so that velocities are high enough to transport particles downstream.

An engineer should also design the storm drain to meet the standard service life of a public storm drain system. If sand or particles are suspended in the water, this will increase the water’s ability to scour the surface of the storm drain. This scouring should be analyzed on proposed storm drains, especially those installed down steep slopes. Typically, in this situation the engineer would choose a thicker-walled concrete storm drain, or redesign the

storm drain to reduce scouring velocities. Refer to the *San Diego County Hydraulic Design Manual* for additional information on storm drain design.

5.4.3 River Engineering

At times, a project may involve constructing improvements crossing creeks, streams, rivers, and other areas subject to floodwaters. These improvements may include but are not limited to streets, bridge abutments, bridge piers, underground utilities, storm drain outlets, or flood control devices (e.g., weirs and flumes). These areas may be susceptible to changes in grades while sediment is transported downstream through what is called a fluvial process. A model is chosen to represent the study area and a series of calculations are performed. Typically, the engineer will submit a computer model due to the complexity of the calculations. The purpose of the calculations is to predict the maximum scour or deposition of soil along the channel bed. This prediction depends on channel characteristics, velocity of water, momentum of water, soil particle size, channel slope, and other criteria. As part of the design, the engineer must include sufficient theory and calculations to verify the improvements will endure anticipated changes in grades due to scour or soil deposition. It is recommended the owner/developer seek a professional engineer experienced in predicting changes in creeks, channels, or riverbeds to perform these models and calculations.

5.4.4 Supplemental References

The following are example references that may be used to supplement the theory of sediment transport in channels:

- (a) *Sedimentation Engineering: Processes, Measurements, Modeling, and Practice*, Manual of Practice No. 110, ASCE, 2008.
- (b) *Erosion and Sedimentation Manual*. U.S. Department of the Interior Bureau of Reclamation, 2006. <https://www.usbr.gov/library/reclamationpubs.html>
- (c) *Sediment and Erosion Design Guide*. Prepared by MEI; prepared for Southern Sandoval County Arroyo Flood Control Authority, 2008.

https://www.sscfca.org/development/documents/sediment_design_guide/Sediment%20Design%20Guide%2012-30-08.pdf

Note: This design guide discusses sediment transport and stream stability principles that apply to arid regions, including San Diego County.

- (d) *Mechanics of Sediment Transportation and Alluvial Stream Problems – Third Edition*. R.J. Garde, 2000.
- (e) *Fluvial Processes in River Engineering*. Howard H. Chang, 1988.

5.5 EROSION CONTROL

5.5.1 Introduction

If an engineer determines that a project has potential to generate certain quantities of sediment, the engineer should then make decisions about implementation of appropriate erosion control devices. The requirement to install erosion control is generally triggered by agency codes, ordinances, conditions of approval, agency grading permits, conditions of California Environmental Quality Act (CEQA) findings based on impacts, and (the latest) National Pollutant Discharge Elimination System (NPDES) permit requirements. Generally, during the grading plan review process, an agency will require the engineer to prepare erosion control plans to address the potential for erosion during construction of the project and the devices to be installed. With these plans, the contractor and agency inspector can ensure that sufficient erosion control devices are available year-round so the project can be secured in the event rainfall is forecasted or unexpectedly occurs. The owner, contractor, and agencies must ensure adequate erosion control devices are installed to prevent eroded material from exiting the site. The engineer should consult the appropriate local agency to determine acceptable best management practices (BMPs) for a project.

5.5.2 Erosion Control Devices

The purpose of erosion control devices is to capture or limit the anticipated erosion from a particular site for a design storm event. Each device has its own efficiency at capturing or limiting sediment yield. The engineer should choose the appropriate devices for each

project. During construction phases of a project, the erosion control plan may require modifications due to unforeseen circumstances (e.g., intermediate topographic changes, availability of certain erosion control items, and unexpected rainstorms).

For a list of erosion control devices, please refer to the *County of San Diego BMP Design Manual* and the Caltrans Standards and website as these are the County's primary standards. This is a comprehensive list of erosion control devices that can be implemented during construction. The list also includes an explanation on each device, its purpose, and applicability. Please note there may be other devices that the industry may provide that are not covered in this handbook. Additional information is provided in the California Stormwater Quality Association (CASQA) BMP Handbooks (www.casqa.org/resources/bmp-handbooks).

5.5.3 Erosion Control Plan

During design of grading plans, agencies may require the preparation of an erosion control plan. Erosion control plans are required year-round. The engineer should contact the agency to determine whether an erosion control plan should accompany the submittal. An erosion control plan is a guide for both the contractor and the agency inspector on how to protect the site and adjacent lands from erosion. However, the drainage characteristics of a site under construction will not be the same day to day. This is due to various grading operations, temporary excavations, temporary stockpiles, and so on. An erosion control plan cannot address every stage of the project. Although a difficult task, the engineer should try to determine most scenarios that will arise during construction and what methods the contractor can employ to ensure that stormwater erosion is controlled. A well-designed erosion control plan allows the contractor to order sufficient devices before the rainy season. The agency inspector can then verify that adequate devices are installed. If conditions arise on the site where the erosion control plan does not supply sufficient guidance to address a condition, the agency inspector may either offer guidance or require the engineer to provide the inspector and contractor with additional devices or measures to address the condition.

5.5.4 Maintenance of Erosion Control Devices

Once erosion control devices (temporary or permanent) are installed as part of a project, the captured eroded material should then be removed from the devices on a regular basis (typically after each rainfall event). This maintenance should be performed so that the efficiency of the devices is adequate to capture sediments from future rainfall events. Improper maintenance of erosion control devices can also lead to the deposition of sediments downstream causing damage to properties, building structures, storm drains, and vegetation, and possibly loss of life.

5.5.5 Maintenance of Storm Drain Infrastructure

Storm drains subject to sedimentation could fail in their function because of the following:

- decreased capacity to convey the stormwater they are designed to handle
- scour damage from the sediments in suspension
- pressure flows for which they are not designed
- “piping of soil” around the joints (which can lead to sink holes)

For more information on the maintenance of storm drain infrastructure, please refer to the *San Diego County Hydraulic Design Manual*.

5.6 SEDIMENT BULKING

5.6.1 Introduction

In 2020, the Hydrology Manual Technical Advisory Committee (TAC) formed a subcommittee tasked with providing guidance on estimating sediment and debris yields, determining when and where sediment or debris bulking factors should be applied, and developing appropriate methods for estimating post-fire increases in clear-water runoff. These considerations are relevant under both normal watershed conditions and during post-fire emergencies.

The need for updated guidance reflects changes in climate, land use, and the frequency and severity of wildfires. In recent years, several public agencies in Southern California have re-evaluated their approaches to sediment and debris hazard mitigation in light of these evolving conditions. Historically, mitigation efforts in the region have often followed damaging events. However, there is a growing recognition of the value of proactive planning to protect life, property, and infrastructure.

A final report titled *San Diego County Hydrology Manual Technical Studies: Bulking Factor Study*, dated October 2022, was prepared by River Focus in collaboration with RICK for the County of San Diego Department of Public Works Flood Control Section. This “Bulking Factor Study” is available for download from the County of San Diego Department of Public Works Flood Control Section website, along with this manual.

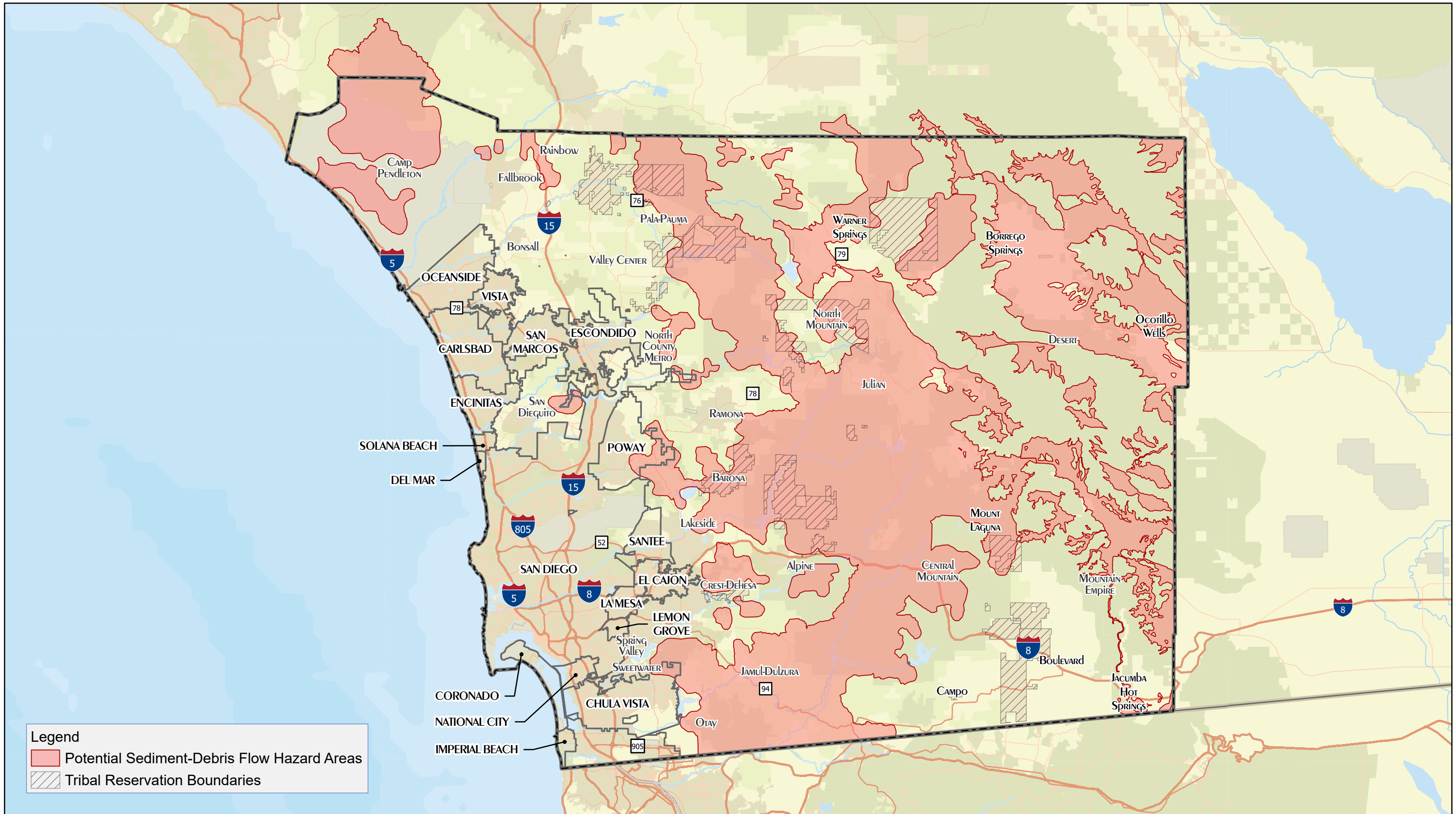
The Bulking Factor Study draws from the practices and experiences of other counties in Southern California, taking into account regional differences in watershed characteristics and available data. It incorporates current best practices and recent research developments, with particular attention to methodologies that are suited to the unique conditions of San Diego County.

Manual users can reference and use the Bulking Factor Study for design guidance on Sediment Bulking under both normal design and post-fire design conditions.

5.6.2 Guidance for Normal Design Conditions

Under normal design conditions, when a new project is located within or immediately downstream of an identified high-risk sediment or debris hazard area (see Figure 5-8), the planned infrastructure may be exposed to such hazards. Engineers should assess whether the planned infrastructure is at risk and confirm their finding with the County. If a risk is confirmed, the County may require the project to implement measures to mitigate potential sediment or debris impacts. Potential mitigation options are presented in Figure 5-9.

An example of a sediment bulking problem along with a step-by-step solution, is provided in the Workbook Section, WB.4.3.

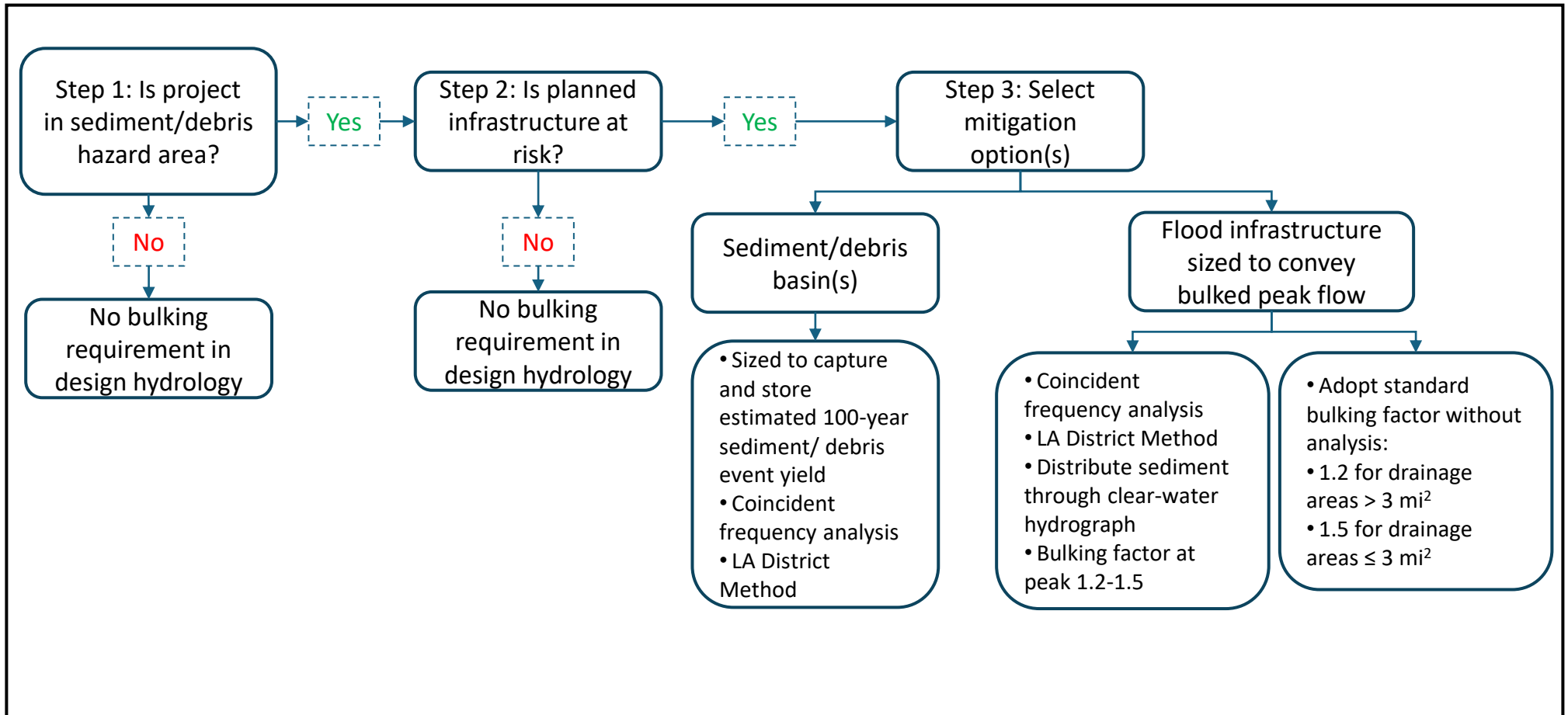


Legend
 ■ Potential Sediment-Debris Flow Hazard Areas
 ▨ Tribal Reservation Boundaries

POTENTIAL SEDIMENT / DEBRIS FLOW HAZARD AREA

FIGURE
5-8

Refer to Section 5 of the 2022 Bulking Factor Study Report at <https://www.sandiegocounty.gov/content/sdc/dpw/flood/hydrologymanual.html> for more information on how the sediment/debris hazard areas were developed.



Flow Chart for Sediment Bulking Under Normal Conditions

FIGURE

5-9

5.6.3 Guidance for Post-Fire Conditions

For design guidance under emergency post-fire conditions and for conditions two to ten years after a fire, refer to the *San Diego County Hydrology Manual Technical Studies: Bulking Factor Study*, dated October 2022. This Bulking Factor Study is available for download from the County of San Diego Department of Public Works Flood Control Section website, along with this manual.

5.6.4 Supplemental References

San Diego County Hydrology Manual Technical Studies: Bulking Factor Study, River Focus, 2022

SECTION 6

RATIONAL METHOD HYDROGRAPH PROCEDURE

6.1 INTRODUCTION

The procedure in this section is for the development of hydrographs from rational method study results for study areas up to approximately one square mile in size. The rational method, discussed in Section 3, is a mathematical formula used to estimate the maximum runoff rate from a given rainfall intensity. It has particular application in urban storm drainage, where it is used to estimate peak runoff rates from small urban and rural watersheds for the design of storm drains and small drainage structures. However, in some instances, such as for the design of detention basins, the single value peak runoff rate is not sufficient. For designs that are dependent on the total storm volume, a hydrograph must be generated to account for the entire volume of runoff from the full duration of the storm event. Unlike the NRCS hydrologic method (discussed in Section 4), the rational method itself does not create a hydrograph. This section provides a procedure that may be used for the development of a hydrograph from a rational method study. The resulting hydrograph may be used with other software such as HEC-HMS for routing purposes. These hydrographs are not compatible with and cannot be combined with or compared to hydrographs calculated using the NRCS methodology. They are typically intended for use in detention basin design for small watersheds using rational method studies for storm water conveyance system design. They are sometimes referred to as a small-area runoff hydrograph.

The County of San Diego has selected the 6-hour storm event duration as the basis for detention basin design for small watersheds using rational method studies for storm water conveyance system design. The rational method hydrograph procedure described in this section will create a hydrograph that will account for the entire volume of runoff from a project during the 6-hour storm event.

6.2 HYDROGRAPH DEVELOPMENT

The concept of the rational method hydrograph procedure is based on the rational method formula and the nested storm pattern. Points on the hydrograph are based on the rational method formula:

$$Q = C I A$$

Where:

Q = peak discharge, in cubic feet per second (cfs)

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

I = average rainfall intensity for a duration equal to the time of concentration, T_c , for the area (inches per hour)

A = drainage area contributing to the study point of interest (acres)

The total volume under the resulting hydrograph is equal to the following equation:

$$VOL = C P_6 A$$

Where:

V = volume of runoff, in acre-inches

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

P_6 = 6-hour storm event rainfall depth (inches)

A = drainage area contributing to the study point of interest (acres)

The shape of the rational method hydrograph is based on a 2/3, 1/3 nested storm pattern. The hydrograph for the entire 6-hour storm event is created by distributing the 6-hour storm event rainfall into incremental blocks of rainfall in a nested storm pattern similar to the nested hyetograph used for the NRCS hydrologic method studies (discussed in Section 4), converting each incremental rainfall into an intensity for the time increment, and

multiplying each intensity by the runoff coefficient and the drainage area to generate the flow rate for each point on the hydrograph. The resulting hydrograph has a volume equal to the 6-hour storm event runoff volume.

The following data are needed to create a rational method hydrograph:

- 5-, 10-, 15-, 30-, 60-, 120-, 180-, 240-, and 360-minute rainfall depths (P_5 , P_{10} , P_{15} , P_{30} , P_{60} , P_{120} , P_{180} , P_{240} , and P_{360}) (in inches) for the design storm event frequency (e.g., 100-year) from NOAA Atlas 14
- Study drainage area (acres)
- Study time of concentration (T_c) rounded to the nearest minute. T_c should be taken from the final subarea process that adds area to the rational method study. If the final process included in the rational method study is a pipe or channel travel process without addition of area, do not include the travel time from that process in the T_c for the rational method hydrograph.
- Composite C value for the study drainage area
- Rational method study peak flow rate (cfs)

The process to create the rational method hydrograph includes four steps: (1) prepare blocks of incremental rainfall for the 6-hour storm event duration based on multiples of T_c , (2) arrange the blocks of incremental rainfall in a 2/3, 1/3 distribution to create the rainfall hyetograph, (3) calculate the rainfall intensity for each incremental rainfall block, and (4) calculate the flow rate for each block to create the points on the hydrograph.

6.2.1 Prepare Incremental Rainfall Blocks

To prepare the incremental rainfall blocks for the 6-hour storm event duration, round the study T_c to the nearest minute and then take consecutive multiples of the rounded T_c until reaching 360 minutes (6 hours), or just under 360 minutes to within $\frac{1}{2}T_c$ of 360 minutes.

The number of rainfall blocks (N) is equal to 360 divided by the rounded T_c and rounded down to the nearest whole number.

$$N = 360 / T_c$$

Where:

N = Number of rainfall blocks

T_c = Rational method study time of concentration rounded to the nearest minute (minutes)

For example, if T_c is 7.2 minutes, round T_c to 7 minutes, and the number of rainfall blocks (N) will be 51 rainfall blocks.

Calculate the total duration for each rainfall block. The total duration of each rainfall block from 1 to N, $T_{T(N)}$, is equal to NT_c .

$$T_{T(N)} = N T_c$$

Where:

$T_{T(N)}$ = Total duration of rainfall block N (minutes)

N = Number of rainfall blocks

T_c = Rational method study time of concentration rounded to the nearest minute (minutes)

For example, if T_c is 7 minutes, the total durations from 1 to N will be 7 minutes at $T_{T(1)}$, 14 minutes at $T_{T(2)}$, 21 minutes at $T_{T(3)}$, and so on until 357 minutes at $T_{T(51)}$.

For each rainfall block from 1 to N, determine the total, or cumulative, rainfall depth $P_{T(N)}$ for the total duration $T_{T(N)}$. The rainfall depth is obtained from NOAA Atlas 14 precipitation depth data. Use log-log interpolation to determine the depth when $T_{T(N)}$ is between durations provided in NOAA Atlas 14 (see Section 3 and the Hydrology Manual Workbook for an equation and examples of log-log interpolation).

$P_{T(N)}$ = Total NOAA Atlas 14 rainfall depth for rainfall block N for the duration $T_{T(N)}$
(inches)

Finally, calculate the incremental rainfall depth $P_{(N)}$ for each rainfall block from 1 to N. The incremental rainfall depth $P_{(N)}$ is the incremental increase in total rainfall that occurs during the time step, and is calculated by taking the difference between the total rainfall at the current time $T_{T(N)}$ and the previous time $T_{T(N-1)}$. When $N = 1$, $P_{(N)} = P_{T(N)}$. Subsequently,

$$P_{(N)} = P_{T(N)} - P_{T(N-1)}$$

Where:

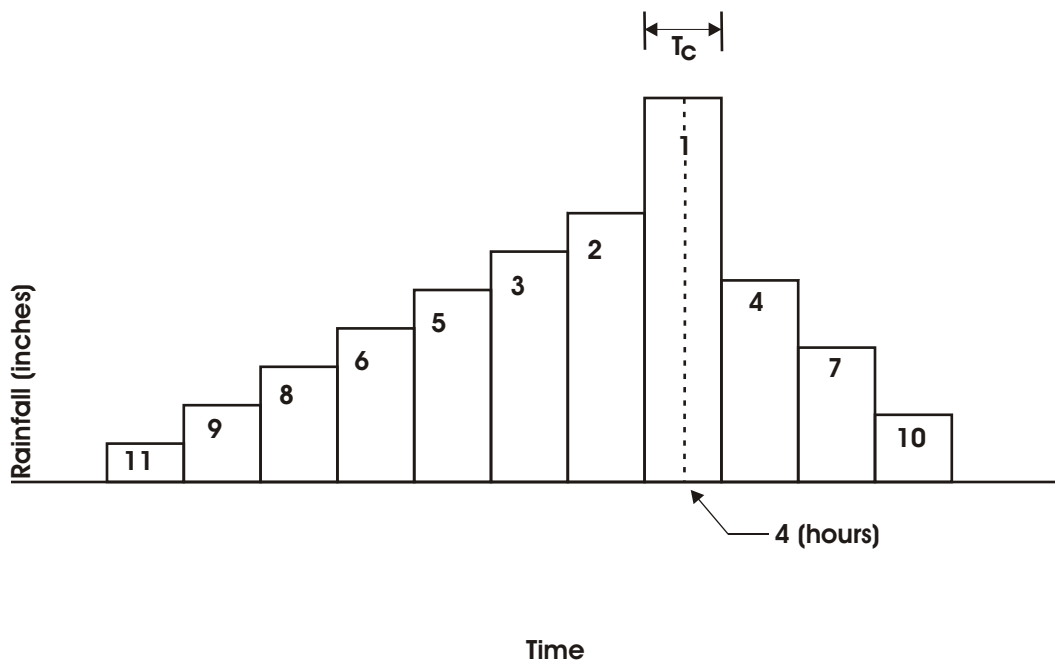
$P_{(N)}$ = Incremental rainfall depth for rainfall block N (inches)

N = Number of rainfall blocks

$P_{T(N)}$ = Total NOAA Atlas 14 rainfall depth for rainfall block N for its associated $T_{T(N)}$ (inches)

$P_{T(N-1)}$ = Total NOAA Atlas 14 rainfall depth for rainfall block N-1 for its associated $T_{T(N-1)}$ (inches)

The sum of the values of $P_{(N)}$ from the blocks 1 to N will be equal to the 6-hour rainfall depth, P_{360} . For an example of this process, see Table WB.5-1 in the Workbook Section of this manual.



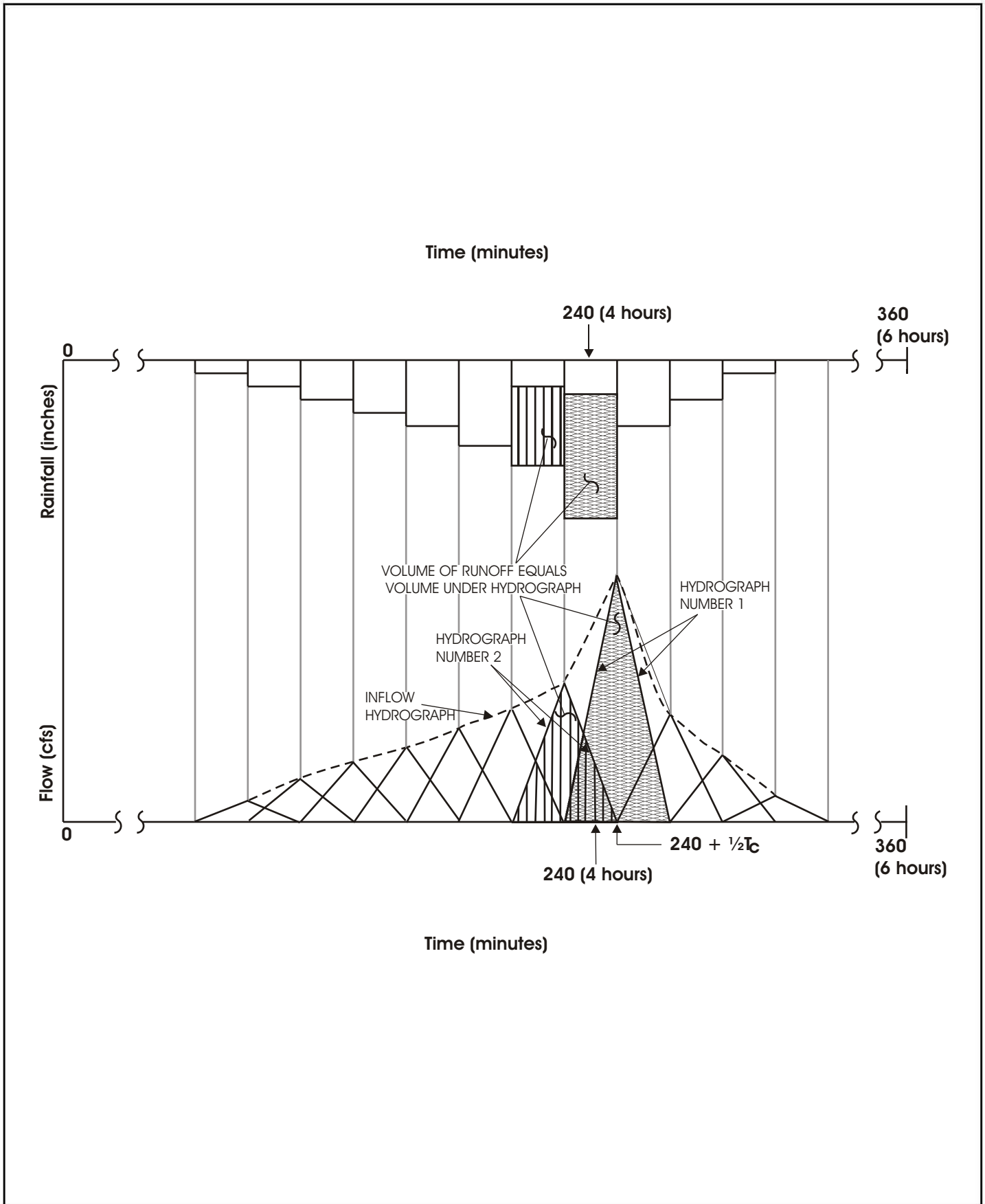
Rainfall Distribution

FIGURE

6-1

6.2.2 Arrange Rainfall Blocks in 2/3, 1/3 Distribution

Arrange the rainfall blocks in a 2/3, 1/3 distribution similar to the nested storm hyetograph used for the NRCS hydrologic method studies (discussed in Section 4). Figure 6-1 demonstrates the distribution of the rainfall blocks based on the block numbers (N). The blocks are distributed using a 2/3, 1/3 distribution in which the peak rainfall block is placed at the 4-hour time within the 6-hour storm duration. The rainfall for block N = 1 is centered at 4 hours (240 minutes). The rainfall for block N = 2, ($2T_c$), is centered at 4 hours - $1T_c$, the rainfall for block N = 3, ($3T_c$), is centered at 4 hours - $2T_c$, and the rainfall for block N = 4, ($4T_c$), is centered at 4 hours + $1T_c$. T_c is the study T_c rounded to the nearest minute in all cases. The sequence continues alternating two blocks to the left and one block to the right, as shown in Figure 6-1. Note that with block N = 1 being centered at 4 hours (240 minutes), the start of that rainfall block occurs at 4 hours (240 minutes) - $\frac{1}{2}T_c$ and the end of that rainfall block occurs at 4 hours (240 minutes) + $\frac{1}{2}T_c$. As discussed in more detail and shown graphically in Section 6.2.4, this results in the overall hydrograph peak flow rate occurring at 4 hours (240 minutes) + $\frac{1}{2}T_c$.



6-Hour Rational Method Hydrograph

FIGURE
6-2

6.2.3 Calculate Intensity for Each Rainfall Block

Calculate the rainfall intensity, $I_{(N)}$, for each incremental rainfall depth $P_{(N)}$. $I_{(N)}$ is equal to the incremental rainfall depth, $P_{(N)}$, determined for the rainfall block divided by the incremental duration of the rainfall block. All rainfall blocks from 1 to N have the same incremental duration, T_c .

$$I_{(N)} = (P_{(N)} / T_c) (60 \text{ minutes} / \text{hour})$$

Where:

$I_{(N)}$ = Rainfall intensity for the incremental rainfall depth $P_{(N)}$ for rainfall block N (inches per hour)

N = Number of rainfall blocks

$P_{(N)}$ = Incremental rainfall depth for rainfall block N (inches)

T_c = Rational method study time of concentration rounded to the nearest minute, (minutes)

6.2.4 Calculate the Peak Flow Rate for Each Rainfall Block

Calculate the peak flow rate, $Q_{(N)}$, for each rainfall block. The peak flow rate is calculated using the intensity for each rainfall block, the total study drainage area, and the composite C value for the study drainage area in the rational method equation.

$$Q_{(N)} = C I_{(N)} A$$

Where:

$Q_{(N)}$ = Peak flow rate for rainfall block N (cfs)

N = Number of rainfall blocks

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

$I_{(N)}$ = Rainfall intensity for the incremental rainfall depth $P_{(N)}$ for rainfall block N (inches per hour)

A = drainage area contributing to the study point of interest (acres)

Substituting the equation for intensity of each rainfall block in the equation above yields:

$$Q_{(N)} = C A (P_{(N)} / T_c) (60 \text{ minutes/hour})$$

Where:

$Q_{(N)}$ = Peak flow rate for rainfall block N (cfs)

N = Number of rainfall blocks

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

A = drainage area contributing to the study point of interest (acres)

$P_{(N)}$ = Incremental rainfall depth for rainfall block N (inches)

T_c = Rational method study time of concentration rounded to the nearest minute, (minutes)

The calculated peak flow rates for the rainfall blocks are the points that make up the rational method hydrograph. The individual hydrograph for each block of rain is a triangular hydrograph that starts at zero at the start of the rainfall block, increases linearly over the duration of the rainfall block (T_c) until reaching the peak discharge for the rainfall block ($Q_{(N)}$), and then decreases linearly back to zero. Since the rainfall for block N = 1 is centered at 4 hours (240 minutes), the peak flow rate $Q_{(1)}$ for block N = 1 occurs at 4 hours plus $\frac{1}{2}T_c$. Figure 6-2 shows the relationship of the distributed rainfall blocks to the individual hydrographs for the blocks and the overall hydrograph for the storm event. Since the peak flow amount for each incremental hydrograph corresponds to a zero-flow amount from the previous and proceeding hydrographs, as shown in Figure 6-2, the overall inflow hydrograph for the storm event can be plotted by connecting the peak flow amounts (see the dashed line in Figure 6-2).

When the rainfall blocks are presented in tabular format (i.e., one point in time associated with the rainfall block), the rainfall block for N = 1 is shown at 4 hours (240 minutes) + $\frac{1}{2}T_c$, which is the end of the rainfall block, and this time is referred to as the hydrograph peak flow rate time (T_p). The rainfall block for N = 2 is shown at $T_p - T_c$, the rainfall block

for $N = 3$ is shown at $T_p - 2T_c$, the rainfall block for $N = 4$ is shown at $T_p + T_c$, and so forth. For an example of this process for an entire storm event, see Table WB.5-2 in the Workbook Section of this manual.

Due to rounding, the T_c to the nearest whole minute at the start of the rational method hydrograph procedure and/or due to confluence analyses within the rational method study, the calculated $Q_{(1)}$ for block $N = 1$ may be slightly different from the study peak flow rate. The actual study peak flow rate should be substituted for the calculated $Q_{(1)}$ for block $N = 1$ for consistency with the rational method study.

In some cases, due to confluences within the study system the actual study peak flow rate may be lower than the calculated $Q_{(2)}$ for block $N = 2$. If the actual study peak flow rate is lower than $Q_{(2)}$ for block $N = 2$; first check that the T_c is taken from the last point that area was added to the study rather than a pipe or channel travel process that extends T_c without addition of area, and check the composite runoff coefficient, C , for the study. If the T_c and C value that were used in the rational method hydrograph calculations are determined to be correct and the calculated $Q_{(2)}$ is higher than the study peak flow rate, the study peak flow rate may be substituted for the calculated $Q_{(2)}$ for block $N = 2$.

6.3 Generating a Rational Method Hydrograph Using the SDHydroTools Computer Program

The rational method hydrograph can be developed using the SDHydroTools computer program provided with this manual. A copy of the SDHydroTools computer program is available at no cost from the County. The rational method hydrograph output from the SDHydroTools computer program may be used with other software such as HEC-HMS for routing purposes.

The rational method hydrograph is prepared by SDHydroTools as described in Sections 6.2.1 through 6.2.4. The user provides the study T_c (rounded to the nearest minute), study drainage area (acres), weighted runoff coefficient, C , for the study drainage area (no units), and the study peak flow rate from the rational method calculations (cfs). The program

calculates the rainfall blocks and arranges the rainfall blocks in the 2/3, 1/3 distribution, then calculates the intensity and flow rate for each block. The program will use the study peak flow rate provided by the user as the flow rate for block $N = 1$ for consistency with the rational method study. If the user-provided peak flow rate is different from the calculated $Q_{(1)}$, the program may adjust $Q_{(2)}$ to correct the hydrograph volume (i.e., to adjust the volume to equal $C P_6 A$).

SECTION 7 WATER QUALITY CONSIDERATIONS

Stormwater quality has increasingly become an integral aspect of stormwater management. The reader is advised to consult the [County's BMP Design Manual](#) for current information regarding implementation of water quality requirements to meet the current regulations. Refer to the County of San Diego Department of Public Works Watershed Protection Program's stormwater regulations webpage for the latest BMP Design Manual.

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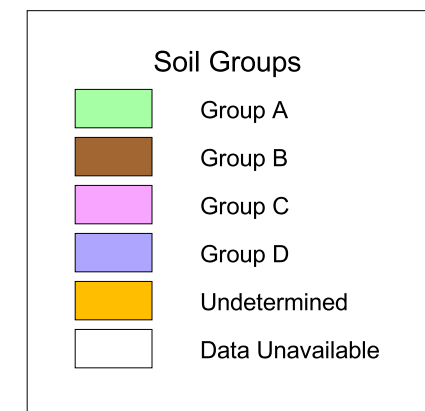
APPENDIX A
HYDROLOGIC SOIL GROUPS MAP

County of San Diego Hydrology Manual

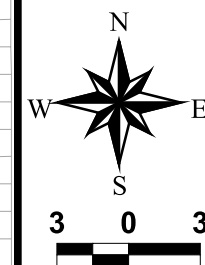


Soil Hydrologic Groups

Legend



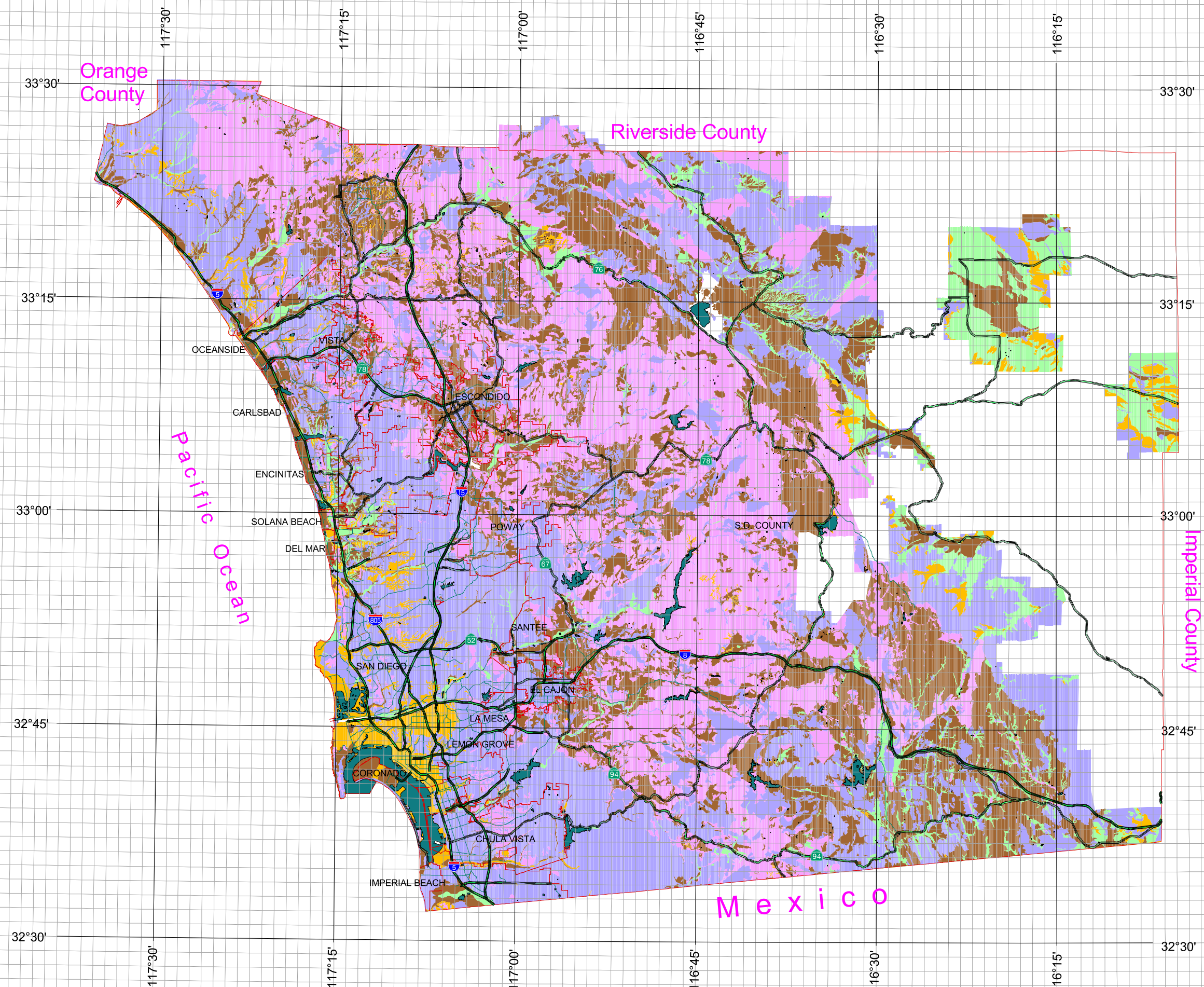
Note:
For the latest soils data, use NRCS [Web Soil Survey](#).



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APPENDIX B

GUIDANCE TO OBTAIN RAINFALL DATA FOR HYDROLOGIC STUDIES

APPENDIX B Guidance to Obtain Rainfall Data for Hydrologic Studies

Rainfall Data for New Project Studies

The County of San Diego has adopted *NOAA Atlas 14 Precipitation-Frequency Atlas of the United States Volume 6* (herein “NOAA Atlas 14”) as the source to be used to obtain rainfall data for new hydrologic studies for project design. NOAA stands for the National Oceanic and Atmospheric Administration. Volume 6 of NOAA Atlas 14 contains precipitation frequency estimates for the state of California, published through a Precipitation Frequency Data Server (PFDS) (<http://hdsc.nws.noaa.gov/hdsc/pfds>). NOAA Atlas 14 documentation can be obtained from NOAA’s National Weather Service (NWS) Hydrometeorological Design Studies Center (HSDC) at <https://www.nws.noaa.gov/oh/hdsc/currentpf.html>. The current release of Volume 6 of NOAA Atlas 14 as of October 2020 is Version 2.3 dated 2014. Based on NOAA Atlas 14 documentation, when new information is completed and added to NOAA Atlas 14 without changing any prior information, the primary version number (i.e., the number 2 in Version 2.3) is not incremented. Therefore, the County has adopted Version 2 and successive releases of Version 2.

With the adoption of NOAA Atlas 14 as the source for rainfall data, the County’s isopluvial maps that were provided in Appendix B of the June 2003 version of this manual and the County’s intensity-duration design chart and corresponding intensity equation $I = 7.44P_6D^{-0.645}$ provided in Figure 3-1 of the June 2003 version of this manual have been retired. New computation procedures described in Sections 3, 4, and 6 of this manual supersede the use of the isopluvial maps, intensity-duration design chart, and intensity equation. This Appendix provides guidance to obtain and document NOAA Atlas 14 rainfall data. Sections 3, 4, and 6 of this manual provide procedures for using the NOAA Atlas 14 rainfall data in hydrologic computations.

Obtaining and Documenting NOAA Atlas 14 Rainfall Data

Rainfall data may be obtained from NOAA’s Precipitation Frequency Data Server (PFDS) at <https://hdsc.nws.noaa.gov/hdsc/pfds/>, which is a point-and-click interface developed to deliver NOAA Atlas 14 precipitation frequency estimates and associated information. Figure B-1 provides a snapshot of the PFDS website (current as of October 2020). Rainfall data for selected durations from 5 minutes to 24 hours for the 100-year storm event within San Diego County only may also be obtained from the SDHydroTools computer program provided with this manual, which incorporated a copy of the 100-year data set from NOAA Atlas 14 Volume 6 in a GIS compatible format.

For studies of relatively small watersheds that will have consistent rainfall throughout the watershed (e.g., most rational method studies), rainfall data may be collected at a single point within the study area. For studies of larger watersheds in which rainfall depths may vary throughout the watershed (e.g., some NRCS unit hydrograph studies), data should be collected at locations selected to represent the variability of the rainfall across subbasins within the study watershed.

NOAA's National Weather Service
Hydrometeorological Design Studies Center
Precipitation Frequency Data Server (PFDS)



Home Site Map News Organization Search NWS All NOAA

General Information

- Homepage
- Progress Reports
- FAQ
- Glossary

Precipitation Frequency

- Data Server
- GIS Grids
- Maps
- Time Series
- Temporals
- Documents

Probable Maximum Precipitation

- Documents

Miscellaneous

- Publications
- Storm Analysis
- Record Precipitation

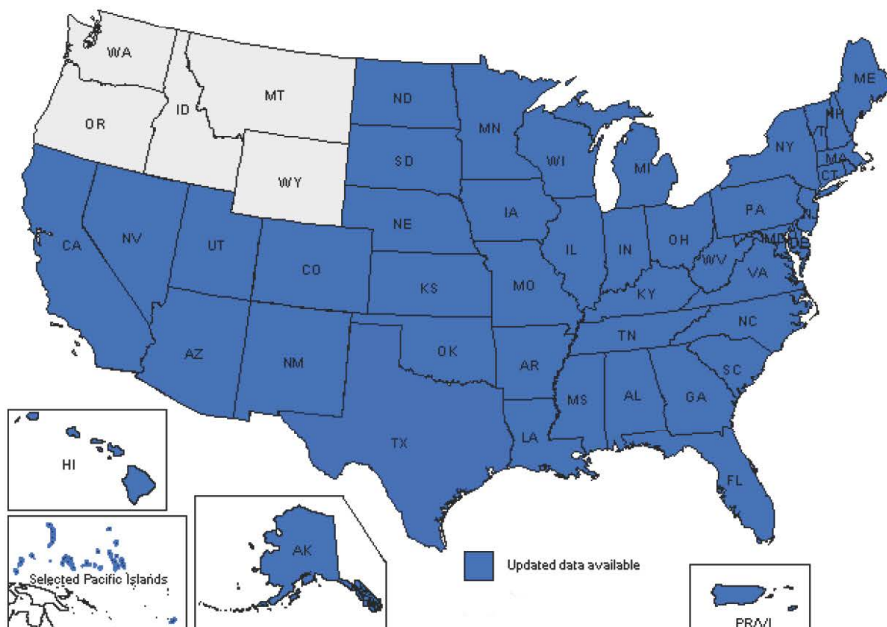
Contact Us

- Inquiries



Precipitation Frequency Data Server (PFDS)

State:



The Precipitation Frequency Data Server (PFDS) is a point-and-click interface developed to deliver NOAA Atlas 14 precipitation frequency estimates and associated information. Upon clicking a state on the map above or selecting a state name from the drop-down menu, an interactive map of that state will be displayed. From there, a user can identify a location for which precipitation frequency estimates are needed.

Estimates and their confidence intervals can be displayed directly as tables or graphs via separate tabs. Links to supplementary information (such as ASCII grids of estimates, associated temporal distributions of heavy rainfall, time series data at observation sites, cartographic maps, etc.) can also be found.

Section 5 of the NOAA Atlas 14 documents provides additional information on the underlying data and functioning of the PFDS.

*PFDS is compatible with all modern web browsers. However, some browsers offer a smoother experience than others. We recommend Chrome, Firefox, Internet Explorer 11+, and Safari. *Microsoft's Edge is NOT Compatible**

Main Link Categories:
[Home](#) | [OWP](#)

US Department of Commerce
 National Oceanic and Atmospheric Administration
 National Weather Service
 Office of Water Prediction (OWP)
 1325 East West Highway
 Silver Spring, MD 20910

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Figure B-1: The Precipitation Frequency Data Server (PFDS) Website

The following workflow is provided to guide the user to obtain data from the PFDS and document the data for a study.

1. Open the PFDS website and select California.
2. In the “Data description” area, select the data type, units, and time series type. The data type depends on the intended use of the data. Select precipitation depth or precipitation intensity as appropriate to the calculation method and tools to be used for the hydrologic study. Select English units and select partial duration as the time series type (Figure B-2).
3. Select the location to obtain the data according to the instructions provided on the PFDS (Figure B-2).
4. Print the data to store in a project file and provide in the study report (Figure B-3). Latitude and longitude must be recorded to ensure reproducible results.

The screenshot shows the NOAA PFDS website interface. The main heading is "NOAA ATLAS 14 POINT PRECIPITATION FREQUENCY ESTIMATES: CA". The page is divided into several sections:

- Data description:** Includes dropdown menus for "Data type" (set to "Precipitation depth"), "Units" (set to "English"), and "Time series type" (set to "Partial duration").
- Select location:** Offers three methods:
 - Manually:
 - a) By location (decimal degrees, use "°" for S and W): Includes input fields for Latitude and Longitude, and a "Submit" button.
 - b) By station (list of CA stations): Includes a "Select station" dropdown menu.
 - c) By address: Includes an input field with "5510 Overland Ave, San Diego, CA, US" and a search icon.
 - Use map: Includes a note about ESRI interactive map loading and a link to <https://js.arcgis.com/>.
- Map:** A topographic map of San Diego with a red crosshair indicating the selected location. A sidebar on the right of the map includes:
 - a) Select location: "Move crosshair or double click"
 - b) Click on station icon: Includes a checkbox for "Show stations on map".
 - Location information:
 - Name: San Diego, California, USA*
 - Latitude: 32.8335°
 - Longitude: -117.1296°
 - Elevation: 430.57 ft **

Three red boxes with arrows point to these key features:

- Data Description:** (set to precipitation intensity, English, and partial duration for this example)
- Location Selection:** (manually selected by address for this example)
- Latitude and Longitude Coordinates:**

Figure B-2: Example of Data Description and Location Selection in the PFDS Website



NOAA Atlas 14, Volume 6, Version 2
 Location name: San Diego, California, USA*
 Latitude: 32.8335°, Longitude: -117.1296°
 Elevation: 430.57 ft**
 * source: ESRI Maps
 ** source: USGS



POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Sarah Dietz, Sarah Heim, Lillian Hiner, Kazungu Maitaria, Deborah Martin, Sandra Pavlovic, Ishani Roy, Carl Trypaluk, Dale Unruh, Fenglin Yan, Michael Yekta, Tan Zhao, Geoffrey Bonnin, Daniel Brewer, Li-Chuan Chen, Tye Parzybok, John Yarchoan

NOAA, National Weather Service, Silver Spring, Maryland

[PF tabular](#) | [PF graphical](#) | [Maps & aeriels](#)

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches/hour) ¹										
Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	1.44 (1.21-1.74)	1.81 (1.52-2.18)	2.29 (1.92-2.77)	2.69 (2.22-3.28)	3.20 (2.56-4.04)	3.60 (2.81-4.64)	3.98 (3.04-5.29)	4.39 (3.25-5.99)	4.92 (3.49-7.02)	5.33 (3.65-7.88)
10-min	1.03 (0.870-1.24)	1.30 (1.09-1.57)	1.64 (1.37-1.99)	1.92 (1.59-2.35)	2.29 (1.84-2.90)	2.57 (2.02-3.33)	2.86 (2.18-3.79)	3.14 (2.33-4.30)	3.53 (2.50-5.03)	3.82 (2.61-5.65)
15-min	0.832 (0.700-1.00)	1.05 (0.880-1.26)	1.33 (1.11-1.60)	1.55 (1.28-1.89)	1.85 (1.48-2.34)	2.08 (1.62-2.68)	2.30 (1.76-3.06)	2.54 (1.88-3.46)	2.84 (2.02-4.06)	3.08 (2.10-4.55)
30-min	0.572 (0.480-0.688)	0.720 (0.602-0.868)	0.910 (0.760-1.10)	1.06 (0.880-1.30)	1.27 (1.01-1.60)	1.42 (1.11-1.84)	1.58 (1.20-2.09)	1.74 (1.29-2.37)	1.95 (1.38-2.78)	2.11 (1.44-3.12)
60-min	0.405 (0.339-0.487)	0.509 (0.426-0.614)	0.644 (0.538-0.778)	0.752 (0.623-0.918)	0.898 (0.718-1.13)	1.01 (0.788-1.30)	1.12 (0.853-1.48)	1.23 (0.911-1.68)	1.38 (0.979-1.97)	1.50 (1.02-2.21)
2-hr	0.276 (0.231-0.332)	0.346 (0.290-0.418)	0.438 (0.366-0.530)	0.512 (0.424-0.624)	0.610 (0.488-0.770)	0.684 (0.536-0.884)	0.759 (0.578-1.01)	0.835 (0.618-1.14)	0.936 (0.663-1.33)	1.01 (0.692-1.50)
3-hr	0.219 (0.184-0.264)	0.276 (0.231-0.333)	0.349 (0.292-0.422)	0.408 (0.338-0.497)	0.486 (0.389-0.614)	0.545 (0.426-0.704)	0.604 (0.461-0.801)	0.664 (0.492-0.907)	0.744 (0.527-1.06)	0.805 (0.550-1.19)
6-hr	0.152 (0.127-0.183)	0.191 (0.160-0.230)	0.241 (0.202-0.292)	0.282 (0.233-0.344)	0.336 (0.268-0.424)	0.376 (0.294-0.486)	0.417 (0.318-0.552)	0.458 (0.339-0.625)	0.512 (0.363-0.730)	0.553 (0.378-0.817)
12-hr	0.101 (0.085-0.122)	0.128 (0.107-0.155)	0.162 (0.136-0.196)	0.190 (0.157-0.231)	0.226 (0.180-0.285)	0.253 (0.198-0.326)	0.279 (0.213-0.370)	0.307 (0.227-0.418)	0.342 (0.242-0.488)	0.369 (0.252-0.546)
24-hr	0.062 (0.054-0.072)	0.078 (0.069-0.091)	0.100 (0.087-0.116)	0.116 (0.101-0.137)	0.138 (0.117-0.168)	0.155 (0.128-0.191)	0.171 (0.138-0.216)	0.187 (0.148-0.243)	0.209 (0.158-0.281)	0.225 (0.165-0.313)
2-day	0.036 (0.032-0.042)	0.046 (0.041-0.054)	0.059 (0.052-0.069)	0.070 (0.061-0.082)	0.083 (0.070-0.101)	0.093 (0.077-0.115)	0.103 (0.083-0.130)	0.113 (0.089-0.146)	0.126 (0.096-0.170)	0.136 (0.100-0.189)
3-day	0.027 (0.024-0.031)	0.035 (0.031-0.041)	0.045 (0.039-0.052)	0.053 (0.046-0.062)	0.063 (0.053-0.077)	0.071 (0.059-0.088)	0.079 (0.064-0.099)	0.086 (0.068-0.112)	0.097 (0.073-0.130)	0.104 (0.076-0.145)
4-day	0.022 (0.020-0.026)	0.029 (0.025-0.034)	0.037 (0.033-0.044)	0.044 (0.038-0.052)	0.053 (0.045-0.064)	0.059 (0.049-0.073)	0.066 (0.053-0.083)	0.072 (0.057-0.094)	0.081 (0.061-0.109)	0.087 (0.064-0.121)
7-day	0.015 (0.013-0.018)	0.020 (0.017-0.023)	0.026 (0.023-0.030)	0.030 (0.026-0.036)	0.036 (0.031-0.044)	0.041 (0.034-0.051)	0.046 (0.037-0.058)	0.050 (0.039-0.065)	0.056 (0.042-0.075)	0.060 (0.044-0.084)
10-day	0.012 (0.010-0.014)	0.015 (0.014-0.018)	0.020 (0.018-0.023)	0.024 (0.021-0.028)	0.029 (0.024-0.035)	0.032 (0.027-0.040)	0.036 (0.029-0.045)	0.039 (0.031-0.051)	0.044 (0.033-0.059)	0.047 (0.035-0.066)
20-day	0.007 (0.006-0.008)	0.009 (0.008-0.011)	0.012 (0.011-0.014)	0.014 (0.013-0.017)	0.017 (0.015-0.021)	0.020 (0.016-0.024)	0.022 (0.018-0.028)	0.024 (0.019-0.031)	0.027 (0.021-0.036)	0.029 (0.022-0.041)
30-day	0.006 (0.005-0.007)	0.007 (0.007-0.009)	0.010 (0.009-0.011)	0.012 (0.010-0.014)	0.014 (0.012-0.017)	0.016 (0.013-0.020)	0.018 (0.014-0.023)	0.020 (0.015-0.025)	0.022 (0.017-0.030)	0.024 (0.018-0.033)
45-day	0.004 (0.004-0.005)	0.006 (0.005-0.007)	0.008 (0.007-0.009)	0.009 (0.008-0.011)	0.011 (0.009-0.013)	0.013 (0.010-0.016)	0.014 (0.011-0.018)	0.016 (0.012-0.020)	0.017 (0.013-0.023)	0.019 (0.014-0.026)
60-day	0.004 (0.003-0.004)	0.005 (0.004-0.006)	0.007 (0.006-0.008)	0.008 (0.007-0.009)	0.010 (0.008-0.012)	0.011 (0.009-0.014)	0.012 (0.010-0.016)	0.014 (0.011-0.018)	0.015 (0.012-0.021)	0.017 (0.012-0.023)

¹ Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS). Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values. Please refer to NOAA Atlas 14 document for more information.

[Back to Top](#)

PF graphical

Figure B-3: Example of Data Obtained from the PFDS

To obtain data from the SDHydroTools computer program,

1. Open the program and navigate to the “Location” window within the “Project” tab.
2. Select the location to obtain the data according to the instructions provided in the program (Figure B-4). A project location must be entered in the “Location” window to activate the “Rainfall Data” window.
3. Click “GET DATA” in the “Location” window.
4. The program will open the “Rainfall Data” window and provide 100-year data for the requested location. The data provided is precipitation depth in English units of inches. The time series type is partial duration. The 100-year data found by the program will be displayed in green (Figure B-5). From this point a user may enter other data to be used in the program computation modules if desired (e.g., data for a storm event other than the 100-year storm). User entered data will be displayed in red.
5. Save a screen capture of the “Location” window and the “Rainfall Data” window to store in a project file and provide in the study report. Latitude and longitude must be recorded to ensure reproducible results. The SDHydroTools computer program does not export rainfall data.

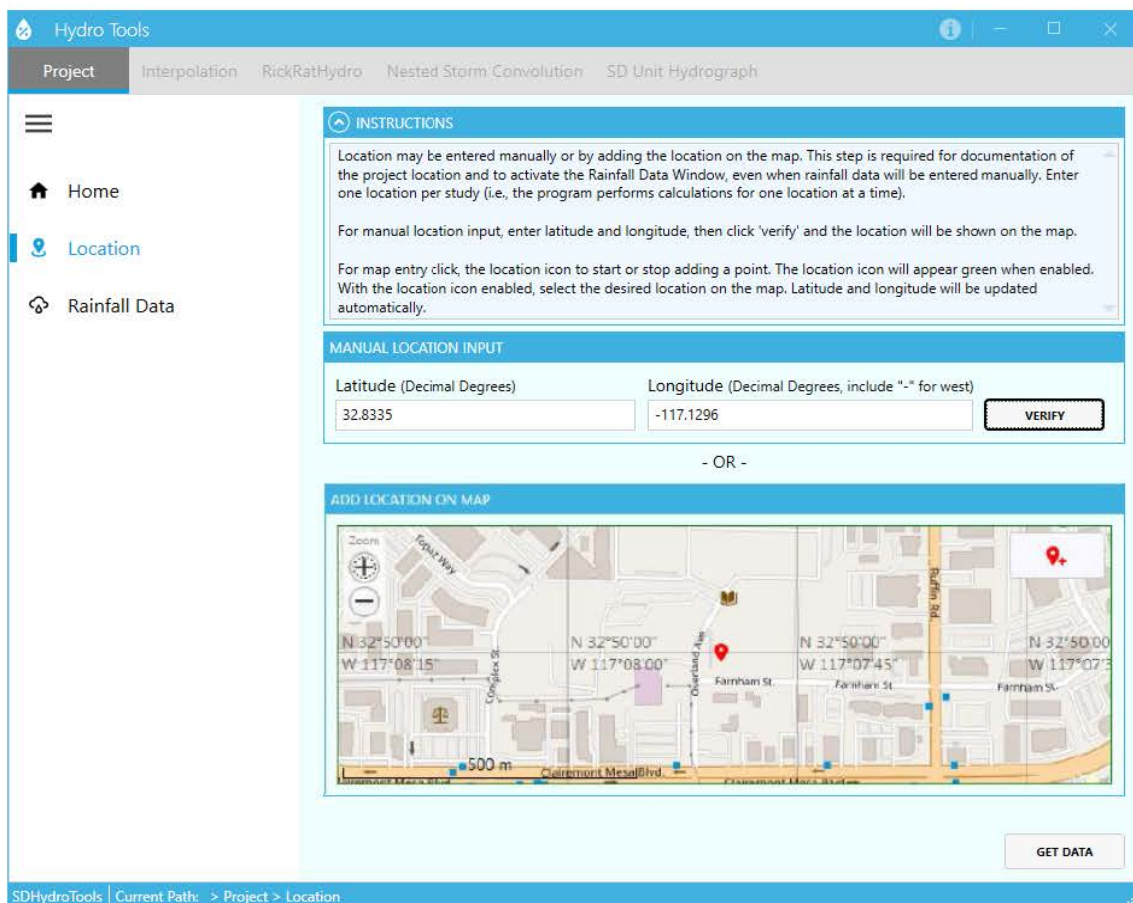


Figure B-4: SDHydroTools Location Window

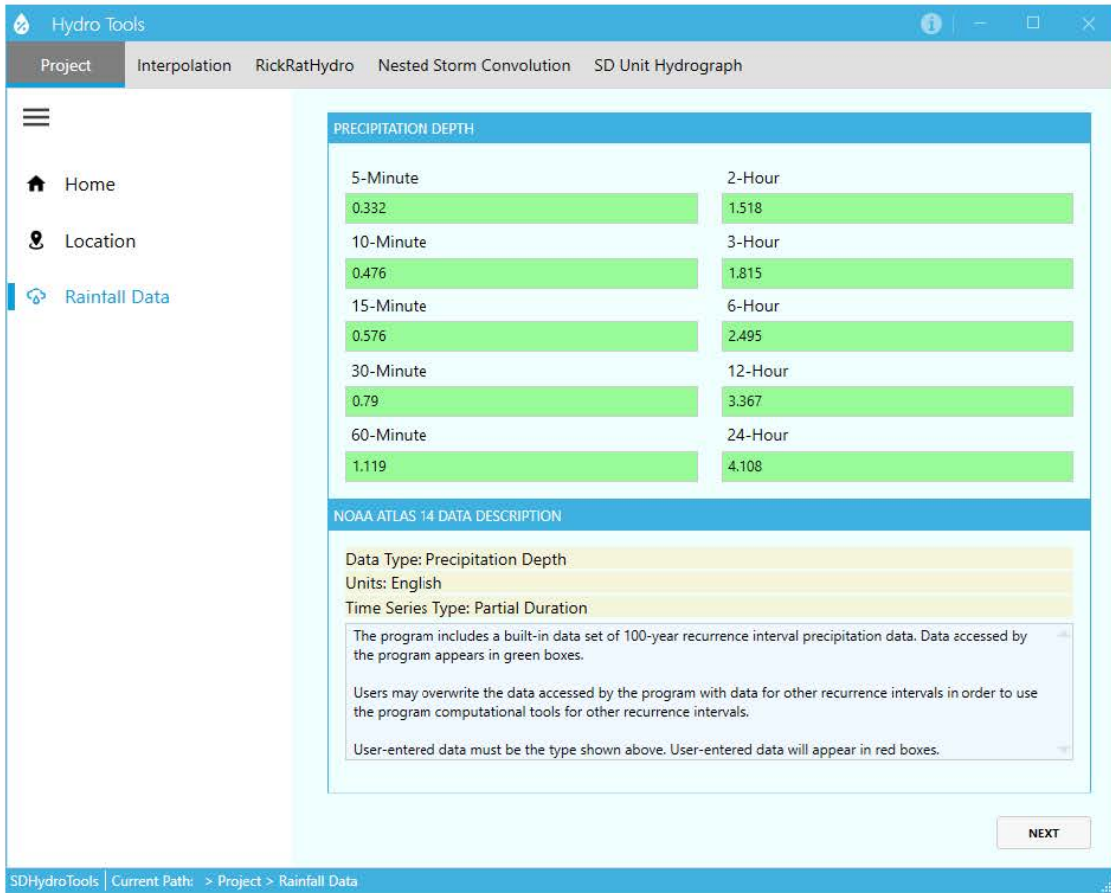


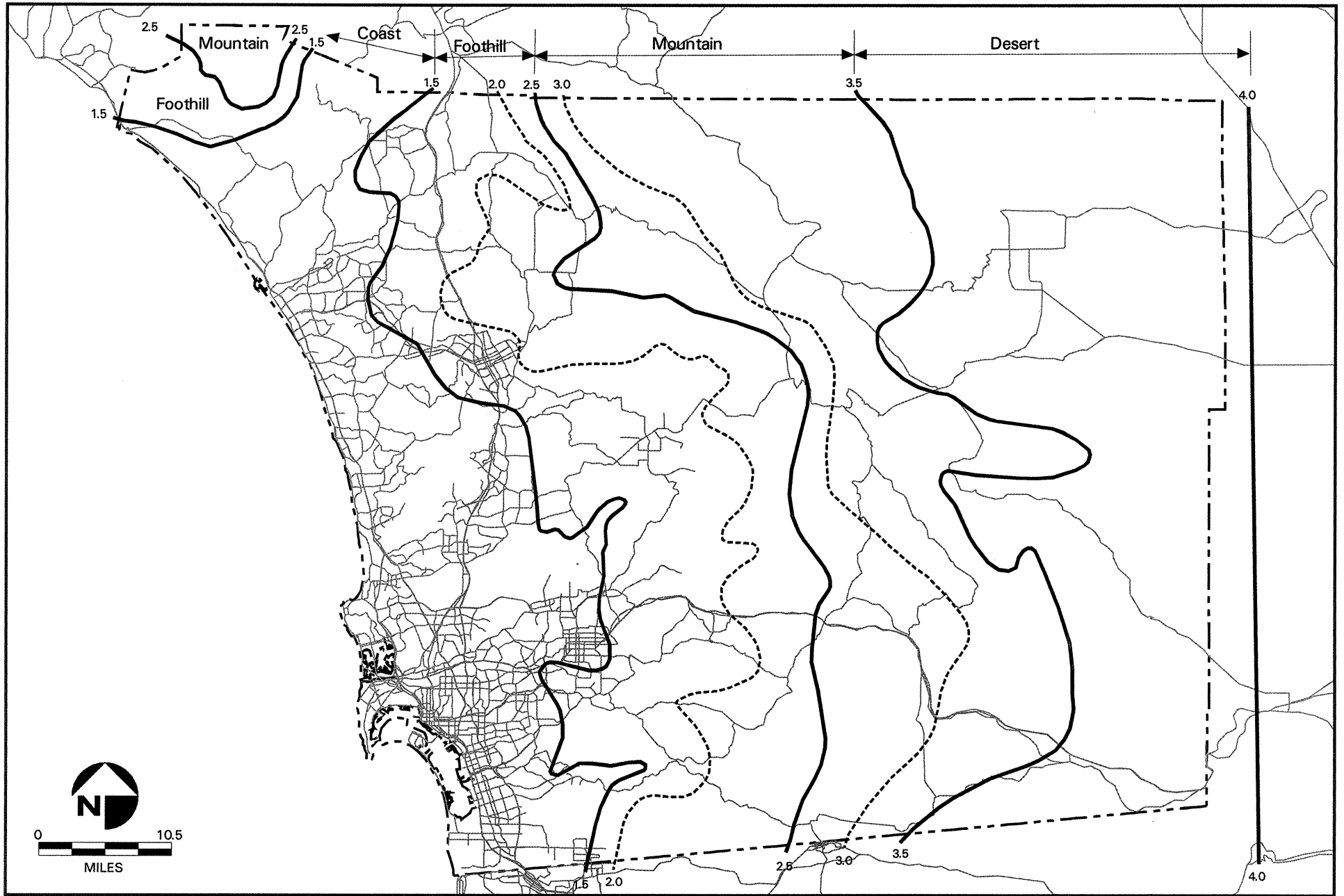
Figure B-5: SDHydroTools Rainfall Data Window

Use of Rainfall Data from Sources Other than NOAA Atlas 14

When it is necessary to use rainfall data from a source other than NOAA Atlas 14 for a study, such as for forensic study, the source of the rainfall data and reason for its use shall be documented. When feasible, latitude and longitude associated to point precipitation values shall be provided.

APPENDIX C

PRECIPITATION ZONE NUMBER (PZN) MAP



County of San Diego Hydrology Manual
Precipitation Zone Numbers (PZN)

FIGURE

C-1

APPENDIX D

**WORKSHEETS FOR NRCS HYDROLOGIC METHOD
CALCULATIONS**

WORKSHEET 4-1

Land Use Worksheet

(name of project)

DATE	BY	HYDROLOGIC CONDITION											
		GOOD 3				FAIR 2				POOR 1			
		A	B	C	D	A	B	C	D	A	B	C	D
FALLOW STRAIGHT ROW	CR												
ROW CROPS STRAIGHT ROW	CR					N/A							
ROW CROPS CONTOURED	CR					N/A							
SMALL GRAIN STRAIGHT ROW	CR					N/A							
SMALL GRAIN CONTOURED	CR					N/A							
CLOSE SEEDED STRAIGHT	CR					N/A							
CLOSE SEEDED CONTOURED	CR					N/A							
IRRIGATED PASTURE	IP												
WATER SURFACES (DURING FLOODS)	WA					N/A				N/A			
ORCHARDS EVERGREEN	OE												
ORCHARDS DECIDUOUS*	OD												
VINEYARDS	VY												
URBAN LOW DENSITY	DL		N/A								N/A		
URBAN MEDIUM DENSITY	DL		N/A								N/A		
URBAN HIGH DENSITY	DL		N/A								N/A		
COMMERCIAL INDUSTRIAL	DL		N/A								N/A		
ANNUAL GRASS	AG												
BROADLEAF CHAPARRAL	BC												
MEADOW	ME												
NARROWLEAF CHAPARRAL	NC		N/A										
OPEN BRUSH	OB												
PERENNIAL GRASS	PG												
WOODLAND GRASS	WG												
WOODS (WOODLAND)	WO												
BARREN	BA		N/A								N/A		
TURF	TU												
FARMSTEADS	FS		N/A								N/A		
ROADS (DIRT)	RD		N/A								N/A		
ROADS (HARD SURFACE)	RD		N/A								N/A		

*For deciduous orchards, select the CN that applies to the land use or the kind and condition of cover during storm periods (winter time). For example, select annual grass CN values for annual grass or grass legume cover. If orchards are kept bare by disking or through the use of herbicides, use fallow CNs.

WORKSHEET 4-3

Peak Discharge Computation

_____ (name of project)

*****For use with NRCS Hydrologic Method Computations*****

Items in boxes are required input parameters for the SDUH Peak Discharge Program.

Computed by: _____ Date: _____

Project Identification (Drainage Area Name):

Geographic location of center of drainage area: Long: _____ " Lat: _____ "

Drainage Area: – square miles

Storm Frequency (Section 2.3): – year

6-Hour Storm Duration Precipitation (Appendix B): – inches

24-Hour Storm Duration Precipitation (Appendix B): – inches

Precipitation Zone Number (PZN): PZN = 1.0 _____ 2.0 _____ 3.0 _____ 4.0
(Section 4.1.2.4 and Appendix C)

PZN Adjustment Factor for 5-year to 35-year storm frequency (interpolate): 1.5 _____ 2.5 _____ 2.0 _____ 1.5
(Section 4.1.2.4 and Table 4-6)

PZN Adjustment Factor for 35-year to 150-year storm frequency (interpolate): 2.0 _____ 3.0 _____ 3.0 _____ 2.0
(Section 4.1.2.4 and Table 4-6)

PZN Adjusted Runoff Curve Number (interpolate between nearest whole number PZN conditions): $CN_{1.0 \text{ or } 2.0}$ _____ CN_x $CN_{2.0 \text{ or } 3.0}$ _____
(Sections 4.1.2.4 and 4.2.4, Tables 4-6 and 4-10)

Watershed Length (L) (Section 4.3.1): _____ – miles

Length to Centroid (L_c) (Section 4.3.1): _____ – miles

Slope (s) (Section 4.3.1): _____ – feet/mile Basin \bar{n} Factor (Section 4.3.5): _____

Corps lag (T_L) = $24 \bar{n} ((L \times L_c)/s^{0.5})^m$ (Section 4.3.1.1)

OR

Corps lag (T_L) = $0.8 T_c$ (Section 4.3.1.2)

Lag Time: – hours

Time to Peak = $0.862 \times$ Corps lag (Section 4.1.5.5):

Time to Peak : _____ – hours

WORKBOOK INTRODUCTION

WB.1 PURPOSE

The purpose of this workbook is to provide example calculations demonstrating the concepts presented in Sections 3, 4, 5, and 6 of the Hydrology Manual. Section 3 of the Hydrology Manual presents the Rational Method (RM) and Modified Rational Method (MRM). Section 4 of the Hydrology Manual presents the Natural Resources Conservation Service (NRCS) unit hydrograph method. Section 5 of the Hydrology Manual presents concepts and procedures for evaluating the erosion potential of a site and sizing desiltation structures, or applying other erosion protection devices to a project. Section 6 of the Hydrology Manual presents the Rational Method Hydrograph procedure. Each example problem references the corresponding Sections of the Hydrology Manual. Figure numbers, Table numbers, and Section numbers that are not preceded by “WB.” indicate Figures, Tables, and Sections from the Hydrology Manual. Figure numbers, Table numbers and Section numbers that are preceded by “WB.” indicate Figures, Tables, and Sections within this workbook.

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WB.2 WORKBOOK EXAMPLES FOR HYDROLOGY MANUAL SECTION 3.0 RATIONAL METHOD AND MODIFIED RATIONAL METHOD

WB.2.1 Rational Method

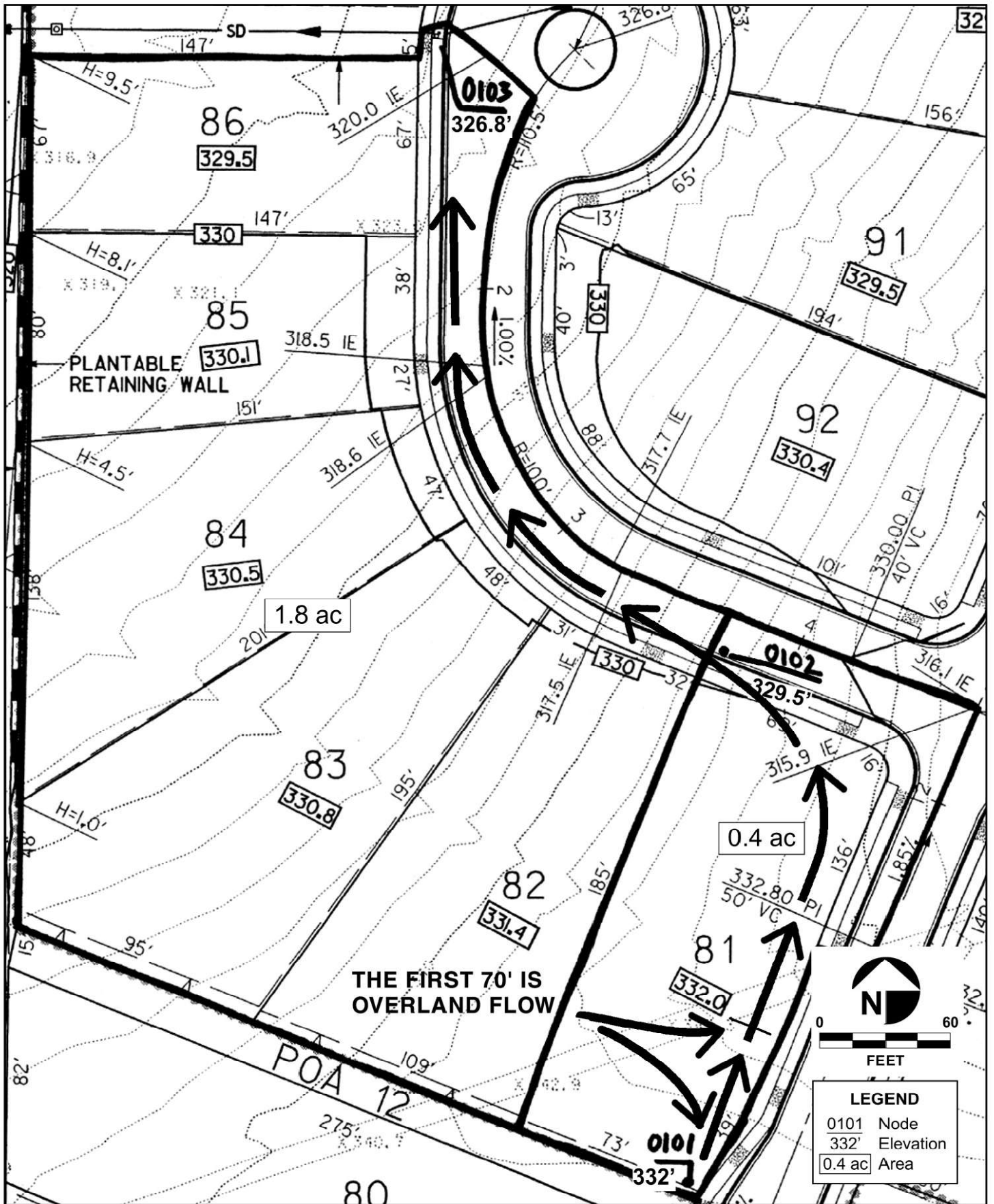
(Reference Hydrology Manual Section 3.3)

The following example details the application of the RM for a single-family residential subdivision to calculate the peak flow entering an inlet in the storm drain system. In this example, the 100-year storm event is used. In this example, the soil type (determined from the soils maps in Appendix A of the Hydrology Manual) is uniform across all subareas and is type D. Figure WB.2-1 shows the drainage map for this example. The following precipitation intensity data pairs obtained from NOAA Atlas 14 will be used in this example:

$I_{5\text{-min}} = 4.87$ inches per hour

$I_{10\text{-min}} = 3.49$ inches per hour

$I_{15\text{-min}} = 2.82$ inches per hour



Example Discharge Area - Rational Method

FIGURE
WB. 2-1

Flow across the initial subarea

First, consider the initial subarea, nodes 0101 to 0102 in Figure WB.2-1.

$C = 0.52$ (read from Table 3-1 of the Hydrology Manual for single-family residential, 4.3 dwelling units per acre [DU/A] or less, type D soil)

$A_{0101-0102} = 0.4$ acres

$\Sigma(CA) = 0.21$

$L = 220$ feet (estimated total flow length after development with house, driveway, garage, etc. Use 70 feet maximum per Table 3-2 of the Hydrology Manual to determine T_i).

$s = \frac{332' - 329.5'}{220'} = 0.011$ or 1.1% slope (typical value for graded residential lot)

To determine the time of concentration, first use a 70' maximum length per Table 3-2 of the Hydrology Manual to determine T_i using the overland time of flow nomograph (Figure 3-1 of the Hydrology Manual). Because the initial subarea is a low-density development in which impervious areas are not directly connected to a storm drain system, use the Kirpich formula (Figure 3-2 of the Hydrology Manual) to determine the travel time across the remaining 150' of length in the subarea.

From Figure 3-1 of the Hydrology Manual, $T_i = 8.5$ minutes for the initial 70' travel length.

From the Kirpich formula presented in Figure 3-2 of the Hydrology Manual, $T_t = 2.1$ minutes for the remaining 150' of length in the subarea.

The total time of concentration across initial subarea nodes 0101 to 0102 is $8.5 + 2.1 = 10.6$ minutes.

Determine the intensity for this total time of concentration. The intensity for the T_c of 10.6 minutes will be between the given data points for $I_{10\text{-min}}$ (3.49 inches per hour) and $I_{15\text{-min}}$ (2.82 inches per hour). Use log-log interpolation to calculate $I_{10.6\text{-min}}$.

$$I_{10.6\text{-min}} = \text{Log}^{-1}(\text{Log}(3.49) + (\text{Log}(10.6 / 10)) (\text{Log}(2.82 / 3.49)/(\text{Log}(15 / 10))))$$
$$I_{10.6\text{-min}} = 3.4 \text{ in/hr}$$

Now calculate Q for node 0102 using the rational method equation.

$$Q_{0102} = \Sigma(CA)I = 0.21 (3.4) = 0.7 \text{ cfs}$$

Flow from point 0102 to 0103

The next step is to determine T_t for the length between point 0102 and 0103. The watercourse is a gutter and to calculate T_t it is necessary to know the water velocity, V , in the gutter. However, because the gutter is not a closed conduit, and flow from the subarea is being added, determination of T_t is an iterative process. To find V , assume an average Q over the watercourse (discharges for small watersheds typically range from 1 to 3 cfs per acre, depending on land use, drainage area, slope, and rainfall intensity). This is accomplished using the following method:

- Estimate Q_{AVG} and slope, s_{AVG} , to determine V . Estimate q_{avg} as 2.5 cfs/acre.

$$\text{Assume } Q_{AVG} = Q_{0102} + ((q_{avg})(A_{0102-0103})/2)$$

$$Q_{AVG} = 0.7 \text{ cfs} + ((2.5 \text{ cfs/acre})(1.8 \text{ acres})/2) \cong 3.0 \text{ cfs}$$

$$s_{AVG} = \frac{329.5' - 326.8'}{285'} = 0.01 = 1\%$$

- From Figure 3-4 of the Hydrology Manual, use Q_{AVG} and slope, s_{AVG} , to determine V .

$$V = 2.4 \text{ fps}$$

Then:

$$T_t = \frac{285'}{2.4 \text{ fps}} = 119 \text{ seconds} = 2.0 \text{ minutes}$$

$$T_c = \text{Time of concentration from previous step} + T_t = 10.6 + 2.0 = 12.6 \text{ minutes}$$

- Determine the intensity, I' for T_c . The intensity for the T_c of 12.6 minutes will be between the given data points for $I_{10\text{-min}}$ (3.49 inches per hour) and $I_{15\text{-min}}$ (2.82 inches per hour). Use log-log interpolation to calculate I' .

$$I' = \text{Log}^{-1}(\text{Log}(3.49) + (\text{Log}(12.6/10)) (\text{Log}(2.82 / 3.49)/(\text{Log}(15 / 10))))$$

$$I' = 3.1 \text{ in/hr}$$

$$Q_p = \Sigma(CA)I$$

$$\begin{aligned} Q_{0103} &= [CA_{0101-0102} + CA_{0102-0103}] I' \\ &= [0.52 (0.4) + 0.52 (1.8)] 3.1 = 3.5 \text{ cfs} \end{aligned}$$

Check the earlier assumption that Q_{AVG} from node 0102 to node 0103 was 3.0 cfs.

$$Q_{AVG} = Q_{0102} + ((Q_{0103} - Q_{0102})/2)$$

$$Q_{AVG} = 0.7 + ((3.5 - 0.7)/2) = 2.1 \text{ cfs} \neq 3.0 \text{ cfs}$$

At this point, retry the calculation with a different estimate of q_{avg} , say 1.6 cfs/acre for the area from nodes 0102-0103.

$$\text{Assume } Q_{AVG} = Q_{0102} + (q_{avg})(A_{0102-0103})$$

$$Q_{AVG} = 0.7 \text{ cfs} + ((1.6 \text{ cfs/acre})(1.8 \text{ acres})/2) = 2.1 \text{ cfs}$$

- From Figure 3-4 of the Hydrology Manual, input Q_{AVG} and slope, s_{AVG} , to determine V .

$$V = 2.3 \text{ fps}$$

Then:

$$T_t = 285' / 2.3 = 124 \text{ seconds} = 2.1 \text{ minutes}$$

$$T_c = T_i + T_t = 10.6 + 2.1 = 12.7 \text{ minutes}$$

- Use T_c and log-log interpolation between given time intensity pairs to redetermine I' .

$$I' = 3.1 \text{ in/hr}$$

$$\begin{aligned} Q_{0103} &= [CA_{0101-0102} + CA_{0102-0103}] I' \\ &= [0.52 (0.4) + 0.52 (1.8)] 3.1 = 3.5 \text{ cfs} \end{aligned}$$

Check the earlier assumption that Q_{AVG} from point 0102 to point 0103 was 2.3 cfs.

$$\begin{aligned} Q_{AVG} &= Q_{0102} + ((Q_{0103} - Q_{0102})/2) \\ Q_{AVG} &= 0.7 + ((3.5 - 0.7)/2) = 2.1 \text{ cfs} = 2.1 \text{ cfs; OK} \end{aligned}$$

Final results for node 0103:

$$Q_{0103} = 3.5 \text{ cfs}$$

$$T_c = 12.7 \text{ minutes}$$

$$I = 3.1 \text{ inches/hour}$$

$$A = 0.4 + 1.8 = 2.2 \text{ acres}$$

$$C = 0.52$$

WB.2.2 MODIFIED RATIONAL METHOD (FOR JUNCTION ANALYSIS) **(Reference Hydrology Manual Section 3.4)**

WB.2.2.1 Example #1, Junction Equation

The objective of this example is to show how the Q_p and T_c are obtained for a multiple subarea junction. The example is a junction of three independent drainage systems, each with Q_p , T_c , and I calculated by the RM.

(102) Input 1

$$Q_{p102} = 6.6 \text{ cfs}$$
$$T_{c102} = 10.2 \text{ minutes}$$
$$I_{102} = 4.9 \text{ in/hr}$$
$$A = 2.3 \text{ acres}$$

(201) Input 2

$$Q_{p201} = 10.5 \text{ cfs}$$
$$T_{c201} = 11.2 \text{ minutes}$$
$$I_{201} = 3.1 \text{ in/hr}$$
$$A = 6.1 \text{ acres}$$

(301) Input 3

$$Q_{p301} = 17.6 \text{ cfs}$$
$$T_{c301} = 9.8 \text{ minutes}$$
$$I_{301} = 5.1 \text{ in/hr}$$
$$A = 4.7 \text{ acres}$$

When the three inputs are inserted into the junction equation, the result is:

$$T_1 < T_2 < T_3$$

$$T_{301} < T_{102} < T_{201}$$

$$\begin{aligned} Q_{T1} &= Q_{301} + \frac{T_{301}}{T_{102}} Q_{102} + \frac{T_{301}}{T_{201}} Q_{201} \\ &= 17.6 + \frac{9.8}{10.2} (6.6) + \frac{9.8}{11.2} (10.5) \\ &= 33.1 \end{aligned}$$

$$\begin{aligned} Q_{T2} &= Q_{102} + \frac{I_{102}}{I_{301}} Q_{301} + \frac{T_{102}}{T_{201}} Q_{201} \\ &= 6.6 + \frac{4.9}{5.1} (17.6) + \frac{10.2}{11.2} (10.5) \\ &= 33.1 \end{aligned}$$

$$\begin{aligned} Q_{T3} &= Q_{201} + \frac{I_{201}}{I_{301}} Q_{301} + \frac{I_{201}}{I_{102}} Q_{102} \\ &= 10.5 + \frac{3.1}{5.1} (17.6) + \frac{3.1}{4.9} (6.6) \\ &= 25.4 \end{aligned}$$

Select the largest Q and use the T_c associated with that Q for further calculations. In this case, $Q_{T1} = Q_{T2} > Q_{T3}$. Select the shorter of the T_c 's associated with the larger Q. Use: $Q_{T1} = 33.1$ cfs and $T_1 = 9.8$ minutes for downstream calculations.

WB.2.2.2 Example #2, Modified Rational Method

This example demonstrates application of the MRM for a small urban watershed. Figure WB.2-2 shows a schematic of the watershed. The northern portion of the watershed is composed of three independent drainage systems that drain to a junction at node 14. A single drainage system continues from node 14 to node 16. Subareas have been defined based on land use, topography, and drainage structures, and node numbers have been placed at points of interest. Data for the example problem are given in Table WB.2-1. The procedure for calculating flow for each subarea is described in the text below. Table WB.2-2 presents a summary of the results.

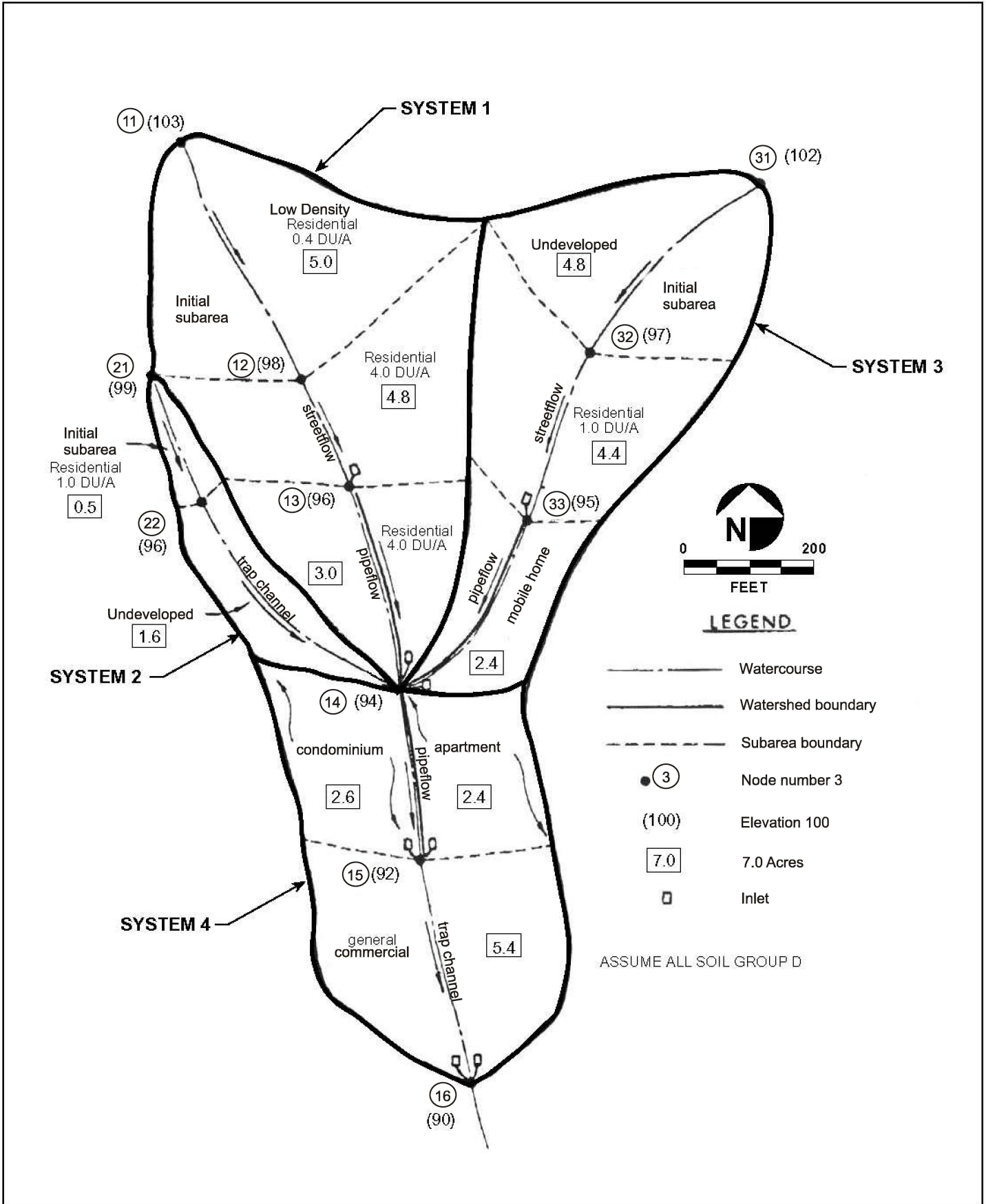
The following precipitation intensity data pairs obtained from NOAA Atlas 14 will be used in this example:

$I_{5\text{-min}} = 4.87$ inches per hour

$I_{10\text{-min}} = 3.49$ inches per hour

$I_{15\text{-min}} = 2.82$ inches per hour

$I_{30\text{-min}} = 1.95$ inches per hour



Example Discharge Area -
Modified Rational Method Example #2

FIGURE
WB.2-2

Table WB.2-1
INPUT DATA FOR MODIFIED RATIONAL METHOD EXAMPLE #2

Upstream Node	Downstream Node	Area (acres)	Runoff Coefficient*	Upstream Elevation (ft)	Downstream Elevation (ft)	Length (ft)	Drainage System
System 1							
11	12	5.0	0.41	103	98	400	Initial Subarea
12	13	4.8	0.52	98	96	175	Street Flow
13	14	3.0	0.52	96	94	325	Pipe Flow
System 2							
21	22	0.5	0.41	99	96	200	Initial Subarea
22	14	1.6	0.35	96	94	425	Trapezoidal Channel
System 3							
31	32	4.8	0.35	102	97	375	Initial Subarea
32	33	4.4	0.41	97	95	275	Street Flow
33	14	2.4	0.79	95	94	350	Pipe Flow
System 4							
14	15	2.6 and 2.4	0.63 and 0.71	94	92	275	Pipe Flow
15	16	5.4	0.82	92	90	350	Trapezoidal Channel

* For the purpose of this example, some assumptions were made about land use categories for selection of the runoff coefficient (e.g., the number of dwelling units per acre was assumed for condominiums, apartments, and mobile homes; and “undeveloped” land was assumed to be permanently undeveloped). The engineer must consult the Tentative Map(s) for the area(s) or the community’s General Plan for this information.

Table WB.2-2

SUMMARY OF MODIFIED RATIONAL METHOD EXAMPLE #2 CALCULATIONS

Upstream Node	Downstream Node	Area (acres)	Runoff Coefficient	ΣA (acres)	$\Sigma(CA)$ (acres)	ΣT_c (minutes)	I (inches/hour)	Q (cfs)
System 1								
11	12	5.0	0.41	5.0	2.1	13.2	3.02	6.3
12	13	4.8	0.52	9.8	4.6	14.1	2.91	13.4
13	14	3.0	0.52	12.8	6.2	15.0	2.82	17.5
System 2								
21	22	0.5	0.41	0.5	0.2	10.8	3.35	0.7
22	14	1.6	0.35	2.1	0.8	13.6	2.97	2.4
System 3								
31	32	4.8	0.35	4.8	1.7	13.7	2.96	5.0
32	33	4.4	0.41	9.2	3.5	15.5	2.77	9.7
33	14	2.4	0.79	11.6	5.4	16.5	2.68	14.5
Junction of Systems 1 through 3				26.5				33.3
System 4								
14	15	2.6 and 2.4	0.63 and 0.71	31.5	15.7	17.1	2.63	41.3
15	16	5.4	0.82	36.9	20.1	18.1	2.55	51.3

The first step is to calculate T_c , intensity, peak flow, and area for each of the three independent drainage systems draining to node 14.

System 1:

Flow at node 14 from System 1 (Q_1) is composed of flow in a pipe from two upstream subareas draining to node 13, and additional subarea flow from the area between nodes 13-14. The initial subarea is a low density residential development in which impervious areas are not directly connected to a storm drain system.

1. Overland flow across low density residential initial subarea nodes 11-12

$$C_{11-12} = 0.41$$

$$A_{11-12} = 5.0 \text{ acres}$$

$$\Sigma(CA) = (0.41)(5.0) = 2.1$$

$L = 400$ feet (use 70 feet maximum length per Table 3-2 of the Hydrology Manual to determine T_i).

$$\Delta E = 103 - 98 = 5 \text{ feet}$$

$$S = (103-98)/400 = 1.3\%$$

To determine the time of concentration, first use a 70' maximum length per Table 3-2 of the Hydrology Manual to determine T_i using the overland time of flow nomograph (Figure 3-1 of the Hydrology Manual). Because the initial subarea is a low-density development in which impervious areas are not directly connected to a storm drain system, use the Kirpich formula (Figure 3-2 of the Hydrology Manual) to determine the travel time T_t across the remaining 330' of length in the subarea.

From Figure 3-1 of the Hydrology Manual, $T_i = 9.5$ minutes for the initial 70' travel length

From Figure 3-2 of the Hydrology Manual, $T_t = 3.7$ minutes for the remaining 330' of length in the subarea.

The total time of concentration across rural initial subarea nodes 11-12 is $9.5 + 3.7 = 13.2$ minutes.

The intensity for the T_c of 13.2 minutes will be between the given data points for $I_{10\text{-min}}$ (3.49 inches per hour) and $I_{15\text{-min}}$ (2.82 inches per hour). Use log-log interpolation to calculate $I_{13.2\text{-min}}$.

$$I_{13.2\text{-min}} = \text{Log}^{-1}(\text{Log}(3.49) + (\text{Log}(13.2 / 10)) (\text{Log}(2.82 / 3.49)/(\text{Log}(15 / 10))))$$

$$I_{13.2\text{-min}} = 3.02 \text{ in/hr}$$

$$Q_{12} = \Sigma(CA)I = (2.1)(3.02) = 6.3 \text{ cfs}$$

2. Street flow from nodes 12-13

Because the gutter is not a closed conduit, additional flow from the subarea is being added during the T_t time of flow from nodes 12-13. An average flow must be assumed for the gutter to determine velocity in the gutter, T_c , and total flow at node 13.

$$C_{12-13} = 0.52$$

$$A_{12-13} = 4.8 \text{ acres}$$

$$\Sigma(CA) = [2.1 + ((0.52)(4.8))] = 4.6$$

$$L = 175 \text{ feet}$$

$$S = (98 - 96) / 175 = 0.011 \text{ (1.1\%)}$$

$$\text{Assume } Q_{\text{AVG}} \text{ from nodes 12-13} = Q_{12} + (q_{\text{avg}})(A_{12-13}/2).$$

Assume q_{avg} is 2.0 cfs/acre.

$$Q_{\text{AVG}} = 6.3 + (2)(4.8/2) = 11.1 \text{ cfs}$$

From Figure 3-4 of the Hydrology Manual, $V = 3.2 \text{ fps}$

$$T_t = (L/V)(1/60) = (175/3.2)(1/60) = 0.9 \text{ minutes}$$

$$T_{c13} = T_i + T_t = 13.2 + 0.9 = 14.1 \text{ minutes}$$

Using log-log interpolation between given time intensity pairs, $I' = 2.91$ inches/hour

$$Q_{13} = \Sigma(CA)I' = (4.6)(2.91) = 13.4 \text{ cfs}$$

Check the assumption that $Q_{AVG} = 11.1$ cfs:

$$Q_{AVG} = Q_{12} + ((Q_{13}-Q_{12})/2) = 6.3 + ((13.4 - 6.3)/2) = 9.9 \text{ cfs} \neq 11.1 \text{ cfs}$$

Try again assuming q_{avg} is 1.5 cfs/acre.

Again assume Q_{AVG} from nodes 12-13 = $Q_{12} + (1.5 \text{ cfs/acre})(A_{12-13}/2)$.

$$Q_{AVG} = 6.3 + (1.5)(4.8/2) = 9.9 \text{ cfs}$$

From Figure 3-4 of the Hydrology Manual, $V = 3.1$ fps

$$T_t = (L/V)(1/60) = (175/3.1)(1/60) = 0.9 \text{ minutes}$$

$$T_{c13} = T_i + T_t = 13.2 + 0.9 = 14.1 \text{ minutes}$$

As determined above, $I' = 2.91$ inches/hour

$$Q_{13} = \Sigma(CA)I' = (4.6)(2.91) = 13.4 \text{ cfs}$$

Check the assumption that $Q_{AVG} = 9.9$ cfs:

$$Q_{AVG} = Q_{12} + ((Q_{13}-Q_{12})/2) = 6.3 + ((13.4 - 6.3)/2) = 9.9 \text{ cfs} = 9.9 \text{ cfs, OK}$$

3. Pipe flow from nodes 13-14 and addition of subarea flow:

Because the pipe is a closed conduit, no additional subarea flow is added to the pipe during travel, and the T_t for flow in the pipe is based on the flow in the pipe. The subarea flow from the area between nodes 13-14 is added into the pipe at node 14 by adding the additional CA for the area between nodes 13-14 to the previous total CA from node 13 and multiplying the total by the intensity at node 14. This will give the new peak discharge in the pipe at node 14. If an inlet is to be designed for the subarea flow between nodes 13-14, then a peak discharge for basin 13-14 should be calculated and a junction analysis should be done at node 14 to combine these flows.

$$Q_{13} = 13.4 \text{ cfs in the pipe (from node 13)}$$

A 24-inch pipe can adequately convey 13.4 cfs at a slope of 0.6%. Assume a 24-inch pipe will be used; $V = 6.1 \text{ fps}$

$$T_t = 325/6.1(1/60) = 0.9 \text{ minutes}$$

$$T_{c14} = T_{c13} + T_{t13-14} = 14.1 + 0.9 = 15.0 \text{ minutes}$$

From the given time intensity pairs, $I = 2.82 \text{ inches/hour}$

$$C_{13-14} = 0.52$$

$$A_{13-14} = 3.0$$

$$\Sigma(\text{CA}) = [4.6 + (0.52)(3.0)] = 6.2$$

System 1 Summary:

$$Q_1 = \Sigma(CA)I = 6.2(2.82) = 17.5 \text{ cfs}$$

$$T_{c1} = 15.0 \text{ minutes}$$

$$I_1 = 2.82 \text{ inches/hour}$$

$$A_1 = 5 + 4.8 + 3.0 = 12.8 \text{ acres}$$

System 2:

Flow at node 14 from System 2 (Q_2) is composed of flow in the trapezoidal channel from the initial subarea draining to node 22, and additional subarea flow from the area between nodes 22-14.

1. Overland flow across urban initial subarea nodes 21-22:

$$C_{21-22} = 0.41$$

$$A_{21-22} = 0.5 \text{ acres}$$

$$\Sigma(CA) = (0.41)(0.5) = 0.2$$

$$L = 200 \text{ feet}$$

$$S = (99 - 96)/200 = 0.015 \text{ (1.5\%)}$$

To determine the time of concentration, first use a 70' maximum length per Table 3-2 of the Hydrology Manual to determine T_i using the overland time of flow nomograph (Figure 3-1 of the Hydrology Manual). Because the initial subarea is a low-density development in which impervious areas are not directly connected to a storm drain system, use the Kirpich formula (Figure 3-2 of the Hydrology Manual) to determine the travel time T_t across the remaining 130' of length in the subarea.

From Figure 3-1 of the Hydrology Manual, $T_i = 9.1$ minutes for the initial 70' travel length.

From Figure 3-2 of the Hydrology Manual, $T_t = 1.7$ minutes for the remaining 130' of length in the subarea.

The total time of concentration across initial subarea is $9.1 + 1.7 = 10.8$ minutes.
Using log-log interpolation between given time intensity pairs, $I = 3.35$ inches/hour

$$Q_{22} = \Sigma(CA)I = (0.2)(3.35) = 0.7 \text{ cfs}$$

2. Channel flow from nodes 22-14 and addition of subarea flow:

For this example, the channel from nodes 22-14 is assumed to be a natural channel. Because the channel is not a closed conduit, additional flow from the subarea is being added during the T_t of flow from nodes 22-14. An average flow must be assumed for the channel to determine velocity in the channel, T_c , and total flow from System 2 at node 14.

$$C_{22-14} = 0.35$$

$$A_{22-14} = 1.6 \text{ acres}$$

$$\Sigma(CA) = [0.2 + ((0.35)(1.6))] = 0.8$$

$$L = 425 \text{ feet}$$

$$S = (96 - 94) / 425 = 0.005 \text{ (0.5\%)}$$

Assume q_{avg} is 1.1 cfs/acre.

$$\text{Assume } Q_{AVG} \text{ from nodes 22-14} = Q_{22} + (1.1 \text{ cfs/acre})(A_{22-14}/2).$$

$$Q_{AVG} = 0.7 + (1.1)(1.6/2) = 1.6 \text{ cfs}$$

Assume that the channel is vegetated, and $n = 0.035$.

A 1-foot-wide channel with 1.5:1 side slopes can adequately convey the flow in the channel. Assume $V \cong 2.5$ fps.

$$T_t = (L/V)(1/60) = (425/2.5)(1/60) = 2.8 \text{ minutes}$$

$$T_c = T_i + T_t = 10.8 + 2.8 = 13.6 \text{ minutes}$$

Using log-log interpolation between given time intensity pairs, $I = 2.97$ inches/hour

$$Q_{14} = \Sigma(CA)I = (0.8)(2.97) = 2.4 \text{ cfs}$$

Check the assumption that $Q_{AVG} = 1.6$ cfs:

$$Q_{AVG} = Q_{22} + ((Q_{22} - Q_{14})/2) = 0.7 + ((2.4 - 0.7)/2) = 1.6 \text{ cfs} = 1.6 \text{ cfs; OK}$$

System 2 Summary:

$$Q_2 = 2.4 \text{ cfs}$$

$$T_{c2} = 13.6 \text{ minutes}$$

$$I_2 = 2.97 \text{ inches/hour}$$

$$A_2 = 0.5 + 1.6 = 2.1 \text{ acres}$$

System 3:

Flow at node 14 from System 3 (Q_3) is composed of flow in the pipe from the subareas draining to node 33, and additional subarea flow from the area between nodes 33-14.

1. Overland flow across natural initial subarea nodes 31-32:

$$C_{31-32} = 0.35$$

$$A_{31-32} = 4.8 \text{ acres}$$

$$\Sigma(CA) = (0.35)(4.8) = 1.7$$

$$\Delta E = 102 - 97 = 5 \text{ feet}$$

$$L = 375 \text{ feet}$$

$$S = (102-97)/375 = 0.013 = 1.3\%$$

To determine the time of concentration, first use a 70' maximum length per Table 3-2 of the Hydrology Manual to determine T_i from Figure 3-1 of the Hydrology Manual, then use the Kirpich formula (Figure 3-2 of the Hydrology Manual) to determine the travel time T_t across the remaining 305' of length in the subarea. From Figure 3-1, $T_i = 10.3$ minutes. From Figure 3-2, $T_t = 3.4$ minutes. The total time of concentration across natural initial subarea nodes 31-32 is $10.3 + 3.4 = 13.7$ minutes.

Using log-log interpolation between given time intensity pairs, $I = 2.96$ inches/hour

$$Q_{32} = \Sigma(CA)I = (1.7)(2.96) = 5.0 \text{ cfs}$$

2. Street flow from nodes 32-33

Because the gutter is not a closed conduit, additional flow from the subarea is being added during the T_t of flow from nodes 32-33. An average flow must be assumed for the gutter to determine velocity in the gutter, T_c , and total flow at node 33.

$$C_{32-33} = 0.41$$

$$A_{32-33} = 4.4 \text{ acres}$$

$$\Sigma(CA) = [1.7 + ((0.41)(4.4))] = 3.5$$

$$L = 275 \text{ feet}$$

$$S = (97 - 95) / 275 = 0.007 \text{ (0.7\%)}$$

Assume q_{avg} is 1.1 cfs/acre.

$$\text{Assume } Q_{AVG} \text{ from nodes 32-33} = Q_{32} + (1.1 \text{ cfs/acre})(A_{32-33}/2).$$

$$Q_{AVG} = 5.0 + (1.1)(4.4/2) = 7.4 \text{ cfs}$$

From Figure 3-4 of the Hydrology Manual, $V = 2.5$ fps

$$T_t = (L/V)(1/60) = (275/2.5)(1/60) = 1.8 \text{ minutes}$$

$$T_{c33} = T_i + T_t = 13.7 + 1.8 = 15.5 \text{ minutes}$$

Using log-log interpolation between given time intensity pairs, $I = 2.77$ inches/hour

$$Q_{33} = \Sigma(CA)I = (3.5)(2.77) = 9.7 \text{ cfs}$$

Check the assumption that $Q_{AVG} = 7.4$ cfs:

$$Q_{AVG} = Q_{32} + ((Q_{33} - Q_{32})/2) = 5.0 + ((9.7 - 5.0)/2) = 7.4 \text{ cfs} = 7.4 \text{ cfs; OK}$$

3. Pipe flow from nodes 33-14 and addition of subarea flow:

Because the pipe is a closed conduit, no additional subarea flow is added to the pipe during travel, and the T_t for flow in the pipe is based on the flow in the pipe. As above for System 1, since we are not sizing an inlet for the subarea from nodes 33-14, the subarea flow from the area between nodes 33-14 is added into the pipe at node 14 by adding the additional CA for the area between nodes 33-14 to the previous total CA from node 33 and multiplying the total by the intensity at node 14. This will give the new peak discharge in the pipe at node 14.

$$Q_{33} = 9.7 \text{ cfs in the pipe (from node 33)}$$

A 24-inch pipe can adequately convey 9.7 cfs at a slope of 0.7%. Assume a 24-inch pipe will be used; $V = 6.1$ fps

$$T_t = (350/6.1)(1/60) = 1.0 \text{ minutes}$$

$$T_{c14} = T_{c33} + T_{t33-14} = 15.5 + 1.0 = 16.5 \text{ minutes}$$

Using log-log interpolation between given time intensity pairs, $I = 2.68$ inches/hour

$$C_{33-14} = 0.79$$

$$C_{33-14} = 2.4 \text{ acres}$$

$$\Sigma(CA) = [3.5 + ((0.79) (2.4))] = 5.4$$

$$\Sigma(CA)I = (5.4)(2.68) = 14.5 \text{ cfs}$$

System 3 Summary:

$$Q_3 = 14.5 \text{ cfs}$$

$$T_{c3} = 16.5 \text{ minutes}$$

$$I_3 = 2.68 \text{ inches/hour}$$

$$A_3 = 4.8 + 4.4 + 2.4 = 11.6 \text{ acres}$$

Junction of Systems 1 through 3

The second step is to combine the three independent drainage systems draining to node 14. To combine independent drainage systems 1 through 3, a junction analysis will be used. Table WB.2-3 summarizes the junction data for the three systems.

Table WB.2-3

**SUMMARY OF CONFLUENCE DATA FOR
MODIFIED RATIONAL METHOD EXAMPLE #2
SYSTEMS 1 THROUGH 3**

System	Q (cfs)	T _c (minutes)	I (inches/hour)	A (acres)	ΣCA
1 = Y	17.5	15.0	2.82	12.8	6.2
2 = X	2.4	13.6	2.97	2.1	0.8
3 = Z	14.5	16.5	2.68	11.6	5.4

When the three inputs are inserted into the junction equation, the result is:

$$T_2 < T_1 < T_3$$

Since the Systems were identified by number, let $T_2 = T_X$, $T_1 = T_Y$, and $T_3 = T_Z$

$$T_X < T_Y < T_Z$$

$$Q_{TX} = Q_X + \frac{T_X}{T_Y} Q_Y + \frac{T_X}{T_Z} Q_Z$$
$$= 2.4 + (13.6/15.0)(17.5) + (13.6/16.5)(14.5)$$

$$= 30.2$$

$$Q_{TY} = Q_Y + \frac{I_Y}{I_X} Q_X + \frac{T_Y}{T_Z} Q_Z$$
$$= 17.5 + (2.82/2.97)(2.4) + (15.0/16.5)(14.5)$$

$$= 33.0$$

$$Q_{TZ} = Q_Z + \frac{I_Z}{I_X} Q_X + \frac{I_Z}{I_Y} Q_Y$$
$$= 14.5 + (2.68/2.97)(2.4) + (2.68/2.82)(17.5)$$

$$= 33.3$$

Select the largest Q and use the T_c associated with that Q for further calculations.

Use $Q_{TZ} = 33.3$ cfs and $T_Z = 16.5$ minutes.

The total area associated with this system is the sum of the drainage areas for the three contributing systems:

$$A = A_1 + A_2 + A_3 = 12.8 + 2.1 + 11.6 = 26.5 \text{ acres}$$

$$\Sigma CA = CA_1 + CA_2 + CA_3 = 6.2 + 0.8 + 5.4 = 12.4$$

System 4: Continuation of Single System to Node 16

The third step is to continue the analysis of the single system from node 14 using Q, A, and T_c from the junction analysis:

$$Q_{14} = 33.3 \text{ cfs}$$

$$T_{c14} = 16.5 \text{ minutes}$$

$$A_{14} = 26.5 \text{ acres}$$

$$\Sigma CA = 12.4$$

3. Pipe flow from nodes 14-15 and addition of subarea flow:

Because the pipe is a closed conduit, no additional subarea flow is added to the pipe during travel, and the T_t for flow in the pipe is based on the flow in the pipe. As above for Systems 1 and 3, since we are not sizing inlets for the subareas between nodes 14-15, flow is added into the pipe at node 15 by adding the additional CA for the area between nodes 14-15 to the previous total CA from node 14 and multiplying the total by the intensity at node 15. This will give the new peak discharge in the pipe at node 15.

$$L = 275 \text{ feet}$$

$$S = (94-92)/275 = 0.007 \text{ (0.7\%)}$$

A 30-inch pipe can adequately convey 33.3 cfs at a slope of 0.7%. Assume a 30-inch pipe will be used; $V = 7.8 \text{ fps}$

$$T_t = (275/7.8)(1/60) = 0.6 \text{ minutes}$$

$$T_{c15} = T_{c14} + T_{t14-15} = 16.5 + 0.6 = 17.1 \text{ minutes}$$

Using log-log interpolation between given time intensity pairs, $I = 2.63 \text{ inches/hour}$

$$C_{14-15} = 0.63 \text{ and } 0.71$$

$$A_{14-15} = 2.6 \text{ and } 2.4 \text{ acres}$$

$$\Sigma(CA) = [12.4 + (0.63)(2.6) + (0.71)(2.4)] = 15.7$$

$$Q_{15} = \Sigma(CA)I = (15.7) 2.63 = 41.3 \text{ cfs}$$

$$T_{c15} = 17.1 \text{ minutes}$$

$$I = 2.63 \text{ inches/hour}$$

$$A = 26.5 + 2.6 + 2.4 = 31.5 \text{ acres}$$

$$\Sigma CA = 15.7$$

4. Flow in trapezoidal channel from nodes 15-16 and addition of subarea flow:

For the general commercial area, the trapezoidal channel should be considered as a closed conduit because the subarea flow is directed to inlets. With no additional subarea flow added to the channel during travel, the T_t for flow in the channel is based on the total flow in the channel from node 15. As for the pipes sized above, since we are not sizing inlets for the subarea between nodes 15-16, the subarea flow from the areas between nodes 15-16 will be added directly to the total flow in the pipe at node 16 using the T_c for the flow in the pipe.

$$L = 350 \text{ feet}$$

$$S = (92-90)/350 = 0.006 (0.6\%)$$

Assume that the channel is concrete, and $n = 0.018$

A 1-foot-wide channel with 1.5:1 side slopes can adequately convey 41.3 cfs at a slope of 0.6%. Assume $V \cong 6.0$ fps.

$$T_{t15-16} = (350/6.0)(1/60) = 1.0 \text{ minutes}$$

$$T_{c16} = T_{c15} + T_{t15-16} = 17.1 + 1.0 = 18.1 \text{ minutes}$$

Using log-log interpolation between given time intensity pairs, $I = 2.55$ inches/hour

$$C_{15-16} = 0.82$$

$$A_{15-16} = 5.4 \text{ acres}$$

$$\Sigma(CA) = 15.7 + (0.82)(5.4) = 20.1$$

$$\Sigma(CA)I = (20.1)(2.55) = 51.3 \text{ cfs}$$

System 4 Summary:

$$Q_{16} = 51.3 \text{ cfs}$$

$$T_{c16} = 18.1 \text{ minutes}$$

$$I = 2.55 \text{ inches/hour}$$

$$A = 31.5 + 5.4 = 36.9 \text{ acre}$$

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WB.3 WORKBOOK EXAMPLES FOR HYDROLOGY MANUAL SECTION 4.0 NATURAL RESOURCES CONSERVATION SERVICE HYDROLOGIC METHOD

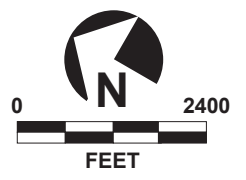
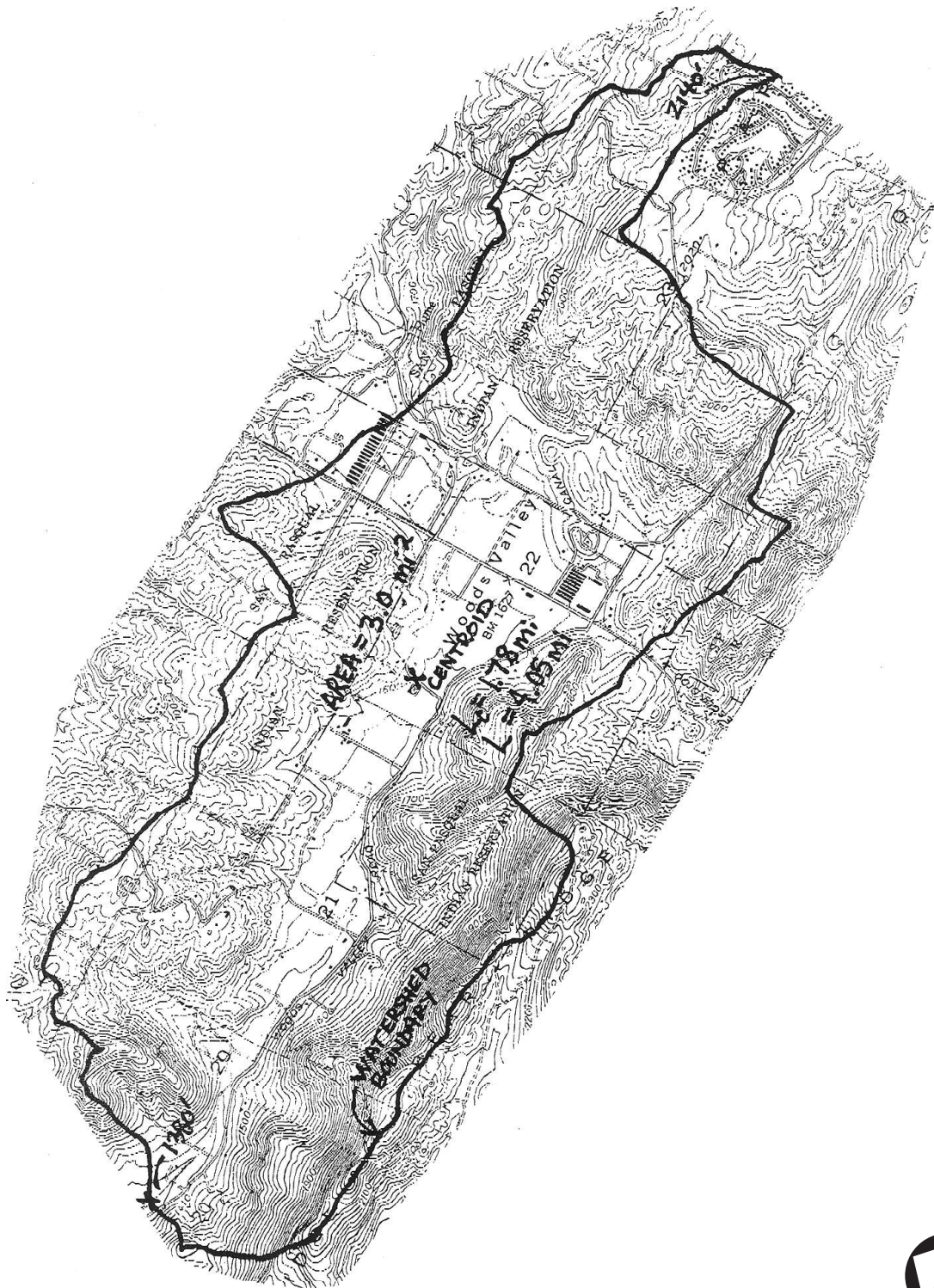
The first example problem in this Section demonstrates the computation of the CN, Corps lag, and T_p values based on NRCS methods and data, and demonstrates the use of the SDHydroTools computer program to obtain the peak discharge. The second example problem demonstrates the development of the rainfall distribution, calculation of excess rainfall, preparation of unit hydrograph ordinates, and convolution of the runoff hydrograph by hand computation. The results of the hand computation are compared to results from the SDHydroTools computer program and HEC-HMS.

WB.3.1 NRCS Hydrologic Method Example #1, Computation of CN, Corps Lag, T_p , and Peak Discharge (Reference Hydrology Manual Sections 4.2 and 4.3)

This example is a 100-year storm event for a 3-square-mile watershed.

WB.3.1.1 Example Watershed Geographic Location, Area, and Physical Characteristics (Reference Hydrology Manual Section 4.2.1)

Figure WB.3-1 shows the example watershed delineated on USGS topographic maps. The watershed area was measured using a planimeter and recorded on the map. Other relevant information that will be used for calculation of T_1 (watershed length, length to centroid, maximum elevation, minimum elevation, and \bar{n}) is also recorded on the map. The watershed length and length to centroid were measured along the watercourse. The centroid is the point where approximately 50% of the watershed area is contributing to the watercourse. Length to centroid is measured from the downstream point of the watershed to the centroid. The average of Manning's n values (\bar{n}) (described in Section 4.3.5 of the Hydrology Manual) for the watercourse and its tributaries was determined by a field visit.



SOURCE: USGS Valley Center and Rodriguez Mountain Quadrangles. 1988.

Example Watershed Geographic Location and Physical Characteristics

FIGURE
WB.3-1

WB.3.1.2 Example Precipitation and Precipitation Zone Number (Reference Hydrology Manual Section 4.2.2)

The design storm precipitation values and the PZN for the watershed were determined from NOAA Atlas 14 and from Appendix C of the Hydrology Manual, respectively. The PZN is 2.5. The following are the 100-year precipitation depth data pairs obtained from NOAA Atlas 14 (obtained for latitude 33.2077, longitude -116.9833) for this example study:

$P_{5\text{-min}} = 0.366$ inches

$P_{10\text{-min}} = 0.525$ inches

$P_{15\text{-min}} = 0.634$ inches

$P_{30\text{-min}} = 0.925$ inches

$P_{60\text{-min}} = 1.46$ inches

$P_{120\text{-min}} = 2.01$ inches

$P_{180\text{-min}} = 2.43$ inches

$P_{360\text{-min}} = 3.42$ inches

$P_{720\text{-min}} = 4.93$ inches

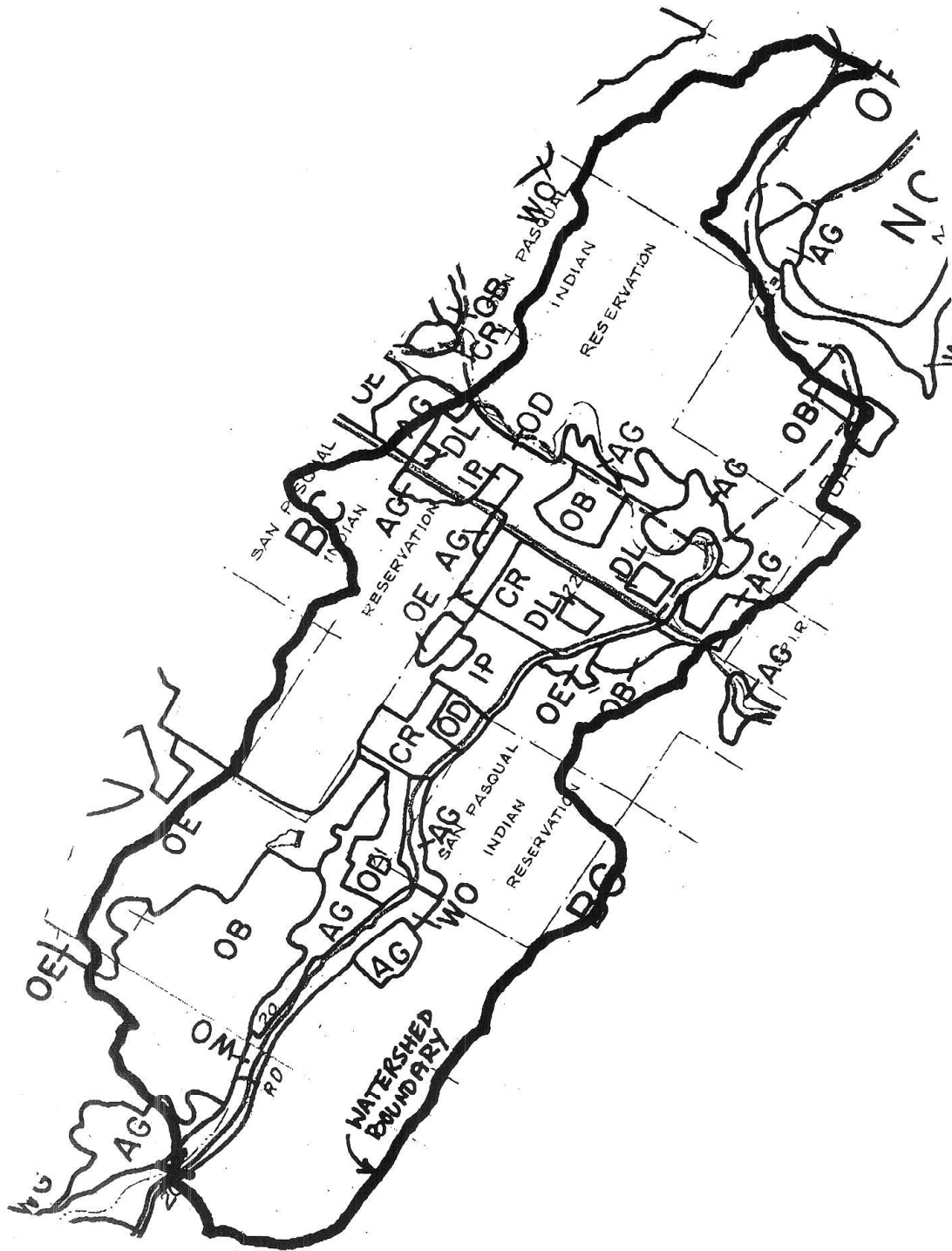
$P_{1440\text{-min}} = 6.79$ inches

The rainfall depth-area adjustment described in Section 4.1.1.3 of the Hydrology Manual will be performed by the SDHydroTools computer program. The SDHydroTools computer program will also prepare the rainfall distribution and calculate excess rainfall.

WB.3.1.3 Example Runoff Curve Number (Reference Hydrology Manual Section 4.2.3 and 4.2.4)

Figures WB.3-2 and WB.3-3 show NRCS hydrologic ground cover (land use) and NRCS soil groups within the project area, respectively. When translucent vellum with grid ticks is overlain on the maps, the land use/soil group combinations are as shown in Figure WB.3-4. Half-inch grid ticks were used for this study because the watershed is less than 10 square miles in size. Worksheet 4-1 of the Hydrology Manual was used to record the number of grid ticks for each combination. See Appendix WB.A (blank copies are provided in Appendix D of the Hydrology Manual).

The CNs for each land use/soil group combination were determined from Table 4-2 of the Hydrology Manual. For the purpose of this example, the hydrologic condition was assumed to be good, and the crop land was assumed to be straight row crops. For the deciduous orchards, cover during storm periods (winter time) was assumed to be annual grass in good condition.

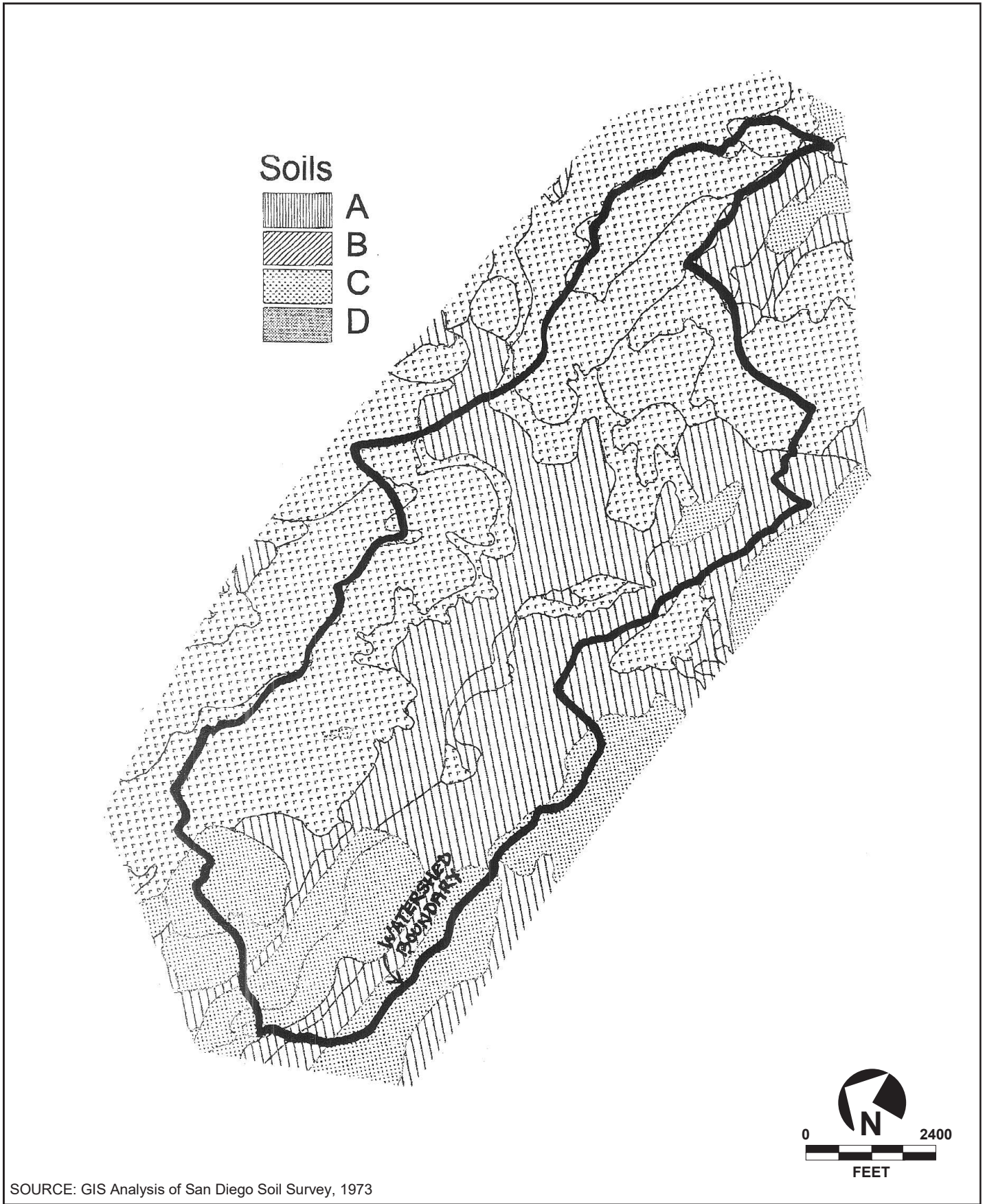


SOURCE: San Diego County Soils Interpretation Study, Ground Cover-Vegetative and Man Made Sheet 24 - Valley Center, 1969 and Sheet 25 - Rodriguez Mountain, 1969

Example Watershed Hydrologic Ground Cover

FIGURE

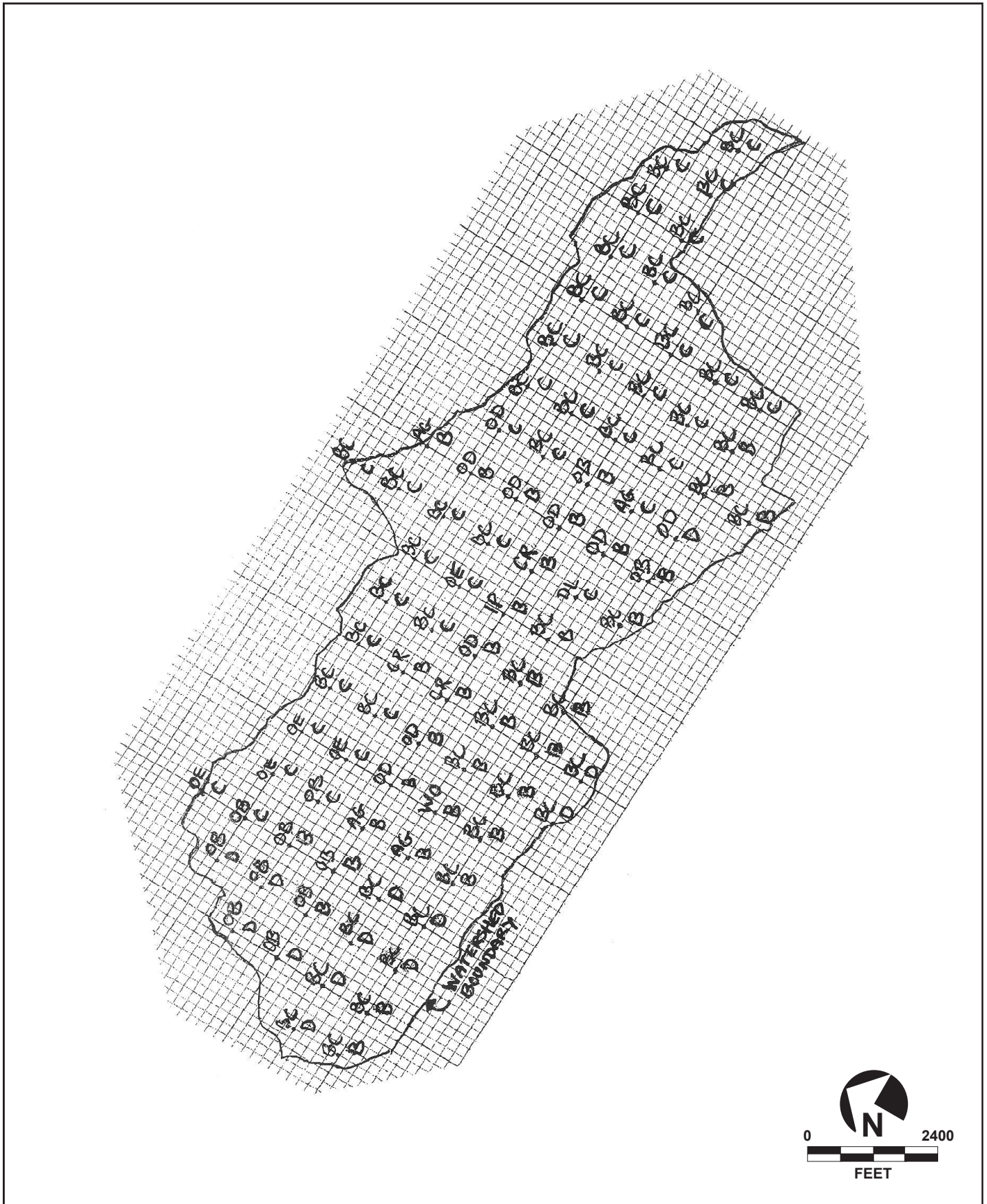
WB.3-2



SOURCE: GIS Analysis of San Diego Soil Survey, 1973

Example Watershed Hydrologic Soils Groups

FIGURE
WB.3-3



Example Watershed Grid Tick Overlay

FIGURE

WB.3-4

Calculations for the composite CN were performed using Worksheet 4-2 of the Hydrology Manual. See Appendix WB.A (blank copies are provided in Appendix D of the Hydrology Manual). A spreadsheet set up in the format of Table 4-9 of the Hydrology Manual could also be used for this calculation. The composite CN for the watershed unadjusted for PZN Condition is 69.

No adjustments are being made to this CN for unconnected impervious areas since the amount of developed land in the watershed as shown in Figure WB.3-1 is small. However, the CN must be adjusted for PZN Condition. The PZN adjustment factor is determined using the data provided in Table 4-6 of the Hydrology Manual. From Table 4-6 of the Hydrology Manual, based on the design storm frequency and the PZN, the PZN adjustment factor for this study is 3.0. From Table 4-10 of the Hydrology Manual, the composite CN for the watershed adjusted for PZN Condition is 84.

WB.3.1.4 Example Watershed Lag Time, Time to Peak, and Computation Interval (Reference Hydrology Manual Section 4.3.1)

Corps lag, defined as the elapsed time (in hours) from the beginning of unit effective rainfall to the instant that the summation hydrograph for the point of concentration reaches 50% of ultimate discharge for the watershed is determined using the lag relationship given by the empirical formula presented in Section 4.1.5.2 of the Hydrology Manual (equation 4-17). The watershed length (miles), length to centroid (miles), and \bar{n} are shown in Figure WB.3-1. The watershed slope is the difference in elevation between the high and low points of the watershed (in feet) divided by the watershed length (in miles). Using equation 4-17:

$$T_l \text{ (hours)} = 24 \bar{n} ((L \times L_c) / s^{0.5})^m$$

$$T_l \text{ (hours)} = 24 \times (0.050) ((4.05 \times 1.78) / 188^{0.5})^{0.38} = 0.94$$

T_p for the watershed is calculated based on Corps lag using the formula presented in Section 4.1.5.2 of the Hydrology Manual (equation 4-19):

$$T_p = 0.862 \text{ Corps } T_1$$

$$T_p = 0.862 \times 0.94 \text{ hours} = 0.81 \text{ hours}$$

An appropriate computation interval (D) for the NRCS hydrologic method calculations must be selected based on T_p . A small amount of variation is allowable in D, however D should be approximately $0.2T_p$.

For this example:

$$0.2 (0.81 \text{ hours}) (60 \text{ minutes} / \text{hour}) = 9.7 \text{ minutes}$$

A computation interval (D) of 5 minutes will be used.

WB.3.1.5 Example Watershed Peak Discharge Determination Using SDHydroTools Computer Program

The calculation of peak flow for this study is performed using the San Diego Unit Hydrograph (“SD Unit Hydrograph”) module in the SDHydroTools computer program. First, the rainfall data for the watershed is obtained from the 100-year data embedded within the program. The rainfall data is obtained by either entering the desired latitude and longitude coordinates or selecting the desired location on the map in the “Location” window under the “Project” tab, then selecting “GET DATA.” Next, the project physical data (watershed area, CN, and Corps lag time) and desired computation interval are entered in the “SD Unit Hydrograph” tab. When the physical data and computation interval have been entered in the “SD Unit Hydrograph” tab, select “CALCULATE.” The SDHydroTools computer program will apply the rainfall depth-area adjustment based on the watershed area, prepare the rainfall distribution based on the depth-area adjusted rainfall, calculate excess rainfall based on the CN, prepare the unit hydrograph ordinates

based on the T_p , which is determined from the user-entered Corps lag, and prepare the study hydrograph.

From the SDHydroTools computer program, using rainfall depths as shown in Section WB.3.1.2 (obtained for latitude 33.2077, longitude -116.9833), a watershed area of 3.0 miles, CN of 84, Corps lag of 0.94 hours, and computation interval of 5 minutes, the peak runoff from the watershed for this study is approximately 2,177 cubic feet per second. The SDHydroTools computer program input and output is provided in Appendix WB.A.

**WB.3.2 NRCS Hydrologic Method Example #2, Convolution of Runoff Hydrograph
by Hand Computation and Comparison of Results with SDHydroTools
Computer Program and HEC-HMS
(Reference Hydrology Manual Section 4.3)**

This example is a 100-year storm event for a watershed with the following data:

Area = 40.0 square-miles

PZN Adjusted CN = 85.0

Corps Lag = 1.74 hours

The following are the 100-year precipitation depth data pairs obtained from NOAA Atlas 14 (obtained for latitude 32.8918, longitude -116.8488) for this example study:

$P_{5\text{-min}} = 0.406$ inches

$P_{10\text{-min}} = 0.582$ inches

$P_{15\text{-min}} = 0.704$ inches

$P_{30\text{-min}} = 0.977$ inches

$P_{60\text{-min}} = 1.40$ inches

$P_{120\text{-min}} = 1.92$ inches

$P_{180\text{-min}} = 2.29$ inches

$P_{360\text{-min}} = 3.02$ inches

$P_{720\text{-min}} = 4.00$ inches

$P_{1440\text{-min}} = 5.38$ inches

**WB.3.2.1 Example Time to Peak and Computation Interval
(Reference Hydrology Manual Section 4.1.5.3)**

An appropriate computation interval (D) for the NRCS hydrologic method calculations must be selected based on T_p . A small amount of variation is allowable in D, however D should be approximately $0.2T_p$. Therefore, T_p must be determined for the watershed. T_p can be calculated based on Corps lag using equation 4-19:

$$T_p = 0.862 \text{ Corps } T_1$$

For this study, with Corps lag given as 1.74 hours:

$$T_p = 0.862 (1.74 \text{ hours}) = 1.5 \text{ hours}$$

$$0.2 (1.5 \text{ hours}) (60 \text{ minutes} / \text{hour}) = 18 \text{ minutes}$$

A computation interval (D) of 15 minutes will be used.

WB.3.2.2 Example Precipitation Distribution (Reference Hydrology Manual Sections 4.1.1 and 4.3.2)

Creation of the 24-hour nested storm rainfall distribution requires rainfall depths for increments of storm duration from the selected computation interval (15 minutes for this study) through 24 hours. Precipitation depth data for selected durations (e.g., 15 minutes, 30 minutes, 60 minutes, etc.) are obtained from NOAA Atlas 14, as shown above in the data given for this example. For a duration that is between given data points from NOAA Atlas 14 (e.g., 45 minutes, 75 minutes, etc.), use log-log interpolation to interpolate the depth for the duration, using the following equation:

$$P = \text{Log}^{-1}(\text{Log}P_1 + (\text{Log}(T / T_1)) (\text{Log}(P_2 / P_1)/(\text{Log}(T_2 / T_1))))$$

Where:

P = Rainfall depth at the desired duration (inches)

T = Desired duration (minutes)

T₁ and T₂ are the bounding times (minutes) of given data points from NOAA Atlas 14
and T₁ < T < T₂

P₁ = Rainfall depth at T₁ (inches per hour) from NOAA Atlas 14

P₂ = Rainfall depth at T₂ (inches per hour) from NOAA Atlas 14

Next, the total rainfall depth for each duration must be adjusted using the appropriate depth-area adjustment values based on the watershed area from Figure 4-2 or Table 4-1 of the

Hydrology Manual (the rainfall amount is multiplied by the depth-area adjustment factor). For durations less than 30 minutes, the 30-minute depth area adjustment value is used. For durations greater than 30 minutes and not equal to durations with data available on Table 4-1 of the Hydrology Manual, the depth area adjustment is interpolated by linear interpolation between the surrounding data points on Table 4-1 of the Hydrology Manual (an alternative is to read the data from Figure 4-2 of the Hydrology Manual).

Next, the ordinates of the hyetograph are created using the depth-area adjusted total rainfall amounts. The first ordinate “ $R(D)$ ” is the depth-area adjusted total rainfall amount for the first time increment (15 minutes in this example study). The second ordinate “ $R(2D) - R(D)$ ” is the depth-area adjusted total rainfall amount for the second time increment (30 minutes in this example study) minus the depth-area adjusted total rainfall amount for the first time increment. The third ordinate “ $R(3D) - R(2D)$ ” is the depth-area adjusted total rainfall amount for the third time increment minus depth-area adjusted total rainfall amount for the second time increment, and so on. Note: the sum of the ordinates of the hyetograph should be equal to the depth-area adjusted total rainfall amount for duration = 24 hours (5.154 inches for this study).

The data resulting from these equations are presented in Table WB.3-1.

Table WB.3-1 (Page 1 of 4)

**NRCS HYDROLOGIC METHOD EXAMPLE #2 RAINFALL DISTRIBUTION DATA
 SORTED IN ORDER OF INCREASING DURATION**

Duration (minutes)	Precipitation for Duration, P (inches)	Depth Area Adjustment for Duration	Depth-Area Adjusted Precipitation (inches)	Hyetograph Ordinate, R (inches)
15	0.704	0.730	0.514	0.514
30	0.977	0.730	0.713	0.199
45	1.206	0.780	0.941	0.227
60	1.400	0.830	1.162	0.221
75	1.550	0.841	1.303	0.141
90	1.684	0.851	1.434	0.131
105	1.807	0.862	1.557	0.124
120	1.920	0.873	1.675	0.118
135	2.021	0.883	1.785	0.109
150	2.116	0.894	1.891	0.106
165	2.205	0.904	1.994	0.103
180	2.290	0.915	2.095	0.101
195	2.364	0.917	2.168	0.073
210	2.435	0.919	2.238	0.070
225	2.503	0.921	2.306	0.068
240	2.569	0.923	2.372	0.066
255	2.632	0.925	2.435	0.064
270	2.692	0.928	2.497	0.062
285	2.751	0.930	2.557	0.060
300	2.808	0.932	2.616	0.059
315	2.863	0.934	2.674	0.057
330	2.917	0.936	2.730	0.056
345	2.969	0.938	2.785	0.055
360	3.02	0.940	2.839	0.054

Table WB.3-1 Continued (Page 2 of 4)

**NRCS HYDROLOGIC METHOD EXAMPLE #2 RAINFALL DISTRIBUTION DATA
 SORTED IN ORDER OF INCREASING DURATION**

Duration (minutes)	Precipitation for Duration, P (inches)	Depth Area Adjustment for Duration	Depth-Area Adjusted Precipitation (inches)	Hyetograph Ordinate, R (inches)
375	3.070	0.940	2.887	0.048
390	3.120	0.941	2.934	0.047
405	3.168	0.941	2.980	0.046
420	3.215	0.941	3.025	0.045
435	3.261	0.941	3.069	0.044
450	3.306	0.942	3.113	0.043
465	3.350	0.942	3.155	0.042
480	3.394	0.942	3.197	0.042
495	3.436	0.942	3.238	0.041
510	3.478	0.943	3.278	0.040
525	3.519	0.943	3.318	0.040
540	3.560	0.943	3.357	0.039
555	3.599	0.943	3.395	0.038
570	3.639	0.944	3.433	0.038
585	3.677	0.944	3.470	0.037
600	3.715	0.944	3.507	0.037
615	3.752	0.944	3.543	0.036
630	3.789	0.945	3.579	0.036
645	3.826	0.945	3.614	0.035
660	3.861	0.945	3.649	0.035
675	3.897	0.945	3.683	0.034
690	3.932	0.946	3.717	0.034
705	3.966	0.946	3.751	0.034
720	4.000	0.946	3.784	0.033

Table WB.3-1 Continued (Page 3 of 4)

**NRCS HYDROLOGIC METHOD EXAMPLE #2 RAINFALL DISTRIBUTION DATA
 SORTED IN ORDER OF INCREASING DURATION**

Duration (minutes)	Precipitation for Duration, P (inches)	Depth Area Adjustment for Duration	Depth-Area Adjusted Precipitation (inches)	Hyetograph Ordinate, R (inches)
735	4.035	0.946	3.819	0.035
750	4.070	0.947	3.853	0.034
765	4.105	0.947	3.886	0.034
780	4.139	0.947	3.920	0.033
795	4.173	0.947	3.953	0.033
810	4.207	0.948	3.986	0.033
825	4.240	0.948	4.018	0.032
840	4.273	0.948	4.050	0.032
855	4.305	0.948	4.082	0.032
870	4.337	0.949	4.114	0.032
885	4.369	0.949	4.145	0.031
900	4.400	0.949	4.176	0.031
915	4.432	0.949	4.207	0.031
930	4.463	0.950	4.237	0.030
945	4.493	0.950	4.267	0.030
960	4.524	0.950	4.297	0.030
975	4.554	0.950	4.327	0.030
990	4.584	0.951	4.357	0.029
1005	4.613	0.951	4.386	0.029
1020	4.642	0.951	4.415	0.029
1035	4.671	0.951	4.444	0.029
1050	4.700	0.952	4.472	0.029
1065	4.729	0.952	4.501	0.028
1080	4.757	0.952	4.529	0.028

Table WB.3-1 Continued (Page 4 of 4)

**NRCS HYDROLOGIC METHOD EXAMPLE #2 RAINFALL DISTRIBUTION DATA
 SORTED IN ORDER OF INCREASING DURATION**

Duration (minutes)	Precipitation for Duration, P (inches)	Depth Area Adjustment for Duration	Depth-Area Adjusted Precipitation (inches)	Hyetograph Ordinate, R (inches)
1095	4.785	0.952	4.557	0.028
1110	4.813	0.953	4.585	0.028
1125	4.841	0.953	4.612	0.028
1140	4.869	0.953	4.640	0.027
1155	4.896	0.953	4.667	0.027
1170	4.923	0.954	4.694	0.027
1185	4.950	0.954	4.721	0.027
1200	4.976	0.954	4.748	0.027
1215	5.003	0.954	4.774	0.027
1230	5.029	0.955	4.800	0.026
1245	5.055	0.955	4.827	0.026
1260	5.081	0.955	4.853	0.026
1275	5.107	0.955	4.879	0.026
1290	5.133	0.956	4.904	0.026
1305	5.158	0.956	4.930	0.026
1320	5.184	0.956	4.955	0.025
1335	5.209	0.956	4.981	0.025
1350	5.234	0.957	5.006	0.025
1365	5.258	0.957	5.031	0.025
1380	5.283	0.957	5.056	0.025
1395	5.307	0.957	5.081	0.025
1410	5.332	0.958	5.105	0.025
1425	5.356	0.958	5.130	0.024
1440	5.380	0.958	5.154	0.024
			Total:	5.154

Finally, sort the ordinates of the hyetograph into the order of the (2/3, 1/3) distribution. Figure 4-9 of the Hydrology Manual shows the construction of the hyetograph. Table WB.3-2 (in Section WB.3.2.3) shows the ordinates of the hyetograph for this study sorted in the order of the (2/3, 1/3) distribution. The first ordinate (calculated above, the depth-area adjusted incremental rainfall amount for the first time increment) is the peak rainfall ordinate. This peak rainfall ordinate occurs at hour 16.0 of the 24-hour storm. The second rainfall ordinate (calculated above) occurs at 16.0 hours – 1D, the third rainfall ordinate (calculated above) occurs at 16.0 hours – 2D, and the fourth rainfall ordinate (calculated above) occurs at 16.0 hours + 1D. The sequence continues alternating two ordinates to the left and one ordinate to the right (see Figure 4-9 of the Hydrology Manual).

WB.3.2.3 Example Effective Rainfall (Reference Hydrology Manual Sections 4.1.3 and 4.3.3)

Excess rainfall is calculated using equation 4-4.

$$Q_a = \frac{(P - 0.2S)^2}{(P + 0.8S)}$$

where:

$$S = 1000/CN - 10$$

Because equation 4-4 is subject to the limitation, $P \geq 0.2S$, calculation of excess rainfall based on the ordinates of the hyetograph (which are incremental rainfall amounts) will result in underestimation of excess rainfall because the incremental rainfall amounts are small. Excess rainfall must be calculated for a cumulative rainfall series. A cumulative rainfall series is created by summing the ordinates of the hyetograph. This must be performed after the ordinates have been sorted into the (2/3, 1/3) distribution. The last ordinate of the series should be equal to the excess runoff from the depth-area adjusted incremental rainfall amount for duration = 24 hours (5.154 inches for this study). Finally, incremental amounts of excess rainfall are created from the cumulative series. The data resulting from these equations are presented in Table WB.3-2.

Table WB.3-2 (Page 1 of 4)

**NRCS HYDROLOGIC METHOD EXAMPLE #2 CALCULATED EXCESS RAINFALL
 SORTED IN ORDER OF (2/3, 13/) RAINFALL DISTRIBUTION**

Time (minutes)	Hyetograph Ordinate (inches)	Cumulative Rainfall (inches)	Cumulative Excess Rainfall (inches)	Incremental Excess Rainfall (inches)
15	0.024	0.024	0.000	0.000
30	0.024	0.049	0.000	0.000
45	0.025	0.074	0.000	0.000
60	0.025	0.098	0.000	0.000
75	0.025	0.124	0.000	0.000
90	0.025	0.149	0.000	0.000
105	0.026	0.175	0.000	0.000
120	0.026	0.200	0.000	0.000
135	0.026	0.226	0.000	0.000
150	0.026	0.253	0.000	0.000
165	0.027	0.279	0.000	0.000
180	0.027	0.306	0.000	0.000
195	0.027	0.333	0.000	0.000
210	0.027	0.360	0.000	0.000
225	0.028	0.388	0.001	0.001
240	0.028	0.415	0.002	0.001
255	0.028	0.444	0.004	0.002
270	0.028	0.472	0.008	0.003
285	0.029	0.501	0.011	0.004
300	0.029	0.530	0.016	0.005
315	0.029	0.559	0.022	0.005
330	0.030	0.589	0.028	0.006
345	0.030	0.619	0.035	0.007
360	0.030	0.650	0.043	0.008

Table WB.3-2 Continued (Page 2 of 4)

**NRCS HYDROLOGIC METHOD EXAMPLE #2 CALCULATED EXCESS RAINFALL
 SORTED IN ORDER OF (2/3, 13/) RAINFALL DISTRIBUTION**

Time (minutes)	Hyetograph Ordinate (inches)	Cumulative Precipitation (inches)	Cumulative Excess Rainfall (inches)	Incremental Excess Rainfall (inches)
375	0.031	0.681	0.051	0.009
390	0.031	0.712	0.061	0.009
405	0.032	0.744	0.071	0.010
420	0.032	0.776	0.082	0.011
435	0.033	0.809	0.094	0.012
450	0.033	0.842	0.106	0.013
465	0.034	0.876	0.119	0.013
480	0.034	0.910	0.134	0.014
495	0.033	0.943	0.148	0.014
510	0.034	0.977	0.163	0.015
525	0.034	1.011	0.179	0.016
540	0.035	1.046	0.195	0.017
555	0.036	1.081	0.213	0.018
570	0.036	1.118	0.231	0.018
585	0.037	1.155	0.251	0.019
600	0.038	1.193	0.271	0.020
615	0.039	1.232	0.292	0.021
630	0.040	1.271	0.314	0.022
645	0.041	1.312	0.338	0.024
660	0.042	1.354	0.362	0.024
675	0.043	1.397	0.388	0.026
690	0.044	1.442	0.415	0.027
705	0.046	1.488	0.444	0.029
720	0.047	1.535	0.474	0.030

Table WB.3-2 Continued (Page 3 of 4)

**NRCS HYDROLOGIC METHOD EXAMPLE #2 CALCULATED EXCESS RAINFALL
 SORTED IN ORDER OF (2/3, 13/) RAINFALL DISTRIBUTION**

Time (minutes)	Hyetograph Ordinate (inches)	Cumulative Precipitation (inches)	Cumulative Excess Rainfall (inches)	Incremental Excess Rainfall (inches)
735	0.054	1.589	0.509	0.035
750	0.055	1.644	0.545	0.036
765	0.057	1.701	0.584	0.039
780	0.059	1.760	0.624	0.040
795	0.062	1.822	0.667	0.043
810	0.064	1.885	0.712	0.045
825	0.068	1.953	0.761	0.049
840	0.070	2.023	0.812	0.051
855	0.101	2.124	0.887	0.075
870	0.103	2.228	0.966	0.078
885	0.109	2.337	1.050	0.084
900	0.118	2.455	1.143	0.093
915	0.131	2.586	1.247	0.104
930	0.141	2.727	1.362	0.114
945	0.227	2.954	1.550	0.188
960	0.199	3.154	1.718	0.168
975	0.514	3.668	2.163	0.445
990	0.221	3.889	2.359	0.196
1005	0.124	4.013	2.469	0.110
1020	0.106	4.119	2.564	0.095
1035	0.073	4.192	2.630	0.066
1050	0.066	4.257	2.689	0.059
1065	0.060	4.317	2.743	0.054
1080	0.056	4.374	2.794	0.051

Table WB.3-2 Continued (Page 4 of 4)

**NRCS HYDROLOGIC METHOD EXAMPLE #2 CALCULATED EXCESS RAINFALL
 SORTED IN ORDER OF (2/3, 13/) RAINFALL DISTRIBUTION**

Time (minutes)	Hyetograph Ordinate (inches)	Cumulative Precipitation (inches)	Cumulative Excess Rainfall (inches)	Incremental Excess Rainfall (inches)
1095	0.048	4.422	2.838	0.044
1110	0.045	4.467	2.879	0.041
1125	0.042	4.509	2.918	0.039
1140	0.040	4.550	2.954	0.037
1155	0.038	4.588	2.989	0.035
1170	0.037	4.625	3.023	0.034
1185	0.035	4.660	3.055	0.032
1200	0.034	4.694	3.086	0.031
1215	0.035	4.728	3.118	0.032
1230	0.033	4.762	3.149	0.031
1245	0.032	4.794	3.178	0.030
1260	0.032	4.826	3.207	0.029
1275	0.031	4.857	3.236	0.028
1290	0.030	4.887	3.263	0.028
1305	0.029	4.916	3.290	0.027
1320	0.029	4.944	3.317	0.026
1335	0.028	4.972	3.343	0.026
1350	0.027	5.000	3.368	0.025
1365	0.027	5.027	3.393	0.025
1380	0.026	5.053	3.417	0.024
1395	0.026	5.079	3.441	0.024
1410	0.025	5.104	3.465	0.024
1425	0.025	5.129	3.488	0.023
1440	0.025	5.154	3.511	0.023
			Total:	3.511

WB.3.2.4 Example Unit Hydrograph Ordinates (Reference Hydrology Manual Sections 4.1.3 and 4.3.3)

The unit hydrograph ordinates are created based on the T_p and the unit hydrograph q_p for the study area. T_p was calculated above based on Corps lag (see Section WB.3.2.1). The unit hydrograph q_p is then calculated using equation 4-10:

$$q_p = \frac{K_s A Q_a}{T_p}$$

where:

$K_s = 484$, a constant reflecting both the conversion of units and the shape of the hydrograph

$Q_a = 1$ inch of effective runoff

The watershed area for this study is 40.0 square miles and T_p is 1.5 hours. Using equation 4-10:

$$q_p = (484)(40.0)(1.0) / 1.5 = 12,907$$

Use T_p and q_p to set up the unit hydrograph ordinates t/T_p and q/q_p . The time increment (t) for unit hydrograph ordinates must be the same duration as the period of effective rainfall or computation interval (D) selected for the rainfall ordinates. The computation interval for this example is 15 minutes. For multiples of t , t/T_p is computed until $t/T_p = 5$. For each t/T_p , the corresponding q/q_p is found from Table 4-7 of the Hydrology Manual. For values of t/T_p that are not given on Table 4-7 of the Hydrology Manual, the corresponding values of q/q_p are interpolated by linear interpolation from the nearest values from Table 4-7 of the Hydrology Manual (an alternative is to read the values from Figure 4-4 of the Hydrology Manual). Table WB.3-3 presents the unit hydrograph ordinates for this example.

Table WB.3-3

NRCS HYDROLOGIC METHOD EXAMPLE #2 UNIT HYDROGRAPH ORDINATES

Time (minutes)	t/T_p	q/q_p	q (cfs/inch)
15	0.167	0.077	990
30	0.333	0.230	2,969
45	0.500	0.470	6,068
60	0.667	0.767	9,897
75	0.833	0.950	12,263
90	1.000	1.000	12,908
105	1.167	0.950	12,262
120	1.333	0.833	10,755
135	1.500	0.680	8,775
150	1.667	0.493	6,366
165	1.833	0.370	4,775
180	2.000	0.280	3,613
195	2.167	0.219	2,828
210	2.333	0.167	2,155
225	2.500	0.127	1,639
240	2.667	0.097	1,252
255	2.833	0.073	946
270	3.000	0.055	710
285	3.167	0.042	548
300	3.333	0.033	421
315	3.500	0.025	323
330	3.667	0.019	245
345	3.833	0.014	185
360	4.000	0.011	142
375	4.167	0.009	116
390	4.333	0.007	90
405	4.500	0.005	64
420	4.667	0.003	43
435	4.833	0.002	21
450	5.000	0.000	0

WB.3.2.5 Example Convolution of Unit Hydrograph (Reference Hydrology Manual Section 4.3.4)

To perform the NRCS hydrologic method calculation, a table is set up with the unit hydrograph ordinates in rows and incremental excess rainfall ordinates in columns. The table is presented in Appendix WB.B. Convolution of the unit hydrograph is performed as follows:

1. The effective rainfall depth for the first unit time period is multiplied by each unit hydrograph ordinate q to determine the flood hydrograph which would result from that increment of effective rainfall.
2. The above process is repeated for each succeeding effective rainfall depth advancing the resultant flood hydrographs one unit time period for each cycle.
3. The flow ordinates found in the steps above are summed across the rows to determine the average flow ordinates per unit time period for the design storm flood hydrograph.

The average flow ordinates that are found in step 3 are ordinates of the runoff hydrograph. The peak runoff from the watershed for this study is approximately 17,245 cubic feet per second, and occurs at time 1050 minutes (17.5 hours).

WB.3.2.6 Comparison of Results of Hand Computation, SDHydroTools Computer Program and HEC-HMS

The data for this study was also processed using the SDHydroTools computer program, and HEC-HMS to compare the peak runoff output from each method. The SDHydroTools computer program input and output, HEC-HMS input and output, are provided in Appendices WB.C, WB.D, and WB.E, respectively.

HEC-HMS require NRCS lag as the input parameter. NRCS lag is determined based on T_p using the equation 4-20:

$$\text{NRCS } T_l = T_p - D/2$$

For this study, with T_p equal to 1.5 hours and D equal to 15 minutes (0.25 hours):

$$\text{NRCS } T_1 = 1.5 \text{ hours} - (0.25 \text{ hours} / 2) = 1.375 \text{ hours (82.5 minutes)}$$

Table WB.3-4 presents the peak discharge output results of the study based on hand computation, SDHydroTools computer program, and HEC-HMS.

Table WB.3-4

**COMPARISON OF STUDY PEAK DISCHARGE OUTPUT RESULTS FROM
HAND COMPUTATION, SDHYDROTOOLS COMPUTER PROGRAM, AND HEC-HMS**

Study Method	Peak Runoff (cfs)
Hand Computation	17,245
SDHydroTools Computer Program	17,219
HEC-HMS	16,961

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WB.4 WORKBOOK EXAMPLES FOR HYDROLOGY MANUAL SECTION 5 EROSION AND SEDIMENTATION

WB.4.1 EXAMPLE SEDIMENTATION YIELD CALCULATIONS USING THE UNIVERSAL SOIL LOSS EQUATION

(Reference Hydrology Manual Section 5.2)

The project is a 20-acre site located in Fallbrook with a soil classification designation of Sandy Loam (FaC) with 20% natural open space area to remain and 80% to be developed. The average slope of the open space is 16% and the developed area will be 1%. Developed area will be filled with import soil (suitable) and is anticipated to be Las Flores Loamy Fine Sand. The distances traveled for the open space and developed areas are 1,100 feet and 120 feet (respectively).

$$A_s = RKLsCP \text{ (USLE)}$$

R Value

Based on a 2-year, 6-hour storm event $P = 1.3$ inches (interpolated)

Based on Figure 5-2 of the Hydrology Manual, $R = 29.5$

K Factor

Using soil erodibility information from Table 5-2 of the Hydrology Manual, verified against San Diego soil survey and current soils report;

Open space ~ Sandy Loam (FAC) $\underline{K = .28}$

Developed ~ Las Flores Loamy Fine Sand $\underline{K = .15}$

Ls Factor

Based on Figure 5-5 of the Hydrology Manual, open space area is 16% slope and 1,100 feet

$$\underline{Ls = 4.7}$$

Developed area with 1% slope and average T_t for each lot is 120 feet

$$\underline{Ls = 0.13}$$

C Factor

Based on site observation, the open space is generally comprised of waist-high weeds (0.5 meters), with bushes being the same height. The bushes hide approximately 25% of the area, from a bird's-eye perspective. The total vegetative cover is estimated at 60%. There is no apparent grass between the weeds or brush (Table 5-3 of the Hydrology Manual).

$$C = 0.082 \text{ (for open space)}$$

$$C = 1.0 \text{ for developed area during construction (no vegetation)}$$

P Factor

$$P = 1.0$$

No tillage, cross-slope farming or contour strip cropping.

Determine Sedimentation Yield

Open Space:

$$A_s = RKLsCP = (29.5) (.16) (4.7) (.082) (1.0) = 1.82 \text{ tons/acre/year}$$

Soil density estimated @ 116 lb/ft³ for on-site soils

$$\text{Volume}_{os} = 1.82 \text{ tons/acre/year} \times \frac{2000 \text{ pounds}}{\text{ton}} \times \frac{\text{ft}^3}{116 \text{ pounds}} \times 20 \text{ acres (20\%)} = \boxed{125.5 \text{ ft}^3}$$

Developed Area:

$$A_s = \text{RKLsCP} = (29.5) (.15) (.13) (1.0) (1.0) = 0.58 \text{ tons/acre/year}$$

Soil density of import soil estimated @ 123 lb/ft³

$$\text{Volume}_{\text{DA}} = .58 \text{ tons/acre/year} \times \frac{2000 \text{ pounds}}{\text{ton}} \times \frac{\text{ft}^3}{123 \text{ pounds}} \times 20 \text{ acres} (80\%) = \boxed{151.0 \text{ ft}^3}$$

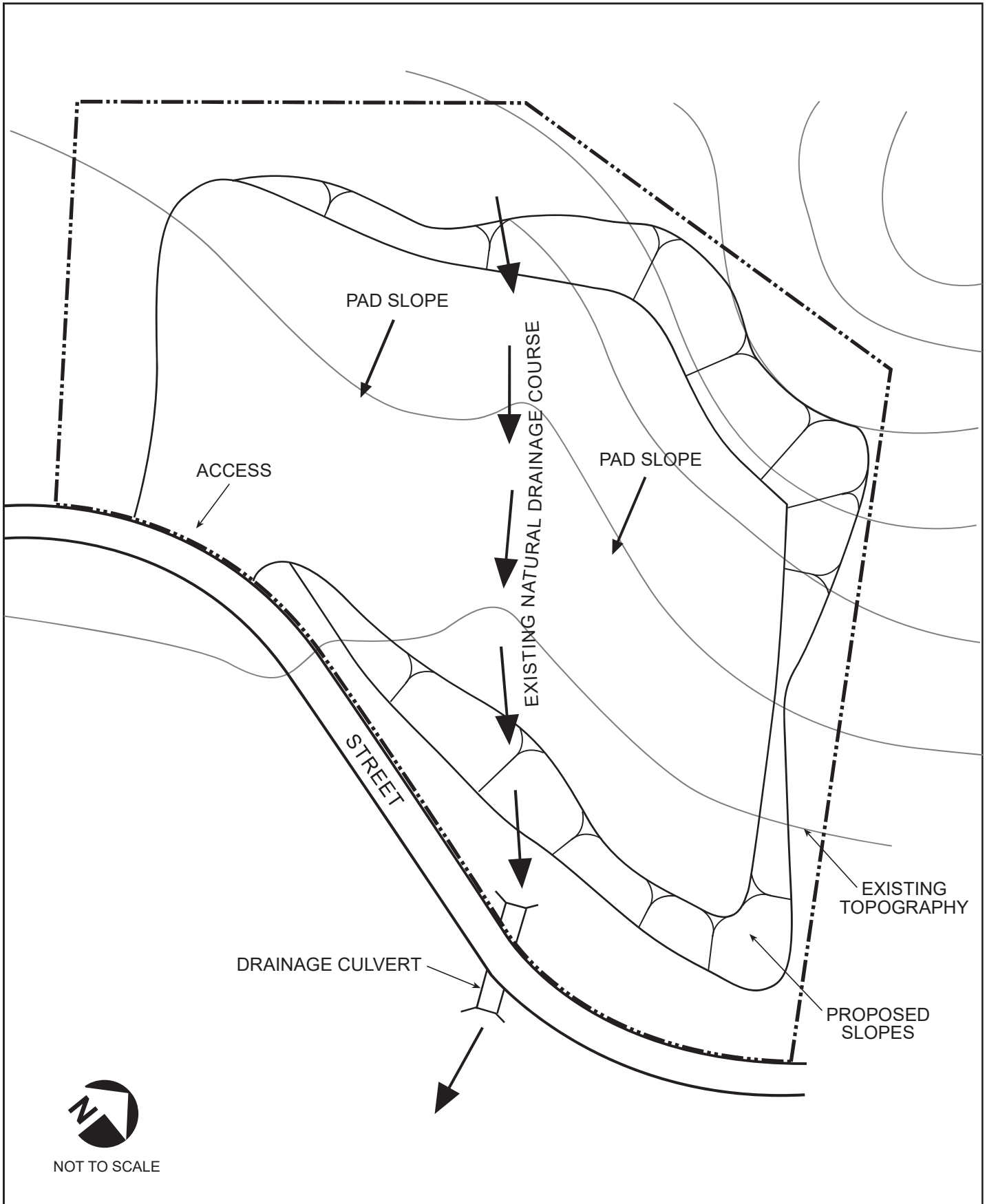
$$\text{Total volume to be captured per year} = \boxed{151 \text{ ft}^3}$$

$$125 + 151 = \boxed{276 \text{ ft}^3 = \text{Volume Total}}$$

WB.4.2 EXAMPLE EROSION CONTROL PLAN (Reference Hydrology Manual Section 5.4)

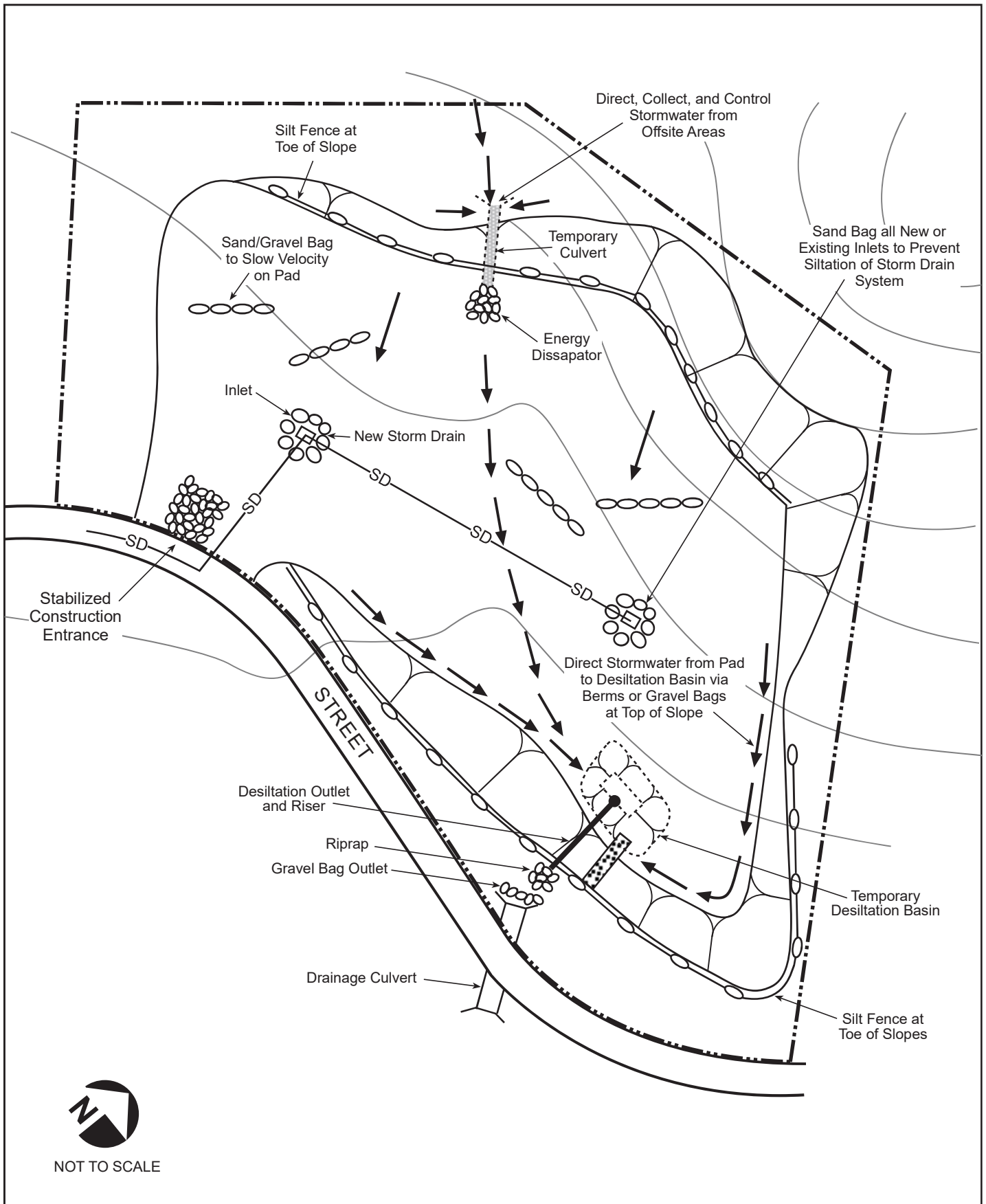
To demonstrate how erosion control devices are prepared, an example project known as “Project A” is provided (see Figure WB.4-1). This project fronts an improved public street and the site will be graded to support a building. The site contains variable slopes that are currently vegetated. The site also includes a natural drainage course crossing the property. Special consideration must be given to the natural drainage course. If earthwork operators disturb the site by removing existing vegetation (scarification) and a rainstorm occurs, the ditch could discharge across the site and erode the site. Stormwater from the natural channel should be collected, conveyed, and discharged across the site during all phases of the project. Based on the size of the project, runoff potential from the drainage course, and proximity to a major culvert crossing, a temporary desiltation basin may be warranted. Berms should be installed at the top of slope to ensure water is not directed over the slopes. Slopes should be planted or hydroseeded after they are manufactured. Energy dissipaters (riprap) should be installed at the end of culverts to slow exiting water to a nonerosive velocity. For an example erosion control plan for “Project A,” refer to Figure WB.4-2.

Erosion control plans are frequently prepared for large developments such as subdivisions. Figure WB.4-3 provides an example of erosion control measures for building pads on residential lots. Please note gravel bags have been installed around the proposed curb inlets to prevent siltation of storm drain infrastructure during construction. The bags installed along the street also slow stormwater to prevent erosion. Construction vehicles can maneuver on the street by weaving around the bags. Make sure that the plan shows the use of straw waddles rather than silt fences.



Example Project "A"

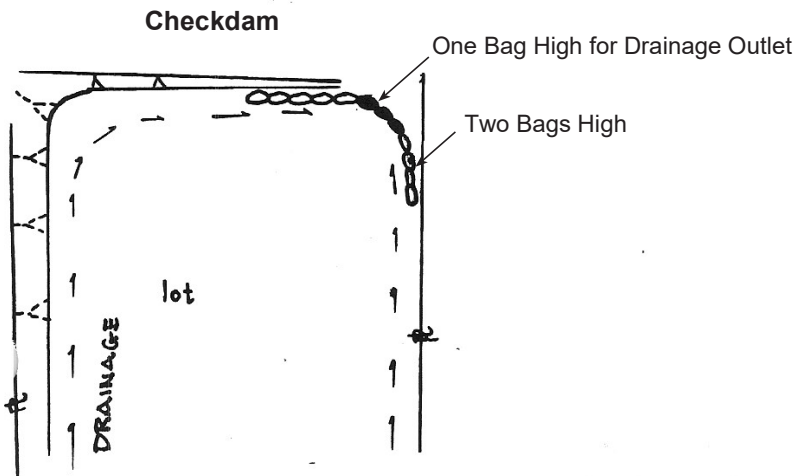
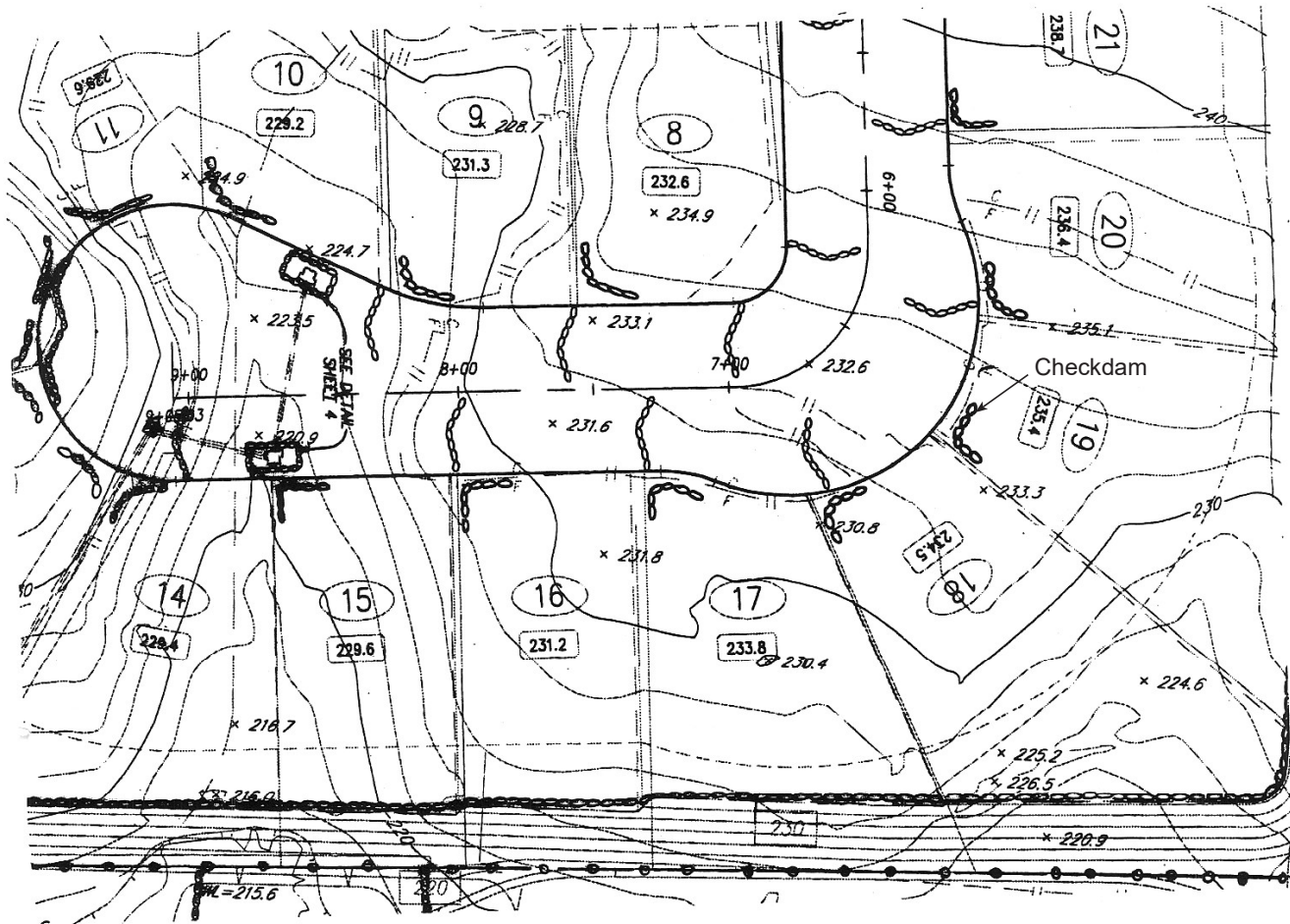
FIGURE
WB. 4-1




 NOT TO SCALE

Erosion Control Plan for Project "A"

FIGURE
WB. 4-2



NOT TO SCALE

Erosion Control Plan for a Subdivision

FIGURE
WB. 4-3

WB.4.3 EXAMPLE SEDIMENT BULKING PROBLEM (Reference Hydrology Manual Section 5.5)

A concrete drainage channel is planned for a hypothetical new development in the project location shown in Figure WB. 4-4, east of Ramona, CA. The project drains an undeveloped, 1-sq-mi watershed and it is not feasible to construct an upstream debris basin. A rainfall-runoff hydrologic model has been developed to compute the design clear-water flows at the project location, including the 100-year clear-water hydrograph, shown in Figure WB. 4-5. Evaluate whether sediment/debris bulking is required for this project, and if so, compute the bulked hydrograph and peak discharge.

Step 1: Review Sediment/Debris Hazard Map to Identify Known Risk

To start, refer to the sediment/debris hazard map in Figure 5-8 to assess whether the project watershed is located within an identified hazard area.

As shown in Figure WB. 4-6, the entire project watershed is located within the hazard area, so sediment/debris loading must be evaluated under normal conditions.

Step 2: Identify Whether Planned Infrastructure is at Risk

Next, evaluate whether the planned infrastructure associated with the project is truly at risk. For this case, the project is a flood control channel in the new development and the adequacy of its design will be directly affected by any flow bulking due to sediment/debris loading. While upstream channel slopes decrease from over 10 percent to approximately 5 percent at the upstream end of the project location, because these are larger than 2 percent, the channel may be affected by debris flows in the upper reaches and must be designed to mitigate these hazards.

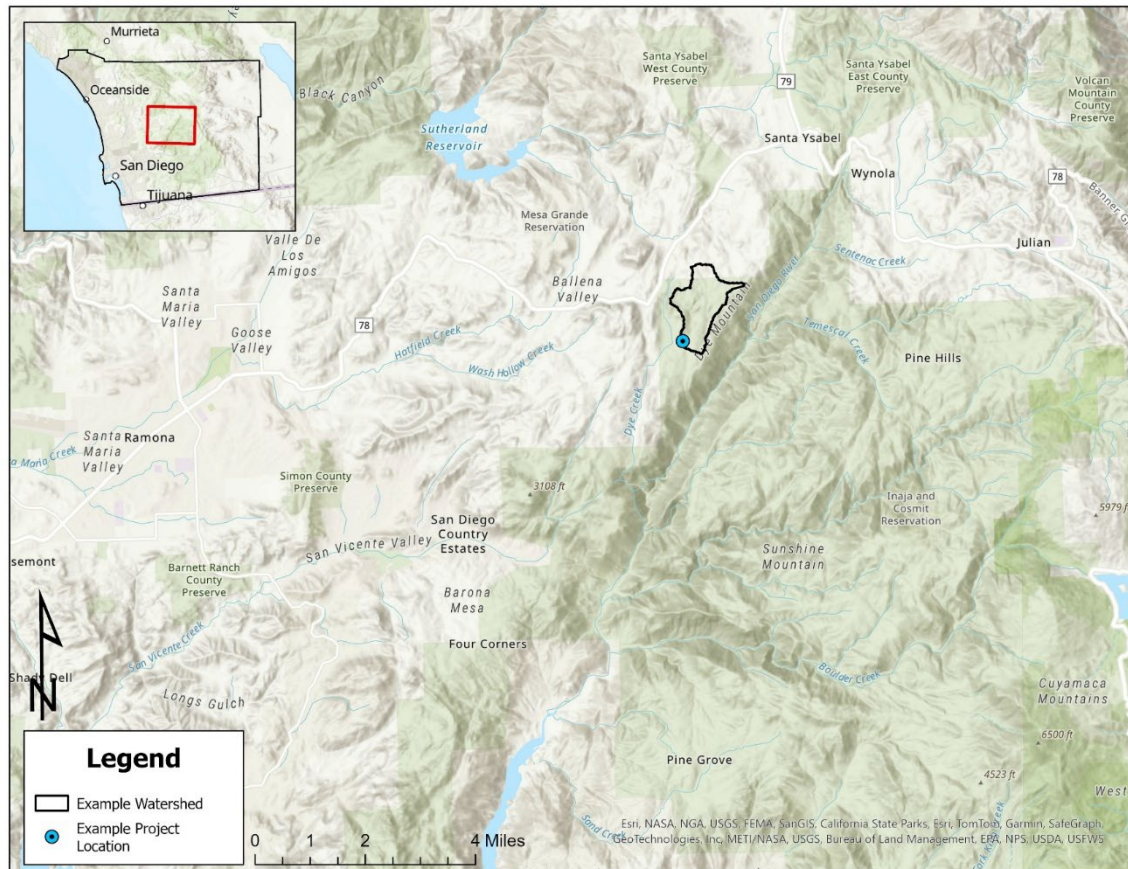


Figure WB. 4-4. Example Project Location and Contributing Watershed

Step 3: Select Appropriate Mitigation Option(s)

As noted in the problem statement, it is not feasible to construct an upstream debris basin to reduce sediment/debris loading at the project location, so the design flow used to size the project channel must be bulked. This may be done in one of two ways:

- Option 1: The 100-year sediment/debris yield can be estimated by performing a coincident frequency analysis, then distributed through the design 100-year clear-water hydrograph.
- Option 2: A standard, conservative bulking factor may be applied without the need for additional analysis.

For this example, the bulked peak flow will be computed using both options.

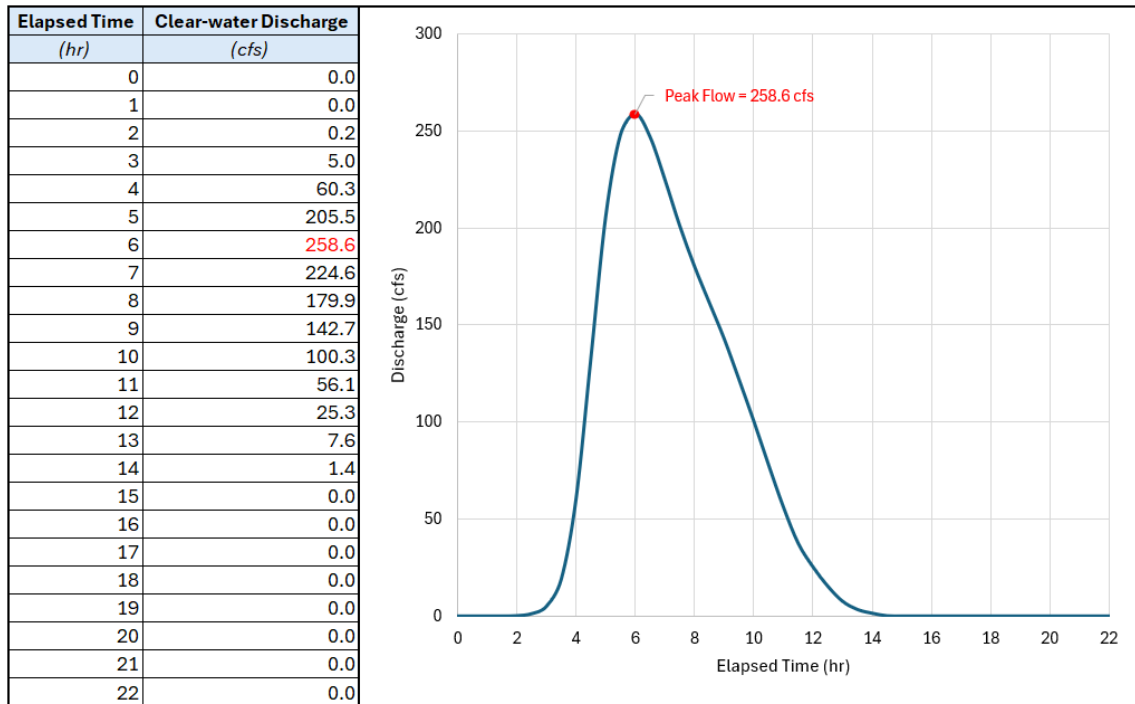


Figure WB. 4-5. Hypothetical 100-Year Clear-Water Hydrograph for Example Project Location

Option 1:

To compute the 100-year sediment/debris, start by preparing the three data inputs required to perform a coincident frequency analysis for fire and precipitation/runoff using methods described in Gatwood et al. (2000), using the USACE Hydrologic Engineering Center - Statistical Software Package (HEC-SSP).ⁱ These include:

- The Fire Factor (FF) vs. Years-Since-100% Wildfire table
- Either the precipitation or discharge frequency table
- Debris response relationship table

ⁱ Available at <https://www.hec.usace.army.mil/software/hec-ssp/>

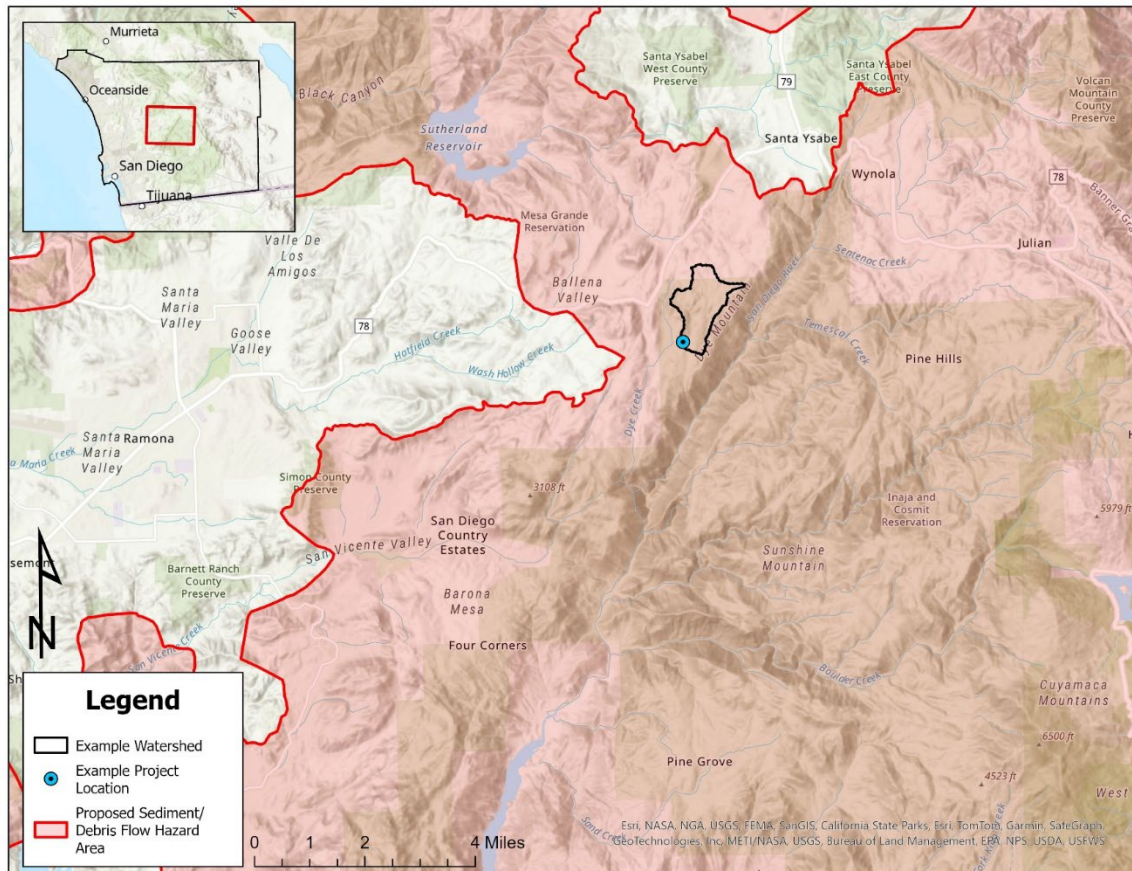


Figure WB. 4-6. Example Project Watershed Located Within the Identified Sediment/Debris Hazard Area

To develop the Fire Factor (FF) vs. Years-Since-100% Wildfire table, start by obtaining the current historical fire perimeter datasetⁱⁱ published annually by the California Department of Forestry and Fire Protection (CAL FIRE) as a geodatabase and open using geographic information system (GIS) software. Clip the historical fire perimeter dataset to the project watershed boundary, then delete perimeters for any pre-1950 fires. Figure WB. 4-7 shows the project watershed with the clipped fire perimeters from 1950-2024. The watershed burned partially or fully four times during this period.

ⁱⁱ Available at <https://www.fire.ca.gov/what-we-do/fire-resource-assessment-program/fire-perimeters>

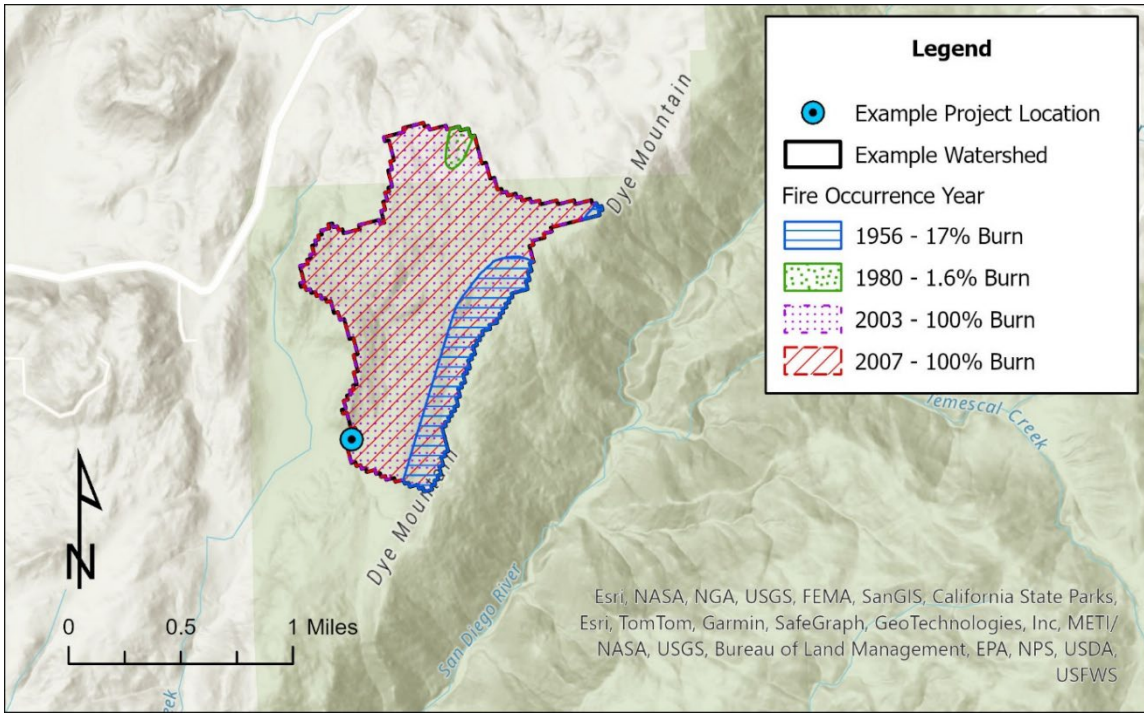


Figure WB. 4-7. Historical 1950-2024 Fire Perimeters, Clipped to Project Watershed

Using GIS, calculate the area of the watershed and each fire boundary and determine the percentage of the project watershed that burned during each fire. Develop a matrix with columns for the fire year, area burned (within the watershed), and percentage of the watershed burned (Table WB. 4-1).

Table WB. 4-1

HISTORICAL FIRE YEARS, AREAS, AND PERCENTAGE OF WATERSHED BURNED

Fire Year <i>(year)</i>	Area <i>(ac)</i>	Portion Burned <i>(Percent)</i>
1956	106.0	17.0
1980	9.8	1.6
2003	622.7	100.0
2007	622.7	100.0

Next, identify the relevant Fire Factor (FF) curve in Gatwood et al. (2000) based on the watershed size:

- Figure A-1 for watersheds 0.1 to 3 sq mi
- Figure A-2 for watersheds 3 to 200 sq mi.

Because the example watershed has an area of 1 sq mi, use Figure A-1 from Gatwood et al. (2000), shown in Figure WB. 4-8.

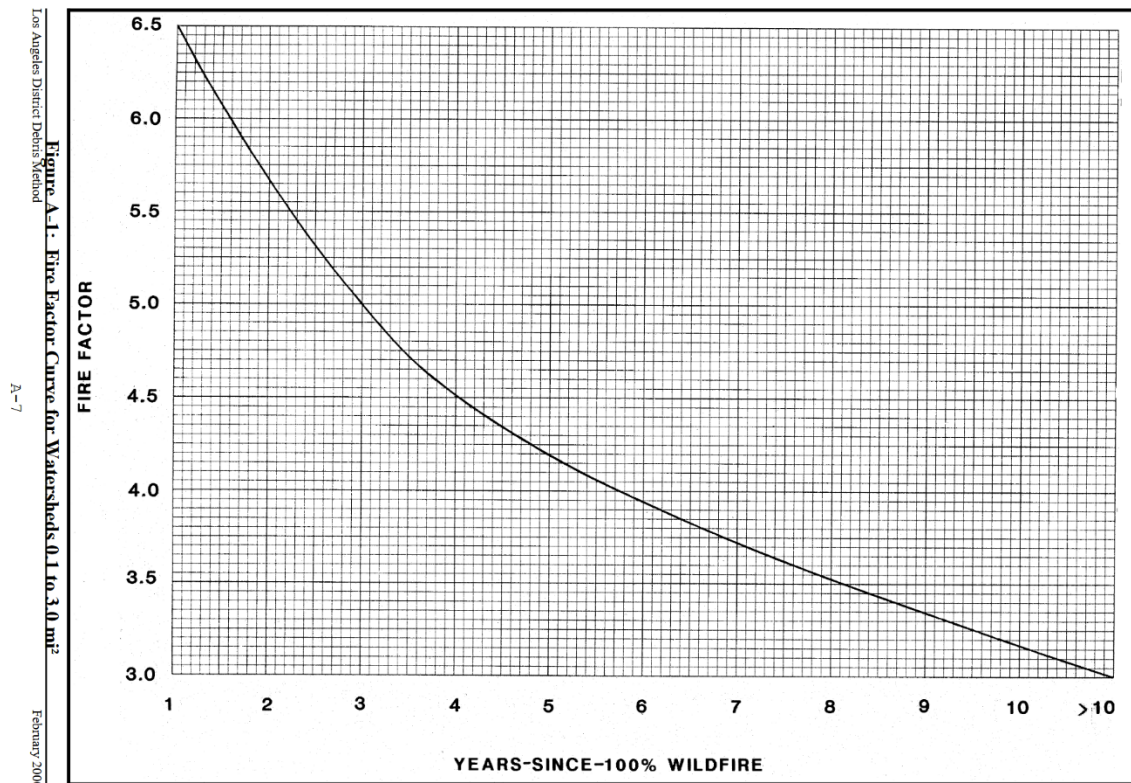


Figure WB. 4-8. Figure A-1 from Gatwood et al. (2000)

Using the figure as a reference, develop a table tracking the watershed’s FF for each year of the fire record (1950-2024) based on these rules:

- Unless the watershed burned in 1950, assume the watershed started fully recovered with an FF of 3.0. Each subsequent year should also have an FF of 3.0 until the year of the first fire.

- Starting the year of the first fire, use the chart to read the FF for a 100-percent burn. (Note that the label on the x-axis is slightly misleading; a value of 1 “Year-Since-100% Wildfire” should be interpreted as the first year, which is the year of the fire.) If the watershed did fully burn, enter that value (here, 6.5) in the table as the FF.
- For partial burns, use an area-weighted average to compute the overall watershed FF. For example, the first fire in the project watershed occurred in 1956 and burned 17 percent of the watershed area. This implies that 83 percent (1-0.17) of the watershed remained unburned. By summing up the products of the FF and watershed percentage for the burned and unburned areas, we compute the average FF for the watershed for 1956:

$$1956 \text{ FF} = 6.5 * 0.17 + 3 * (1 - 0.17) = 3.6.$$

- Follow the same procedures described above for each subsequent year following the fire, reading the 100-percent FF from the curve and weighting as necessary. So, for example, for 1957, the FF for a watershed which burned completely the year prior would be approximately 5.7, so the FF for a watershed with 17 percent burned area would be:

$$1957 \text{ FF} = 5.7 * 0.17 + 3 * (1 - 0.17) = 3.5.$$

- When a new fire occurred before the watershed fully recovered from a previous fire, the location of each fire must be considered and the FF for each burned area accounted for separately. While this is not relevant for the example problem, if the 1980 fire had instead occurred in 1960, then that year, 17 percent would have a “5 years after burn” FF, 1.6 percent would have a “1 year after burn” FF, and the remaining 81.4 percent (1-0.17-0.016) would have an FF of 3, resulting in an average FF of:

$$1960 \text{ FF} = 4.2 * 0.17 + 6.5 * 0.016 + 3 * (1 - 0.17 - 0.016) = 3.26.$$

If a given area burns before it has fully recovered from a previous fire, the FF for that overlapping area is evaluated based only on the most recent fire.

Table WB. 4-2 summarizes the burn percentages and average FF computed for each year during the period of record (1950-2024).

Table WB. 4-2.

EXAMPLE FF HISTORY TABLE FROM 1950 THROUGH 2024

Year	Percent Burned	Avg. FF	Year	Percent Burned	Avg. FF	Year	Percent Burned	Avg. FF
1950	0.0	3.0	1975	0.0	3.0	2000	0.0	3.0
1951	0.0	3.0	1976	0.0	3.0	2001	0.0	3.0
1952	0.0	3.0	1977	0.0	3.0	2002	0.0	3.0
1953	0.0	3.0	1978	0.0	3.0	2003	100.0	6.5
1954	0.0	3.0	1979	0.0	3.0	2004	0.0	5.7
1955	0.0	3.0	1980	1.6	3.1	2005	0.0	5.0
1956	17.0	3.6	1981	0.0	3.0	2006	0.0	4.5
1957	0.0	3.5	1982	0.0	3.0	2007	100.0	6.5
1958	0.0	3.3	1983	0.0	3.0	2008	0.0	5.7
1959	0.0	3.3	1984	0.0	3.0	2009	0.0	5.0
1960	0.0	3.2	1985	0.0	3.0	2010	0.0	4.5
1961	0.0	3.2	1986	0.0	3.0	2011	0.0	4.2
1962	0.0	3.1	1987	0.0	3.0	2012	0.0	3.9
1963	0.0	3.1	1988	0.0	3.0	2013	0.0	3.7
1964	0.0	3.1	1989	0.0	3.0	2014	0.0	3.5
1965	0.0	3.0	1990	0.0	3.0	2015	0.0	3.3
1966	0.0	3.0	1991	0.0	3.0	2016	0.0	3.2
1967	0.0	3.0	1992	0.0	3.0	2017	0.0	3.0
1968	0.0	3.0	1993	0.0	3.0	2018	0.0	3.0
1969	0.0	3.0	1994	0.0	3.0	2019	0.0	3.0
1970	0.0	3.0	1995	0.0	3.0	2020	0.0	3.0
1971	0.0	3.0	1996	0.0	3.0	2021	0.0	3.0
1972	0.0	3.0	1997	0.0	3.0	2022	0.0	3.0
1973	0.0	3.0	1998	0.0	3.0	2023	0.0	3.0
1974	0.0	3.0	1999	0.0	3.0	2024	0.0	3.0

After determining the FF for each year on record, sort and rank the FF from largest to smallest, then use the following formula to assign an exceedance frequency to each FF

$$Exceedance\ Frequency = \frac{m - 0.3}{N + 0.4} * 100$$

where:

m is the rank of the FF value

N is the total number of observations.

Table WB. 4-3 shows the results, for the example watershed, for the 75-year record.

Table WB. 4-3

**RANKED FF AND ASSOCIATED EXCEEDANCE FREQUENCIES FOR
 EXAMPLE WATERSHED**

Rank	FF	Exceedance Freq.	Rank	FF	Exceedance Freq.	Rank	FF	Exceedance Freq.
1	6.5	0.9	26	3.0	34.1	51	3.0	67.2
2	6.5	2.3	27	3.0	35.4	52	3.0	68.6
3	5.7	3.6	28	3.0	36.7	53	3.0	69.9
4	5.7	4.9	29	3.0	38.1	54	3.0	71.2
5	5.0	6.2	30	3.0	39.4	55	3.0	72.5
6	5.0	7.6	31	3.0	40.7	56	3.0	73.9
7	4.5	8.9	32	3.0	42.0	57	3.0	75.2
8	4.5	10.2	33	3.0	43.4	58	3.0	76.5
9	4.2	11.5	34	3.0	44.7	59	3.0	77.9
10	3.9	12.9	35	3.0	46.0	60	3.0	79.2
11	3.7	14.2	36	3.0	47.3	61	3.0	80.5
12	3.6	15.5	37	3.0	48.7	62	3.0	81.8
13	3.5	16.8	38	3.0	50.0	63	3.0	83.2
14	3.5	18.2	39	3.0	51.3	64	3.0	84.5
15	3.3	19.5	40	3.0	52.7	65	3.0	85.8
16	3.3	20.8	41	3.0	54.0	66	3.0	87.1
17	3.3	22.1	42	3.0	55.3	67	3.0	88.5
18	3.2	23.5	43	3.0	56.6	68	3.0	89.8
19	3.2	24.8	44	3.0	58.0	69	3.0	91.1
20	3.2	26.1	45	3.0	59.3	70	3.0	92.4

21	3.1	27.5	46	3.0	60.6	71	3.0	93.8
22	3.1	28.8	47	3.0	61.9	72	3.0	95.1
23	3.1	30.1	48	3.0	63.3	73	3.0	96.4
24	3.1	31.4	49	3.0	64.6	74	3.0	97.7
25	3.0	32.8	50	3.0	65.9	75	3.0	99.1

Derive a fire duration curve for the watershed by plotting the FF values vs. the exceedance frequency values, as shown in Figure WB. 4-9. Add a suitable trendline, extrapolating both ends if necessary, so that the maximum and minimum FF have estimated exceedance frequencies. In the case of the example watershed, because there were complete watershed burns and complete recovery, exceedance frequencies are plotted for the full range of potential FF.

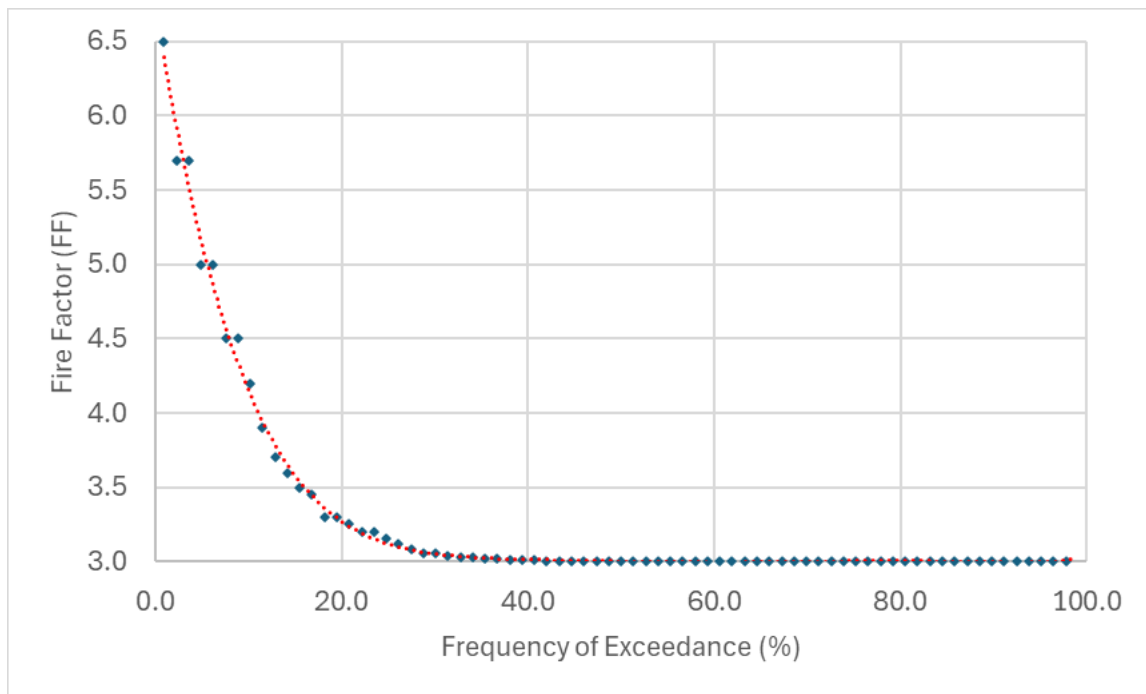


Figure WB. 4-9. Fire Duration Curve Based on FF Exceedance Frequencies

Create a table with a number of rows equal to the number of years required for the watershed to recover (based on the watershed size). Add the following columns:

- Years-Since-100% Wildfire, numbered from 1 through the number corresponding to full recovery (depending on watershed size)
- The associated FF for each year during watershed recovery, read from either Figures A-1 or A-2
- The cumulative frequency percentage for each FF, based on the developed fire duration curve. (This is the frequency for which Years-Since-100% Wildfire is equaled or exceeded.)
- The incremental frequency percentage for each FF

The resulting Fire Factor (FF) vs. Years-Since-100% Wildfire table for the example watershed is shown in Table WB. 4-4; this is the first input for the coincident frequency analysis.

Table WB. 4-4

EXAMPLE FIRE FACTOR FREQUENCY TABLE

Years-Since-100% Wildfire <i>(year)</i>	Fire Factor (FF) <i>(-)</i>	Cumulative Frequency <i>(%)</i>	Incremental Frequency <i>(%)</i>
1	6.5	1.6	1.6
2	5.7	4.2	2.7
3	5.0	6.9	2.7
4	4.5	9.5	2.7
5	4.2	11.5	2.0
6	3.9	12.9	1.3
7	3.7	14.2	1.3
8	3.5	16.8	2.7
9	3.3	20.8	4.0
10	3.2	24.8	4.0
11	3.0	100.0	75.2

Next, to develop the relevant precipitation or discharge frequency table, start by identifying which of the five regression equations presented in Gatwood et al. (2000) and Table WB. 4-5, below, will be used to compute the unit sediment/debris yield. The most important distinction is between that of the equation for the smallest watersheds (Equation 1, for watersheds 0.1 to 3 sq mi) and larger watersheds (Equations 2 through 5). Because the maximum 1-hr precipitation (P) is a good predictor of debris yield for small watersheds

(for which runoff data are not typically available), Equation 1 relies on P while Equations 2 through 5 rely on the unit peak runoff (Q).

Table WB. 4-5

UNIT DEBRIS YIELD EQUATIONS FROM GATWOOD ET AL. (2000)

Equation #	Equation	Watershed Size (mi²)
1	$\text{Log(Dy)} = 0.65 * (\text{Log(P)}) + 0.62 * (\text{Log(RR)}) + 0.18 * (\text{Log(A)}) + 0.12 * (\text{FF})$	0.1 to 3
2	$\text{Log(Dy)} = 0.85 * (\text{Log(Q)}) + 0.53 * (\text{Log(RR)}) + 0.04 * (\text{Log(A)}) + 0.22 * (\text{FF})$	3 to 10
3	$\text{Log(Dy)} = 0.88 * (\text{Log(Q)}) + 0.48 * (\text{Log(RR)}) + 0.06 * (\text{Log(A)}) + 0.2 * (\text{FF})$	10 to 25
4	$\text{Log(Dy)} = 0.94 * (\text{Log(Q)}) + 0.32 * (\text{Log(RR)}) + 0.14 * (\text{Log(A)}) + 0.17 * (\text{FF})$	25 to 50
5	$\text{Log(Dy)} = 1.02 * (\text{Log(Q)}) + 0.23 * (\text{Log(RR)}) + 0.16 * (\text{Log(A)}) + 0.13 * (\text{FF})$	50 to 200

where:

P is the max 1-hr precipitation (inches, to two places after the decimal, times 100)

Q is the unit peak runoff (cfs / sq mi)

RR is the relief ratio (drainage slope, ft / mi)

A is drainage area (ac)

FF is the non-dimensional fire factor (-)

Dy is the unit debris yield (cu yd / sq mi)

The example watershed is 1 sq mi, so Equation 1 is selected. Obtain 1-hr maximum precipitation data from an appropriate source; generally, Atlas 14 precipitation frequency estimates from the National Oceanic and Atmospheric Administration (NOAA) Precipitation Frequency Data Server (PFDS)ⁱⁱⁱ should be used. Table WB. 4-6 presents these data for the example watershed; this is the second data input required for the coincident frequency analysis.

ⁱⁱⁱ Available at https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=ca

Table WB. 4-6

WATERSHED 1-HR PRECIPITATION FREQUENCY RELATIONSHIP

Recurrence Interval	Annual Exceedance Probability	Max 1-hr Precipitation
<i>(year)</i>	<i>(%)</i>	<i>(in*100)</i>
500	0.2	270
200	0.5	224
100	1.0	193
50	2.0	166
25	4.0	141
10	10.0	112
5	20.0	92
2	50.0	69
1	100.0	53

Next, develop a debris response relationship table using representative ranges of FF values and P (or Q) values, the selected equation, and other watershed variables to compute the unit sediment/debris yield for representative combinations of precipitation and FF.

For the example watershed, use these values:

Relief Ratio (RR, ft / mi) = 498.3 (computed using GIS)

Drainage Area, (A, ac) = 640

The resulting matrix, computed using Equation 1 for the example watershed, is shown in Table WB. 4-7. This is the third required data input for the coincident frequency analysis.

Table WB. 4-7
DEBRIS YIELD MATRIX FOR COINCIDENT FREQUENCY ANALYSIS

Maximum 1-hr Precipitation (in*100)	Years-Since-100% Wildfire										
	1	2	3	4	5	6	7	8	9	10	11
	Fire Factor (FF) after N-years since 100% Wildfire										
	6.5	5.7	5.0	4.5	4.2	3.9	3.7	3.5	3.3	3.2	3.0
Unit Sediment/Debris Yield (cu yd / sq mi)											
270	34,509	27,665	22,800	19,858	18,278	16,824	15,920	15,064	14,254	13,865	13,120
224	30,564	24,502	20,193	17,588	16,188	14,901	14,100	13,342	12,624	12,280	11,620
193	27,743	22,241	18,330	15,965	14,695	13,526	12,798	12,110	11,459	11,147	10,548
166	25,154	20,166	16,619	14,475	13,323	12,263	11,604	10,980	10,390	10,107	9,563
141	22,622	18,136	14,946	13,018	11,982	11,029	10,436	9,875	9,344	9,089	8,601
112	19,478	15,615	12,869	11,208	10,317	9,496	8,985	8,502	8,045	7,826	7,405
92	17,140	13,741	11,324	9,863	9,078	8,356	7,907	7,482	7,080	6,887	6,516
69	14,217	11,397	9,393	8,181	7,530	6,931	6,558	6,206	5,872	5,712	5,405
53	11,976	9,601	7,913	6,892	6,343	5,839	5,525	5,228	4,947	4,812	4,553

Once all required data inputs are prepared, perform the coincident frequency analysis in HEC-SSP. Create a new study and create a new Coincident Frequency Analysis.

Under the “General” tab, set Variable A and B as independent. Set the number of index values for Variable B as equal to the number of FF in Table WB. 4-4 (i.e., 11, for the example watershed). Set the output label as “Unit Debris Yield” and the units to “cu yd / sq mi.” Set the output frequency ordinates equal to the precipitation frequency curve values. Note that HEC-SSP cannot handle a 100-percent frequency for the 1-year precipitation, but a value of 99.99 can be entered as an approximation.

Under the “Variable A” tab, label the data “1-hr Precipitation” and set the units. Then, insert the corresponding 1-hr precipitation rates from Table WB. 4-6 to the exceedance probabilities set in the “General” tab.

Under the “Variable B” tab, input the incremental exceedance probabilities for each FF value determined in Table WB. 4-4. Then, input the FF values in the index column. Ensure that the probabilities sum to a value of 100. (Note that incremental values shown in Table WB. 4-4 are rounded.)

Under the “Response Curves” tab, elect to use the same Variable A for each index, then import Variable A. Copy and paste the debris yield matrix values from Table WB. 4-7. Click “Plot Response Surface” to ensure that the debris yield curves were input correctly.

Under the “Results” tab, click “Compute.” The debris yield exceedance curve is plotted, as shown in Figure WB. 4-10. Check for any error messages, then identify the 100-year unit sediment/debris yield by viewing the table in the “Results” tab.

For the example watershed, the computed 100-year unit sediment/debris yield is 15,455 cu yd / sq mi.

Because the example watershed is in the Peninsular Ranges, which are generally less erosive and have less sediment/debris yield potential than the watersheds of the Transverse

Ranges from which the regression equations were developed, an Adjustment-Transposition (A-T) Factor must be used to determine the adjusted unit sediment/debris yield.

Appendix B of Gatwood et al. (2000) provides four techniques for determining an appropriate A-T Factor. In this case, assume that an A-T Factor of 0.9 is recommended by a qualified geologist or engineer, based on a desk study, field observations, and the provided guide in Table B-1 of Gatwood et al. (2000). Multiplying this by the computed 100-year unit sediment/debris yield results in an adjusted unit sediment/debris yield of 13,910 cu yd / sq mi.

Given the size of the watershed (1 sq mi), the adjusted 100-year sediment/debris yield is 13,910 cu yd.

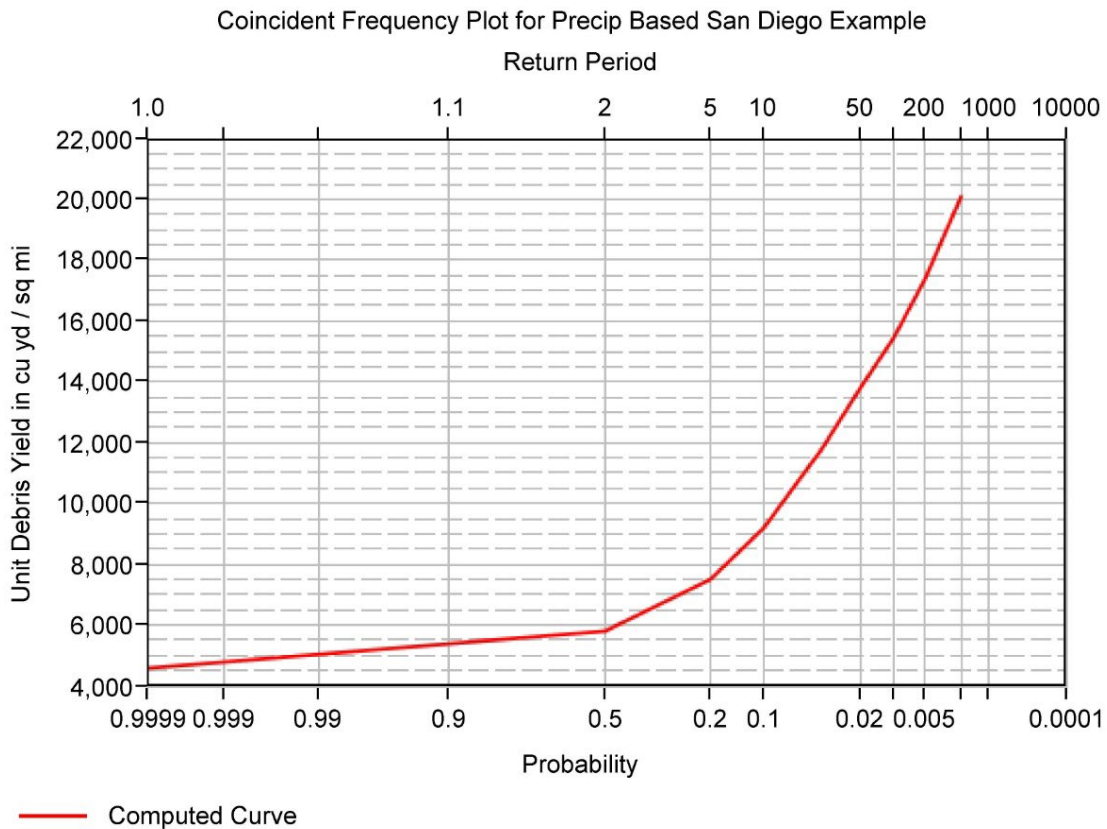


Figure WB. 4-10. Coincident Frequency Plot from HEC-SSP with Unit Debris Yield Probabilities

Next, distribute the adjusted 100-year sediment/debris yield through the 100-year clear-water hydrograph using Equation 3.1 in the 2022 Bulking Factor Technical Study Report, which is available for download from the County of San Diego Department of Public Works Flood Control website:

$$Q_s = a(Q_w)^n$$

where:

Q_s is the sediment discharge (cfs)

Q_w is the clear-water discharge (cfs), and

a and n are a bulking constant and exponent, respectively, and are fixed throughout the hydrograph.

The bulking exponent, n , is taken as 3, and Equation 3.1 is applied at each timestep to compute the sediment/debris discharge and develop a sediment/debris discharge time series equivalent to a hydrograph. Note that the integral of the sediment “hydrograph” represents the computed total sediment/debris yield for the flood event. The bulking constant, a , must be selected such that this computed total sediment/debris yield is equal to the adjusted 100-year sediment/debris yield assessed in the coincident frequency analysis (i.e., 13,910 cu yd).

Figure WB. 4-11 shows the result, where the sediment discharge peaks at 38 cfs and this, combined with the clear-water peak, results in a computed bulked peak flow of 297 cfs.

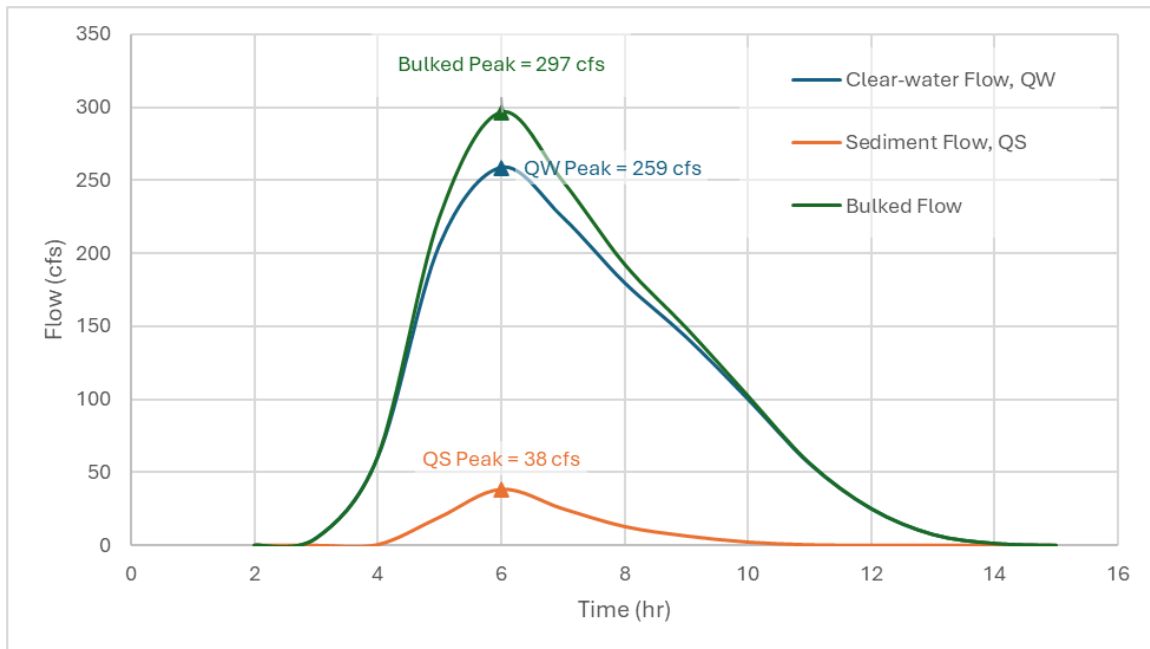


Figure WB. 4-11. 100-Year Sediment/Debris Yield Distributed Through Design Clear-Water Hydrograph to Produce Computed Bulked Hydrograph

Dividing the bulked peak flow by the clear-water peak flow results in a computed bulking factor of 1.15. Based on guidance in the 2022 Bulking Factor Technical Study Report, which is available for download from the County of San Diego Department of Public Works Flood Control website, in locations where bulking is warranted, the computed bulking factor must be at minimum 1.2 and maximum 1.5. Thus, a bulking factor of 1.2 is assumed. This produces a peak bulked flow of 310 cfs, as shown in Figure WB. 4-12.

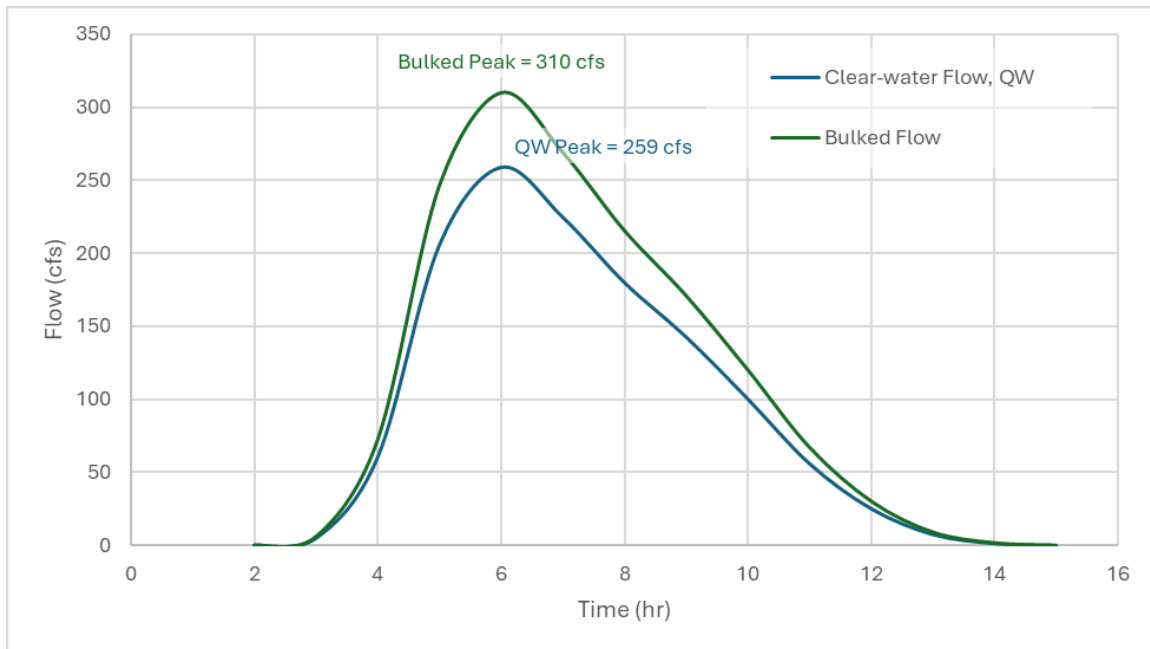


Figure WB. 4-12. 100-Year Bulked Hydrograph, Computed Based on Clear-Water Hydrograph and Minimum Bulking Factor of 1.2

Option 2:

In lieu of performing the detailed analysis described above, users can instead opt to apply a conservative bulking factor of 1.5 to the watershed, based on its size. As shown in Figure WB. 4-13, this produces a peak bulked flow of 388 cfs.

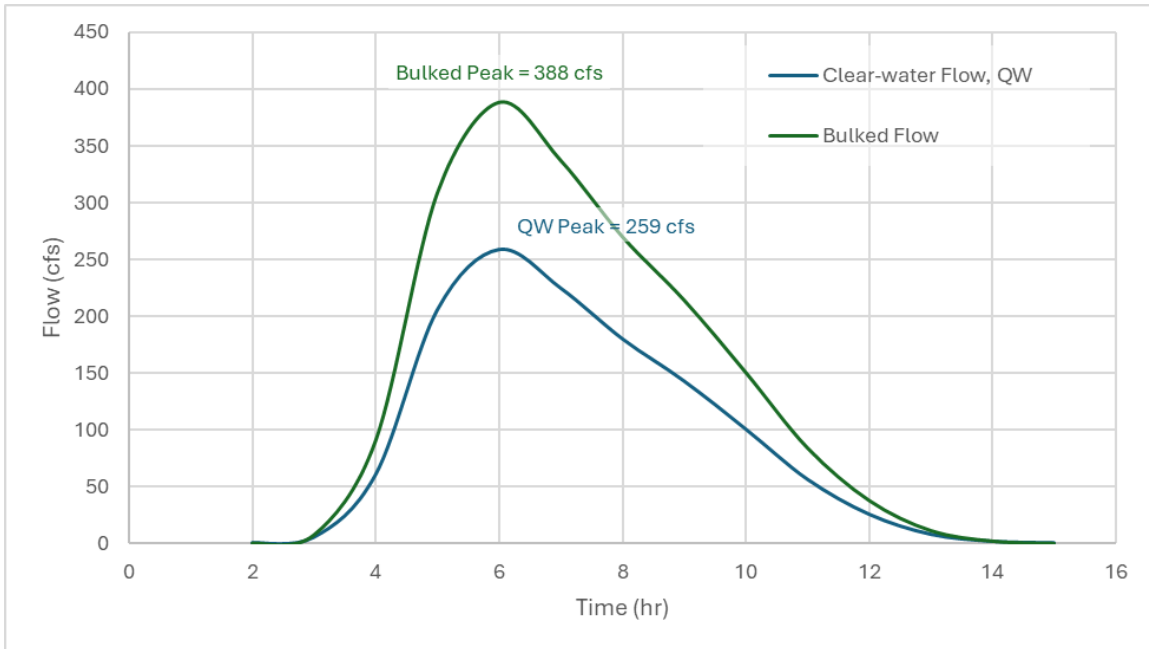


Figure WB. 4-13. 100-Year Bulked Hydrograph, Computed Based on Clear-Water Hydrograph and Conservative Bulking Factor of 1.5

Note that in this case, applying Option 2 results in a design peak bulked flow which is 25 percent larger than that selected using Option 1; this illustrates the potential reduction in design flow that can be justified based on a more-detailed analysis. Option 2 will typically result in a comparatively large bulking factor to account for the larger uncertainty if the sediment/debris yield is not evaluated explicitly.

In this case, the bulked hydrograph shown in Figure WB. 4-12 (based on Option 1) is used to size the project channel.

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WB.5 WORKBOOK EXAMPLE FOR HYDROLOGY MANUAL SECTION 6 RATIONAL METHOD HYDROGRAPH PROCEDURE

WB.5.1 EXAMPLE HYDROGRAPH DEVELOPMENT (Reference Hydrology Manual Section 6.2)

The following example demonstrates the development of a hydrograph from Rational Method (RM) study results using the procedure presented in Section 6 of the Hydrology Manual. The example drainage area is a 10-acre drainage area with a runoff coefficient of 0.80. A RM study of the drainage area was performed for the 100-year storm event. The following are the 100-year precipitation depth data pairs obtained from NOAA Atlas 14 for this example study:

$$P_{5\text{-min}} = 0.406 \text{ inches}$$

$$P_{10\text{-min}} = 0.582 \text{ inches}$$

$$P_{15\text{-min}} = 0.704 \text{ inches}$$

$$P_{30\text{-min}} = 0.977 \text{ inches}$$

$$P_{60\text{-min}} = 1.40 \text{ inches}$$

$$P_{120\text{-min}} = 1.92 \text{ inches}$$

$$P_{180\text{-min}} = 2.29 \text{ inches}$$

$$P_{360\text{-min}} = 3.02 \text{ inches}$$

The RM study results are as follows:

$$\text{Time of concentration } (T_c) = 9.8 \text{ minutes}$$

$$\text{Peak discharge } (Q) = 28.2 \text{ cfs}$$

The first step of the rational method hydrograph procedure is to prepare the incremental rainfall blocks. The number of rainfall blocks (N) in the rainfall distribution is determined by rounding the time of concentration to the nearest minute and then dividing the duration of the storm (360 minutes) by the rounded T_c :

$$\text{Time of concentration } (T_c) = 9.8 \text{ minutes} = 10 \text{ minutes}$$

$$N = 360 / 10 = 36$$

The total duration of each block from 1 to N (1 to 36), $T_{T(N)}$, is equal to NT_c .

$$T_{T(N)} = N T_c$$

The total, or cumulative, rainfall depth $P_{T(N)}$ for each rainfall block at duration $T_{T(N)}$ is obtained from NOAA Atlas 14 precipitation depth data, using log-log interpolation to determine the depth when $T_{T(N)}$ is between durations provided in NOAA Atlas 14.

The incremental rainfall depth, $P_{(N)}$, for each rainfall block is the incremental increase in total rainfall that occurs during the time step, and is calculated by taking the difference between the total rainfall at the current time $T_{T(N)}$ and the previous time $T_{T(N-1)}$.

$$P_{(N)} = P_{T(N)} - P_{T(N-1)}$$

The data resulting from these equations are presented in Table WB.5-1. These data are calculated for the rainfall blocks sorted in order from $N = 1$ to $N = 36$, and are shown in that order in Table WB.5-1.

Table WB.5-1

**EXAMPLE CALCULATED DATA WITH
RAINFALL BLOCKS IN NUMERICAL ORDER**

N	T_{T(N)} (minutes)	P_{T(N)} (inches)	P_N (inches)
1	10	0.582	0.582
2	20	0.807	0.225
3	30	0.977	0.170
4	40	1.134	0.157
5	50	1.274	0.139
6	60	1.400	0.126
7	70	1.502	0.102
8	80	1.596	0.094
9	90	1.684	0.088
10	100	1.767	0.083
11	110	1.845	0.078
12	120	1.920	0.075
13	130	1.988	0.068
14	140	2.053	0.065
15	150	2.116	0.062
16	160	2.176	0.060
17	170	2.234	0.058
18	180	2.290	0.056
19	190	2.340	0.050
20	200	2.388	0.048
21	210	2.435	0.047
22	220	2.481	0.046
23	230	2.525	0.044
24	240	2.569	0.043
25	250	2.611	0.042
26	260	2.652	0.041
27	270	2.692	0.040
28	280	2.732	0.039

Table WB.5-1 (Continued)

**EXAMPLE CALCULATED DATA WITH
RAINFALL BLOCKS IN NUMERICAL ORDER**

N	T_{T(N)} (minutes)	P_{T(N)} (inches)	P_N (inches)
29	290	2.770	0.039
30	300	2.808	0.038
31	310	2.845	0.037
32	320	2.881	0.036
33	330	2.917	0.036
34	340	2.952	0.035
35	350	2.986	0.034
36	360	3.020	0.034

For the rainfall distribution, the rainfall at block $N = 1$, ($1T_c$), is centered at 4 hours, the rainfall at block $N = 2$, ($2T_c$), is centered at 4 hours $- 1T_c$, the rainfall at block $N = 3$, ($3T_c$), is centered at 4 hours $- 2T_c$, and the rainfall at at block $N = 4$, ($4T_c$), is centered at 4 hours $+ 1T_c$. The sequence continues alternating two blocks to the left and one block to the right (see Figure 6-1 of the Hydrology Manual). Table WB.5-2 shows the rainfall data, P_N , arranged in the order of the rainfall distribution. The $P_{T(N)}$ data are not shown in Table WB.5-2 because calculation of $P_{T(N)}$ is an intermediate step in the process that is only necessary for calculation of $P_{(N)}$. The calculated intensity data and peak flow data are added to the table.

$I_{(N)}$ is equal to the incremental rainfall depth, $P_{(N)}$, determined for the rainfall block divided by the incremental duration of the rainfall block. All rainfall blocks from 1 to N have the same incremental duration, the rounded study T_c .

$$I_{(N)} = (P_{(N)} / T_c) (60 \text{ minutes} / \text{hour})$$

The peak flow rate is calculated using the intensity for each rainfall block, the total study drainage area, and the composite C value for the study drainage area in the rational method equation.

$$Q_{(N)} = C I_{(N)} A$$

The time that corresponds to the peak discharge is added to the Table WB.5-2. The time corresponding to the peak discharge for block $N = 1$ is 4 hours $+ \frac{1}{2}T_c$, the time corresponding to the peak discharge for block $N = 2$ is 4 hours $- \frac{1}{2}T_c$, the time corresponding to the peak discharge for block $N = 3$ is 4 hours $- 1\frac{1}{2}T_c$, and the time corresponding to the peak discharge for block $N = 4$ is 4 hours $+ 1\frac{1}{2}T_c$. Note that with rainfall block $N = 1$ being centered at 4 hours (240 minutes), the start of that rainfall block occurs at 4 hours (240 minutes) $- \frac{1}{2}T_c$ and the end of that rainfall block occurs at 4 hours (240 minutes) $+ \frac{1}{2}T_c$. As discussed in more detail and shown graphically in Section 6.2.4, this results in the overall hydrograph peak flow rate occurring at 4 hours (240 minutes) $+ \frac{1}{2}T_c$. In order to present the rainfall blocks in tabular format (i.e., one point in time associated with the rainfall block), the rainfall block for $N = 1$ is shown at 4 hours (240

minutes) + $\frac{1}{2}T_c$, which is the end of the rainfall block, and this time is referred to as the hydrograph peak flow rate time (T_p). The rainfall block for $N = 2$ is shown at $T_p - T_c$, the rainfall block for $N = 3$ is shown at $T_p - 2T_c$, the rainfall block for $N = 4$ is shown at $T_p + T_c$, and so forth. This is the reason that the times corresponding to the peak flow rates are not the same as the durations associated with each rainfall block in the first step of the process in which the rainfall blocks are calculated. Finally, because the hydrograph must begin and end with zero, start and end values of zero have been inserted into the table for time equal to zero (start time) and time equal to 6 hours + $\frac{1}{2}T_c$ (end time).

The values of $Q_{(N)}$ in Table WB.5-2 are the ordinates of the overall hydrograph. Due to rounding the time of concentration to the nearest minute to create the rainfall blocks, the peak flow rate shown in the hydrograph is 27.9 cfs, which is slightly different from the study peak flow rate of 28.2 cfs. The actual study peak flow rate of 28.2 cfs may be substituted for the calculated flow rate for the peak ordinate (associated to rainfall block $N = 1$) for consistency with the rational method study. The total volume under the hydrograph is equal to the following equation:

$$VOL = C P_6 A$$

The total volume under the example hydrograph in acre-feet is:

$$VOL = (0.80)(3.02 \text{ inches})(1 \text{ foot} / 12 \text{ inches}) (10.0 \text{ acres}) = 2 \text{ acre-feet}$$

Table WB.5-2 (Continued)

**EXAMPLE CALCULATED DATA ARRANGED BASED
 ON THE (2/3, 1/3) RAINFALL DISTRIBUTION**

N	P_N (inches)	I_N (in / hour)	Q_N (cfs)	Time (min)	Time (hours)
-	0.000	0.000	0.0	0	0
36	0.034	0.203	1.6	5	0.08
35	0.034	0.206	1.6	15	0.25
33	0.036	0.214	1.7	25	0.42
32	0.036	0.218	1.7	35	0.58
30	0.038	0.226	1.8	45	0.75
29	0.039	0.231	1.8	55	0.92
27	0.040	0.242	1.9	65	1.08
26	0.041	0.247	2.0	75	1.25
24	0.043	0.260	2.1	85	1.42
23	0.044	0.267	2.1	95	1.58
21	0.047	0.282	2.3	105	1.75
20	0.048	0.290	2.3	115	1.92
18	0.056	0.337	2.7	125	2.08
17	0.058	0.349	2.8	135	2.25
15	0.062	0.375	3.0	145	2.42
14	0.065	0.390	3.1	155	2.58
12	0.075	0.448	3.6	165	2.75
11	0.078	0.471	3.8	175	2.92
9	0.088	0.528	4.2	185	3.08
8	0.094	0.565	4.5	195	3.25
6	0.126	0.758	6.1	205	3.42
5	0.139	0.836	6.7	215	3.58
3	0.170	1.023	8.2	225	3.75

Table WB.5-2 (Continued)

**EXAMPLE CALCULATED DATA ARRANGED BASED
ON THE (2/3, 1/3) RAINFALL DISTRIBUTION**

N	P_N (inches)	I_N (in / hour)	Q_N (cfs)	Time (min)	Time (hours)
2	0.225	1.347	10.8	235	3.92
1	0.582	3.492	27.9	245	4.08
4	0.157	0.944	7.6	255	4.25
7	0.102	0.611	4.9	265	4.42
10	0.083	0.497	4.0	275	4.58
13	0.068	0.408	3.3	285	4.75
16	0.060	0.361	2.9	295	4.92
19	0.050	0.300	2.4	305	5.08
22	0.046	0.274	2.2	315	5.25
25	0.042	0.253	2.0	325	5.42
28	0.039	0.236	1.9	335	5.58
31	0.037	0.222	1.8	345	5.75
34	0.035	0.210	1.7	355	5.92
-	0.000	0.000	0.0	365	6.08

APPENDIX WB.A
SDHYDROTOOLS COMPUTER PROGRAM INPUT AND OUTPUT FOR
NRCS HYDROLOGIC METHOD EXAMPLE #1

Hydro Tools

Project Interpolation RickRatHydro Nested Storm Convolution SD Unit Hydrograph

- Home
- Location
- Rainfall Data

INSTRUCTIONS

Location may be entered manually or by adding the location on the map. This step is required for documentation of the project location and to activate the Rainfall Data Window, even when rainfall data will be entered manually. Enter one location per study (i.e., the program performs calculations for one location at a time).

For manual location input, enter latitude and longitude, then click 'verify' and the location will be shown on the map.

For map entry click, the location icon to start or stop adding a point. The location icon will appear green when enabled. With the location icon enabled, select the desired location on the map. Latitude and longitude will be updated automatically.

MANUAL LOCATION INPUT

Latitude (Decimal Degrees) Longitude (Decimal Degrees, include "-" for west)

- OR -

ADD LOCATION ON MAP

SDHydroTools | Current Path: > Project > Location

Hydro Tools

Project Interpolation RickRatHydro Nested Storm Convolution SD Unit Hydrograph

Home
Location
Rainfall Data

PRECIPITATION DEPTH

5-Minute 0.366	2-Hour 2.014
10-Minute 0.525	3-Hour 2.431
15-Minute 0.634	6-Hour 3.418
30-Minute 0.925	12-Hour 4.929
60-Minute 1.456	24-Hour 6.794

NOAA ATLAS 14 DATA DESCRIPTION

Data Type: Precipitation Depth
Units: English
Time Series Type: Partial Duration

The program includes a built-in data set of 100-year recurrence interval precipitation data. Data accessed by the program appears in green boxes.

Users may overwrite the data accessed by the program with data for other recurrence intervals in order to use the program computational tools for other recurrence intervals.

User-entered data must be the type shown above. User-entered data will appear in red boxes.

NEXT

SDHydroTools | Current Path: > Project > Rainfall Data

Hydro Tools

Project Interpolation RickRatHydro Nested Storm Convolution **SD Unit Hydrograph**

Home Table

SDUH INPUT

Computation Interval (D) (min)
D should be $\leq 0.2T_p$
5

Watershed Area (mi²)
3.0

Adjusted Curve Number for Watershed Area
84

Watershed Corps Lag Time (hrs)
0.94

RESULTS SUMMARY

Export

ATTRIBUTE	VALUE	UNIT
-----------	-------	------

CALCULATE

SDHydroTools | Current Path: > SD Unit Hydrograph > Home

Hydro Tools

Project Interpolation RickRatHydro Nested Storm Convolution **SD Unit Hydrograph**

Home Table

SDUH INPUT

Computation Interval (D) (min)
D should be $\leq 0.2T_p$

Watershed Area (mi²)

Adjusted Curve Number for Watershed Area

Watershed Corps Lag Time (hrs)

5 3.0

84 0.94

RESULTS SUMMARY

Export

ATTRIBUTE	VALUE	UNIT
Study/Project Title Information	WB.3.1.5	
Latitude	33.2077	
Longitude	-116.9833	
Study Recurrence Interval	100-year	
Total Rainfall Depth (24-Hour Rainfall)	6.79	inch
Area Adjusted Rainfall Depth	6.75	inch
Watershed Area	3.0	mi ²
Adjusted Curve Number for Watershed Area	84	
Watershed Corps Lag Time	0.94	hours
Computation Interval (D)	5	min
Peak Flow Rate	2177.35	cfs
Time Ran	11/17/2020 9:25:03 AM	

CALCULATE

SDHydroTools | Current Path: > SD Unit Hydrograph > Home

Hydro Tools

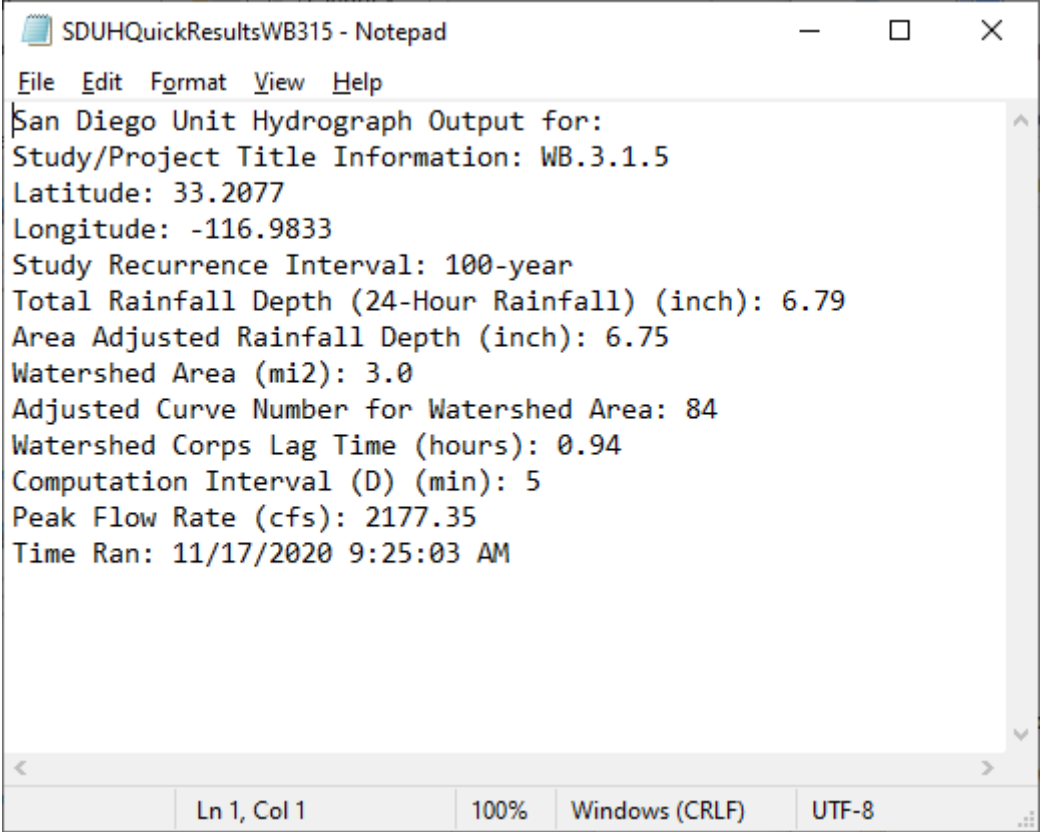
Project Interpolation RickRatHydro Nested Storm Convolution SD Unit Hydrograph

SDUH Table Unit Hydrograph UH Time

Export

CUMMULATIVE EXCESS RAIN (IN)	INCREMENTAL EXCESS RAIN (IN)	HYDROGRAPH ORDINA (QN) (CFS)
2.5195	0.0932	1,018.7
2.6562	0.1367	1,101.9
2.9741	0.3179	1,205.0
3.0683	0.0942	1,328.5
3.1572	0.0889	1,465.8
3.2370	0.0798	1,627.5
3.2873	0.0503	1,790.5
3.3323	0.0450	1,946.9
3.3734	0.0411	2,069.7
3.4114	0.0381	2,147.0
3.4468	0.0353	2,177.3
3.4801	0.0333	2,163.2
3.5118	0.0316	2,110.0
3.5419	0.0302	2,026.1
3.5722	0.0302	1,923.7
3.6013	0.0291	1,805.1
3.6293	0.0281	1,673.5
3.6565	0.0271	1,541.3
3.6828	0.0263	1,422.0

SDHydroTools | Current Path: > SD Unit Hydrograph > Table



```
SDUHQuickResultsWB315 - Notepad
File Edit Format View Help
San Diego Unit Hydrograph Output for:
Study/Project Title Information: WB.3.1.5
Latitude: 33.2077
Longitude: -116.9833
Study Recurrence Interval: 100-year
Total Rainfall Depth (24-Hour Rainfall) (inch): 6.79
Area Adjusted Rainfall Depth (inch): 6.75
Watershed Area (mi2): 3.0
Adjusted Curve Number for Watershed Area: 84
Watershed Corps Lag Time (hours): 0.94
Computation Interval (D) (min): 5
Peak Flow Rate (cfs): 2177.35
Time Ran: 11/17/2020 9:25:03 AM
Ln 1, Col 1 | 100% | Windows (CRLF) | UTF-8
```

APPENDIX WB.B

**TABLE FOR CONVOLUTION OF UNIT HYDROGRAPH FOR NRCS
HYDROLOGIC METHOD EXAMPLE #2**

NRCS HYDROLOGIC METHOD EXAMPLE #2 TABLE FOR CONVOLUTION OF UNIT HYDROGRAPH
(Page 1 of 8)

Time (minutes)	UH Ordinate (cfs/inch)	15	30	45	60	75	90	105	120	135	150	165	180	Discharge* (cfs)
15	990	0												0
30	2,969	0	0											0
45	6,068	0	0	0										0
60	9,897	0	0	0	0									0
75	12,263	0	0	0	0	0								0
90	12,908	0	0	0	0	0	0							0
105	12,262	0	0	0	0	0	0	0						0
120	10,755	0	0	0	0	0	0	0	0					0
135	8,775	0	0	0	0	0	0	0	0	0				0
150	6,366	0	0	0	0	0	0	0	0	0	0			0
165	4,775	0	0	0	0	0	0	0	0	0	0	0		0
180	3,613	0	0	0	0	0	0	0	0	0	0	0	0	0
195	2,828	0	0	0	0	0	0	0	0	0	0	0	0	0
210	2,155	0	0	0	0	0	0	0	0	0	0	0	0	0
225	1,639	0	0	0	0	0	0	0	0	0	0	0	0	1
240	1,252	0	0	0	0	0	0	0	0	0	0	0	0	4
255	946	0	0	0	0	0	0	0	0	0	0	0	0	11
270	710	0	0	0	0	0	0	0	0	0	0	0	0	26
285	548	0	0	0	0	0	0	0	0	0	0	0	0	50
300	421	0	0	0	0	0	0	0	0	0	0	0	0	84
315	323	0	0	0	0	0	0	0	0	0	0	0	0	129
330	245	0	0	0	0	0	0	0	0	0	0	0	0	182
345	185	0	0	0	0	0	0	0	0	0	0	0	0	243
360	142	0	0	0	0	0	0	0	0	0	0	0	0	309
375	116	0	0	0	0	0	0	0	0	0	0	0	0	378
390	90	0	0	0	0	0	0	0	0	0	0	0	0	450
405	64	0	0	0	0	0	0	0	0	0	0	0	0	525
420	43	0	0	0	0	0	0	0	0	0	0	0	0	601
435	21	0	0	0	0	0	0	0	0	0	0	0	0	677
450	0	0	0	0	0	0	0	0	0	0	0	0	0	755
465			0	0	0	0	0	0	0	0	0	0	0	834
480				0	0	0	0	0	0	0	0	0	0	914
495					0	0	0	0	0	0	0	0	0	993
510						0	0	0	0	0	0	0	0	1,071
525							0	0	0	0	0	0	0	1,148
540								0	0	0	0	0	0	1,223
555									0	0	0	0	0	1,296
570										0	0	0	0	1,369
585											0	0	0	1,444
600												0	0	1,520
615													0	1,600

*Discharge includes values from columns not shown on this sheet.

NRCS HYDROLOGIC METHOD EXAMPLE #2 TABLE FOR CONVOLUTION OF UNIT HYDROGRAPH
(Page 2 of 8)

		Effective Rainfall (inches)											Discharge* (cfs)	
		195	210	225	240	255	270	285	300	315	330	345		360
Time (minutes)	UH Ordinate (cfs/inch)													
195	2,828	0												0
210	2,155	0	0											0
225	1,639	0	0	1										1
240	1,252	0	0	2	1									4
255	946	0	0	4	4	2								11
270	710	0	0	6	9	7	3							26
285	548	0	0	8	15	14	9	4						50
300	421	0	0	8	18	23	19	12	5					84
315	323	0	0	8	19	28	31	24	14	5				129
330	245	0	0	7	18	30	38	39	28	16	6			182
345	185	0	0	6	16	28	40	48	46	33	19	7		243
360	142	0	0	4	13	25	38	50	57	54	38	21	8	309
375	116	0	0	3	9	20	33	48	60	67	62	43	23	378
390	90	0	0	2	7	15	27	42	57	71	77	70	47	450
405	64	0	0	2	5	11	20	34	50	67	81	87	77	525
420	43	0	0	1	4	8	15	25	41	59	77	91	96	601
435	21	0	0	1	3	6	11	19	30	48	67	87	101	677
450	0	0	0	1	2	5	9	14	22	35	55	76	96	755
465		0	0	1	2	4	7	11	17	26	40	62	84	834
480		0	0	0	1	3	5	8	13	20	30	45	69	914
495		0	0	0	1	2	4	6	10	16	23	34	50	993
510		0	0	0	1	2	3	5	8	12	18	26	37	1,071
525		0	0	0	1	1	2	4	6	9	13	20	28	1,148
540		0	0	0	0	1	2	3	4	7	10	15	22	1,223
555		0	0	0	0	1	1	2	3	5	8	12	17	1,296
570		0	0	0	0	1	1	2	3	4	6	9	13	1,369
585		0	0	0	0	0	1	1	2	3	4	7	10	1,444
600		0	0	0	0	0	1	1	2	2	3	5	7	1,520
615		0	0	0	0	0	0	1	1	2	3	4	6	1,600
630		0	0	0	0	0	0	1	1	1	2	3	4	1,683
645			0	0	0	0	0	0	1	1	2	2	3	1,770
660				0	0	0	0	0	1	1	1	2	3	1,860
675					0	0	0	0	0	1	1	1	2	1,954
690						0	0	0	0	0	1	1	1	2,052
705							0	0	0	0	1	1	1	2,155
720								0	0	0	0	1	1	2,263
735									0	0	0	0	1	2,380
750										0	0	0	1	2,510
765											0	0	0	2,657
780												0	0	2,824
795													0	3,008

*Discharge includes values from columns not shown on this sheet.

NRCS HYDROLOGIC METHOD EXAMPLE #2 TABLE FOR CONVOLUTION OF UNIT HYDROGRAPH
(Page 3 of 8)

		Effective Rainfall (inches)											Discharge* (cfs)	
		375	390	405	420	435	450	465	480	495	510	525	540	
Time (minutes)	UH Ordinate (cfs/inch)	0.009	0.009	0.010	0.011	0.012	0.013	0.013	0.014	0.014	0.015	0.016	0.017	
375	116	9												378
390	90	26	9											450
405	64	52	28	10										525
420	43	85	57	30	11									601
435	21	106	93	62	32	12								677
450	0	111	115	101	66	35	12							755
465		106	121	125	108	71	37	13						834
480		93	115	131	134	116	76	40	14					914
495		76	101	125	141	144	124	81	42	14				993
510		55	82	110	134	152	153	132	86	42	15			1,071
525		41	60	89	117	144	161	164	140	87	44	16		1,148
540		31	45	65	96	127	153	173	173	141	91	47	16	1,223
555		24	34	49	70	103	134	164	182	175	148	96	49	1,296
570		19	26	37	52	75	110	144	173	184	184	157	101	1,369
585		14	20	29	39	56	80	117	152	175	193	195	164	1,444
600		11	15	22	31	43	60	85	124	153	184	205	204	1,520
615		8	12	17	24	33	45	64	90	125	161	195	214	1,600
630		6	9	13	18	25	35	48	67	91	131	171	204	1,683
645		5	7	10	14	19	27	38	51	68	95	139	179	1,770
660		4	5	7	10	15	20	29	40	52	71	101	146	1,860
675		3	4	6	8	11	16	22	30	40	54	76	106	1,954
690		2	3	4	6	8	12	17	23	31	42	57	79	2,052
705		2	2	3	5	6	9	13	18	23	32	45	60	2,155
720		1	2	2	4	5	7	9	13	18	25	34	47	2,263
735		1	1	2	3	4	5	7	10	14	19	26	36	2,380
750		1	1	1	2	3	4	6	8	10	14	20	27	2,510
765		1	1	1	2	2	3	4	6	8	11	15	21	2,657
780		0	1	1	1	2	2	3	5	6	8	11	16	2,824
795		0	0	1	1	1	2	2	3	5	6	9	12	3,008
810		0	0	0	1	1	1	2	3	3	5	7	9	3,204
825			0	0	0	1	1	2	2	3	4	5	7	3,411
840				0	0	1	1	1	2	2	3	4	5	3,628
855					0	0	1	1	1	2	2	3	4	3,876
870						0	0	1	1	1	2	2	3	4,176
885							0	0	1	1	1	2	2	4,557
900								0	0	1	1	1	2	5,041
915									0	0	1	1	1	5,614
930										0	0	1	1	6,263
945											0	0	1	7,043
960												0	0	7,983
975													0	9,378

*Discharge includes values from columns not shown on this sheet.

NRCS HYDROLOGIC METHOD EXAMPLE #2 TABLE FOR CONVOLUTION OF UNIT HYDROGRAPH
(Page 4 of 8)

Time (minutes)	UH Ordinate (cfs/inch)	555	570	585	600	615	630	645	660	675	690	705	720	Discharge* (cfs)
555	17													1,296
570	52	18												1,369
585	107	54	19											1,444
600	174	111	58	20										1,520
615	215	181	118	60	21									1,600
630	227	225	192	123	63	22								1,683
645	215	237	238	200	130	66	23							1,770
660	189	225	250	248	211	135	70	24						1,860
675	154	197	238	261	262	220	143	73	26					1,954
690	112	161	209	248	276	273	233	149	77	27				2,052
705	84	117	170	217	262	287	289	242	157	80	28			2,155
720	63	88	123	177	230	273	304	300	257	164	85	30		2,263
735	50	66	93	129	187	239	288	316	318	267	174	89		2,380
750	38	52	70	96	136	195	253	300	335	331	284	181		2,510
765	29	40	55	73	102	142	206	263	318	349	352	296		2,657
780	22	30	42	57	77	106	150	215	279	331	371	367		2,824
795	17	23	32	44	60	80	112	156	228	290	352	386		3,008
810	12	17	24	33	46	63	85	117	165	237	309	367		3,204
825	10	13	18	25	35	48	67	88	124	172	252	322		3,411
840	7	10	14	19	27	36	51	69	94	129	183	262		3,628
855	6	8	11	14	20	28	39	53	73	98	137	190		3,876
870	4	6	8	11	15	21	29	40	56	76	104	143		4,176
885	3	4	6	9	12	16	22	31	43	58	81	108		4,557
900	2	3	5	7	9	12	17	23	32	44	62	85		5,041
915	2	3	4	5	7	9	13	17	25	34	47	64		5,614
930	2	2	3	4	5	7	10	13	18	26	36	49		6,263
945	1	2	2	3	4	5	8	10	14	19	27	37		7,043
960	1	1	2	2	3	4	6	8	11	15	20	28		7,983
975	0	1	1	2	2	3	4	6	8	11	16	21		9,378
990	0	0	1	1	2	3	3	5	6	9	12	16		11,232
1005		0	0	1	1	2	3	3	5	7	9	13		13,381
1020			0	0	1	1	2	3	4	5	7	10		15,537
1035				0	0	1	2	2	3	4	5	7		16,911
1050					0	0	1	2	2	3	4	6		17,245
1065						0	1	1	2	2	3	4		16,678
1080							0	1	1	2	3	3		15,427
1095								0	1	1	2	3		13,816
1110									0	1	1	2		12,006
1125										0	1	1		10,449
1140											0	1		9,145
1155												0		8,077

*Discharge includes values from columns not shown on this sheet.

NRCS HYDROLOGIC METHOD EXAMPLE #2 TABLE FOR CONVOLUTION OF UNIT HYDROGRAPH
(Page 5 of 8)

Time (minutes)	UH Ordinate (cfs/inch)	735	750	765	780	795	810	825	840	855	870	885	900	Discharge* (cfs)
		0.035	0.036	0.039	0.040	0.043	0.045	0.049	0.051	0.075	0.078	0.084	0.093	
735	35													2,380
750	104	36												2,510
765	212	108	38											2,657
780	346	221	115	40										2,824
795	429	360	234	119	43									3,008
810	452	446	382	244	128	45								3,204
825	429	469	474	398	261	134	48							3,411
840	376	446	498	493	426	273	145	51						3,628
855	307	391	474	519	528	445	296	152	74					3,876
870	223	319	415	493	556	552	482	311	223	78				4,176
885	167	231	339	433	528	581	597	507	457	233	84			4,557
900	126	174	246	353	463	552	629	629	745	476	251	92		5,041
915	99	131	184	256	378	484	597	662	923	776	513	275		5,614
930	75	103	140	192	274	395	524	629	971	961	836	563		6,263
945	57	78	109	145	206	287	427	551	923	1,012	1,036	918		7,043
960	44	60	83	114	156	215	310	450	809	961	1,090	1,137		7,983
975	33	46	63	87	122	163	233	326	660	843	1,036	1,197		9,378
990	25	34	48	66	93	127	176	245	479	688	909	1,137		11,232
1005	19	26	37	50	71	97	138	185	359	499	741	997		13,381
1020	15	20	27	38	54	74	105	145	272	374	538	814		15,537
1035	11	15	21	29	41	56	80	110	213	283	403	590		16,911
1050	9	12	16	22	31	43	61	84	162	222	305	443		17,245
1065	6	9	12	17	24	32	46	64	123	169	239	335		16,678
1080	5	7	9	13	18	25	35	49	94	128	182	262		15,427
1095	4	5	7	10	14	19	27	36	71	98	138	200		13,816
1110	3	4	5	7	11	15	21	28	53	74	106	152		12,006
1125	2	3	4	6	8	11	16	22	41	56	80	116		10,449
1140	2	2	3	5	6	8	12	17	32	43	60	88		9,145
1155	1	2	2	4	5	6	9	13	24	33	46	66		8,077
1170	0	1	2	3	4	5	7	9	18	25	36	51		7,175
1185		0	1	2	3	4	6	7	14	19	27	39		6,417
1200			0	1	2	3	4	6	11	14	21	30		5,788
1215				0	1	2	3	5	9	11	16	23		5,263
1230					0	1	2	3	7	9	12	17		4,830
1245						0	1	2	5	7	10	13		4,480
1260							0	1	3	5	8	11		4,193
1275								0	2	3	5	8		3,955
1290									0	2	4	6		3,756
1305										0	2	4		3,586
1320											0	2		3,440
1335												0		3,313

*Discharge includes values from columns not shown on this sheet.

NRCS HYDROLOGIC METHOD EXAMPLE #2 TABLE FOR CONVOLUTION OF UNIT HYDROGRAPH
(Page 6 of 8)

Time (minutes)	UH Ordinate (cfs/inch)	Effective Rainfall (inches)											Discharge* (cfs)			
		915	930	945	960	975	990	1005	1020	1035	1050	1065		1080		
915	103	0.104														5,614
930	310	0.114	0.104													6,263
945	634	0.188	0.114	0.104												7,043
960	1,033	0.168	0.188	0.168	0.104											7,983
975	1,280	0.445	0.168	0.168	0.168	0.104										9,378
990	1,348	0.196	0.445	0.168	0.168	0.168	0.104									11,232
1005	1,280	0.110	0.445	0.168	0.168	0.168	0.168	0.104								13,381
1020	1,123	0.095	0.445	0.168	0.168	0.168	0.168	0.168	0.104							15,537
1035	916	0.066	0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.104						16,911
1050	665	0.059	0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.104					17,245
1065	499	0.054	0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.104				16,678
1080	377	0.051	0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.104			15,427
1095	295		0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.104		13,816
1110	225		0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.104	12,006
1125	171		0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	10,449
1140	131		0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	9,145
1155	99		0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	8,077
1170	74		0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	7,175
1185	57		0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	6,417
1200	44		0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	5,788
1215	34		0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	5,263
1230	26		0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	4,830
1245	19		0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	4,480
1260	15		0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	4,193
1275	12		0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	3,955
1290	9		0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	3,756
1305	7		0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	3,586
1320	4		0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	3,440
1335	2		0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	3,313
1350	0		0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	3,197
1365			0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	3,090
1380			0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	2,993
1395			0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	2,904
1410			0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	2,823
1425			0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	2,755
1440			0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	2,694
1455			0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	2,615
1470			0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	2,497
1485			0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	2,313
1500			0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	2,048
1515			0.445	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	0.168	1,735

*Discharge includes values from columns not shown on this sheet.

NRCS HYDROLOGIC METHOD EXAMPLE #2 TABLE FOR CONVOLUTION OF UNIT HYDROGRAPH
(Page 7 of 8)

Time (minutes)	UH Ordinate (cfs/inch)	Discharge* (cfs)
1095	43	13,816
1110	41	12,006
1125	265 122 38	10,449
1140	433 249 115 36	9,145
1155	536 406 235 109 35	8,077
1170	564 503 383 223 104 33	7,175
1185	536 529 474 364 213 100 32	6,417
1200	470 502 499 451 347 204 96 31	5,788
1215	384 441 474 474 430 332 196 92 31	5,263
1230	278 360 416 451 452 412 319 189 94 30	4,830
1245	209 261 340 395 430 433 396 308 192 91 30	4,480
1260	158 196 246 322 377 412 416 381 313 186 89 29	4,193
1275	124 148 185 234 308 361 396 401 388 304 181 86	3,955
1290	94 116 140 175 223 295 347 381 409 376 295 176	3,756
1305	72 88 109 133 167 214 283 335 388 396 366 287	3,586
1320	55 67 83 104 127 160 205 273 340 376 385 356	3,440
1335	41 51 63 79 99 121 154 198 278 330 366 375	3,313
1350	31 39 48 60 76 95 117 148 202 269 321 356	3,197
1365	24 29 37 46 57 72 91 112 151 195 262 312	3,090
1380	18 22 27 35 44 55 70 88 114 147 190 255	2,993
1395	14 17 21 26 33 42 53 67 90 111 142 185	2,904
1410	11 13 16 20 25 32 40 51 68 87 108 139	2,823
1425	8 10 12 15 19 24 31 39 52 66 84 105	2,755
1440	6 8 9 12 15 18 23 29 40 50 64 82	2,694
1455	5 6 7 9 11 14 18 22 30 38 49 63	2,615
1470	4 5 5 7 9 11 14 17 22 29 37 48	2,497
1485	3 4 4 5 6 8 10 13 17 22 28 36	2,313
1500	2 3 3 4 5 6 8 10 13 17 21 27	2,048
1515	1 2 2 3 4 5 6 8 10 13 16 21	1,735
1530	0 1 2 2 3 4 5 6 8 10 13 16	1,414
1545	0 1 2 2 3 4 4 6 8 10 12	1,114
1560	0 1 2 2 3 4 4 6 7 9	853
1575	0 1 2 3 4 4 6 7	641
1590	0 1 1 2 3 4 4 5	486
1605	0 1 1 2 3 3 4	371
1620	0 1 1 2 3 3	283
1635	0 1 1 2 3	215
1650	0 1 1 2	162
1665	0 1 1	123
1680	0 1	93
1695	0	70

*Discharge includes values from columns not shown on this sheet.

NRCS HYDROLOGIC METHOD EXAMPLE #2 TABLE FOR CONVOLUTION OF UNIT HYDROGRAPH
(Page 8 of 8)

Time (minutes)	UH Ordinate (cfs/inch)	Effective Rainfall (inches)	Discharge* (cfs)
1275	28	0.028	3,955
1290	84	0.028	3,756
1305	172	0.027	3,586
1320	280	0.026	3,440
1335	347	0.026	3,313
1350	365	0.025	3,197
1365	347	0.025	3,090
1380	304	0.024	2,993
1395	248	0.024	2,904
1410	180	0.024	2,823
1425	135	0.023	2,755
1440	102	0.023	2,694
1455	80		2,615
1470	61		2,497
1485	46		2,313
1500	35		2,048
1515	27		1,735
1530	20		1,414
1545	16		1,114
1560	12		853
1575	9		641
1590	7		486
1605	5		371
1620	4		283
1635	3		215
1650	3		162
1665	2		123
1680	1		93
1695	1		70
1710	0		53
1725			39
1740			29
1755			21
1770			16
1785			11
1800			8
1815			5
1830			3
1845			1
1860			0
1875			0

*Discharge includes values from columns not shown on this sheet.

APPENDIX WB.C
SDHYDROTOOLS COMPUTER PROGRAM INPUT AND OUTPUT FOR
NRCS HYDROLOGIC METHOD EXAMPLE #2

Hydro Tools

Project Interpolation RickRatHydro Nested Storm Convolution SD Unit Hydrograph

- Home
- Location**
- Rainfall Data

INSTRUCTIONS

Location may be entered manually or by adding the location on the map. This step is required for documentation of the project location and to activate the Rainfall Data Window, even when rainfall data will be entered manually. Enter one location per study (i.e., the program performs calculations for one location at a time).

For manual location input, enter latitude and longitude, then click 'verify' and the location will be shown on the map.

For map entry click, the location icon to start or stop adding a point. The location icon will appear green when enabled. With the location icon enabled, select the desired location on the map. Latitude and longitude will be updated automatically.

MANUAL LOCATION INPUT

Latitude (Decimal Degrees) Longitude (Decimal Degrees, include "-" for west)

- OR -

ADD LOCATION ON MAP

The map displays the San Diego area, including Oceanside, Escondido, Poway, San Diego, Chula Vista, Tijuana, Tecate, El Centro, and Mexicali. A red location pin is placed near Cleveland National Forest. A scale bar indicates 100 km. A 'GET DATA' button is located at the bottom right of the map area.

SDHydroTools | Current Path: > Project > Location

The screenshot shows the Hydro Tools application window. The title bar reads "Hydro Tools" and includes standard window controls. The breadcrumb navigation at the top shows the path: Project > Interpolation > RickRatHydro > Nested Storm Convolution > SD Unit Hydrograph. The left sidebar contains a menu with "Home", "Location", and "Rainfall Data" (which is selected). The main content area is titled "PRECIPITATION DEPTH" and displays a table of values for various durations. Below this is a section titled "NOAA ATLAS 14 DATA DESCRIPTION" which provides context about the data source and user input requirements. A "NEXT" button is located at the bottom right of the main content area. The status bar at the bottom shows "SDHydroTools | Current Path: > Project > Rainfall Data".

PRECIPITATION DEPTH	
5-Minute	2-Hour
0.406	1.916
10-Minute	3-Hour
0.582	2.287
15-Minute	6-Hour
0.704	3.016
30-Minute	12-Hour
0.977	4.002
60-Minute	24-Hour
1.396	5.384

NOAA ATLAS 14 DATA DESCRIPTION

Data Type: Precipitation Depth
Units: English
Time Series Type: Partial Duration

The program includes a built-in data set of 100-year recurrence interval precipitation data. Data accessed by the program appears in green boxes.

Users may overwrite the data accessed by the program with data for other recurrence intervals in order to use the program computational tools for other recurrence intervals.

User-entered data must be the type shown above. User-entered data will appear in red boxes.

NEXT

SDHydroTools | Current Path: > Project > Rainfall Data

Hydro Tools

Project Interpolation RickRatHydro Nested Storm Convolution **SD Unit Hydrograph**

Home Table

SDUH INPUT

Computation Interval (D) (min)
D should be $\leq 0.2T_p$

15

Watershed Area (mi²)

40.0

Adjusted Curve Number for Watershed Area

85

Watershed Corps Lag Time (hrs)

1.74

RESULTS SUMMARY

Export

ATTRIBUTE	VALUE	UNIT
-----------	-------	------

CALCULATE

SDHydroTools | Current Path: > SD Unit Hydrograph > Home

Hydro Tools

Project Interpolation RickRatHydro Nested Storm Convolution **SD Unit Hydrograph**

Home Table

SDUH INPUT

Computation Interval (D) (min)
D should be $\leq 0.2T_p$

Watershed Area (mi²)

Adjusted Curve Number for Watershed Area

Watershed Corps Lag Time (hrs)

15 40.0

85 1.74

RESULTS SUMMARY

Export

ATTRIBUTE	VALUE	UNIT
Study/Project Title Information	WB.3.2.6	
Latitude	32.8918	
Longitude	-116.8488	
Study Recurrence Interval	100-year	
Total Rainfall Depth (24-Hour Rainfall)	5.38	inch
Area Adjusted Rainfall Depth	5.16	inch
Watershed Area	40.0	mi ²
Adjusted Curve Number for Watershed Area	85	
Watershed Corps Lag Time	1.74	hours
Computation Interval (D)	15	min
Peak Flow Rate	17218.51	cfs
Time Ran	11/17/2020 11:10:09 AM	

CALCULATE

SDHydroTools | Current Path: > SD Unit Hydrograph > Home

Hydro Tools

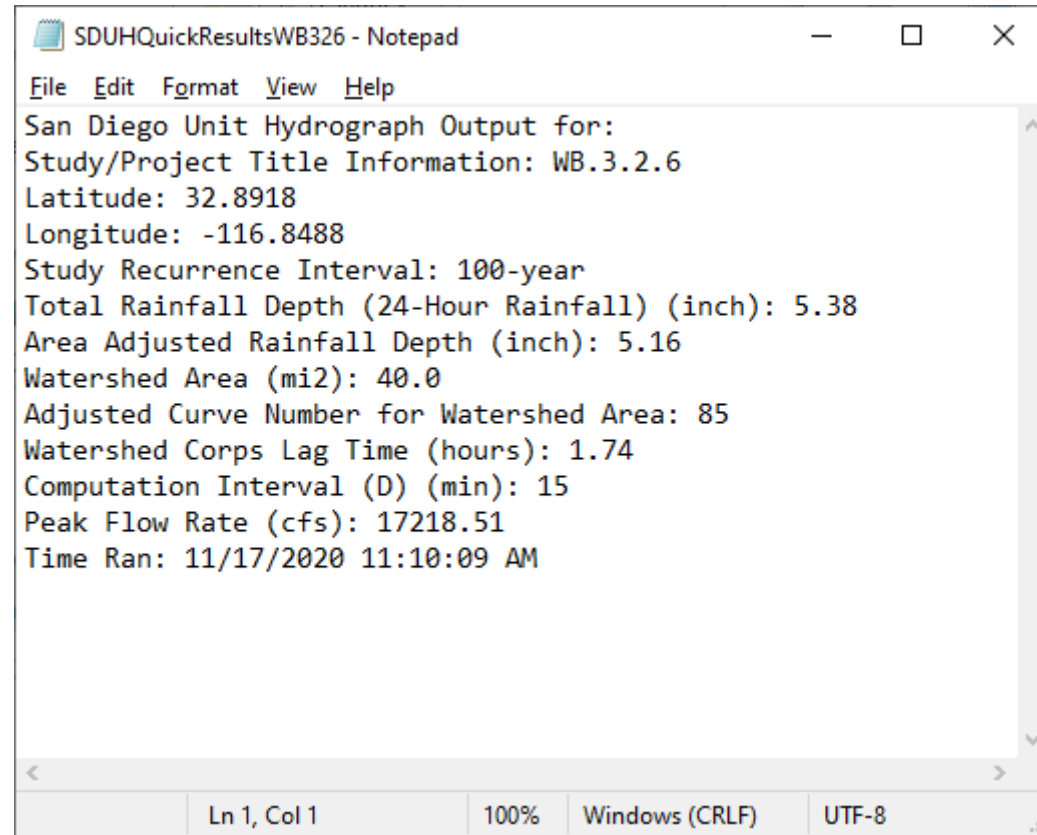
Project Interpolation RickRatHydro Nested Storm Convolution **SD Unit Hydrograph**

SDUH Table Unit Hydrograph UH Time

Export

UMMULATIVE EXCESS RAIN (IN)	INCREMENTAL EXCESS RAIN (IN)	HYDROGRAPH ORDINATE (QN) (CFS)
1469	0.0928	5,046.9
1514	0.1044	5,622.1
1657	0.1143	6,271.6
1527	0.1869	7,051.0
1209	0.1682	7,988.1
1658	0.4450	9,379.2
1602	0.1943	11,226.1
1703	0.1101	13,367.8
1655	0.0953	15,517.4
1311	0.0655	16,886.6
1901	0.0590	17,218.5
1445	0.0544	16,653.5
1953	0.0508	15,405.6
1393	0.0439	13,798.0
1805	0.0412	11,991.1
1194	0.0389	10,438.9
1563	0.0370	9,140.3
1916	0.0353	8,076.6

SDHydroTools | Current Path: > SD Unit Hydrograph > Table



SDUHQuickResultsWB326 - Notepad

File Edit Format View Help

San Diego Unit Hydrograph Output for:
Study/Project Title Information: WB.3.2.6
Latitude: 32.8918
Longitude: -116.8488
Study Recurrence Interval: 100-year
Total Rainfall Depth (24-Hour Rainfall) (inch): 5.38
Area Adjusted Rainfall Depth (inch): 5.16
Watershed Area (mi2): 40.0
Adjusted Curve Number for Watershed Area: 85
Watershed Corps Lag Time (hours): 1.74
Computation Interval (D) (min): 15
Peak Flow Rate (cfs): 17218.51
Time Ran: 11/17/2020 11:10:09 AM

Ln 1, Col 1 100% Windows (CRLF) UTF-8

Appendix WB.C
 SDHydroTools Exported SDUH Results Table for NRCS Hydrologic Method Example #2
 (with formatting added)

N	Time (min)	Duration (min)	Depth Area		Incremental Hyetograph		Cummulative	Incremental	Hydrograph	
			Precipitation for D	Adjustment (in)	Depth Area Adjusted P (in)	Ordinate (R) (in)	Rainfall (in)	Excess Rain (in)	Excess Rain (in)	Ordinate (QN) (cfs)
96	15	1440	5.38	0.96	5.16	0.02	0.02	0.00	0.00	0.00
95	30	1425	5.36	0.96	5.13	0.02	0.05	0.00	0.00	0.00
93	45	1395	5.31	0.96	5.08	0.02	0.07	0.00	0.00	0.00
92	60	1380	5.29	0.96	5.06	0.02	0.10	0.00	0.00	0.00
90	75	1350	5.24	0.96	5.01	0.03	0.12	0.00	0.00	0.00
89	90	1335	5.21	0.96	4.98	0.03	0.15	0.00	0.00	0.00
87	105	1305	5.16	0.96	4.93	0.03	0.17	0.00	0.00	0.00
86	120	1290	5.14	0.96	4.91	0.03	0.20	0.00	0.00	0.00
84	135	1260	5.08	0.96	4.86	0.03	0.23	0.00	0.00	0.00
83	150	1245	5.06	0.95	4.83	0.03	0.25	0.00	0.00	0.00
81	165	1215	5.01	0.95	4.78	0.03	0.28	0.00	0.00	0.00
80	180	1200	4.98	0.95	4.75	0.03	0.31	0.00	0.00	0.00
78	195	1170	4.93	0.95	4.70	0.03	0.33	0.00	0.00	0.00
77	210	1155	4.90	0.95	4.67	0.03	0.36	0.00	0.00	0.03
75	225	1125	4.84	0.95	4.62	0.03	0.39	0.00	0.00	0.75
74	240	1110	4.82	0.95	4.59	0.03	0.42	0.00	0.00	3.63
72	255	1080	4.76	0.95	4.53	0.03	0.44	0.00	0.00	11.05
71	270	1065	4.73	0.95	4.50	0.03	0.47	0.01	0.00	25.93
69	285	1035	4.67	0.95	4.45	0.03	0.50	0.01	0.00	50.46
68	300	1020	4.65	0.95	4.42	0.03	0.53	0.02	0.00	85.37
66	315	990	4.59	0.95	4.36	0.03	0.56	0.02	0.01	130.26
65	330	975	4.56	0.95	4.33	0.03	0.59	0.03	0.01	183.92
63	345	945	4.50	0.95	4.27	0.03	0.62	0.04	0.01	244.73
62	360	930	4.47	0.95	4.24	0.03	0.65	0.04	0.01	310.73
60	375	900	4.40	0.95	4.18	0.03	0.68	0.05	0.01	380.44
59	390	885	4.37	0.95	4.15	0.03	0.71	0.06	0.01	452.85
57	405	855	4.31	0.95	4.08	0.03	0.74	0.07	0.01	527.30
56	420	840	4.27	0.95	4.05	0.03	0.78	0.08	0.01	603.26
54	435	810	4.21	0.95	3.99	0.03	0.81	0.09	0.01	680.41
53	450	795	4.18	0.95	3.96	0.03	0.84	0.11	0.01	758.48
51	465	765	4.11	0.95	3.89	0.03	0.88	0.12	0.01	837.34
50	480	750	4.07	0.95	3.85	0.03	0.91	0.13	0.01	916.90
48	495	720	4.00	0.95	3.79	0.03	0.94	0.15	0.01	996.51
47	510	705	3.97	0.95	3.75	0.03	0.98	0.16	0.02	1075.43
45	525	675	3.90	0.95	3.68	0.03	1.01	0.18	0.02	1152.90
44	540	660	3.86	0.95	3.65	0.04	1.05	0.20	0.02	1228.33

Appendix WB.C
SDHydroTools Exported SDUH Results Table for NRCS Hydrologic Method Example #2
(with formatting added)

N	Time (min)	Duration (min)	Depth Area		Incremental Hyetograph		Cummulative Rainfall (in)	Cummulative Excess Rain (in)	Incremental Excess Rain (in)	Hydrograph Ordinate (QN) (cfs)
			Precipitation for D	Adjustment (in)	Depth Area Adjusted P (in)	Ordinate (R) (in)				
42	555	630	3.79	0.94	3.58	0.04	1.08	0.21	0.02	1302.73
41	570	615	3.75	0.94	3.54	0.04	1.12	0.23	0.02	1377.26
39	585	585	3.68	0.94	3.47	0.04	1.16	0.25	0.02	1453.09
38	600	570	3.64	0.94	3.43	0.04	1.20	0.27	0.02	1531.02
36	615	540	3.56	0.94	3.36	0.04	1.24	0.29	0.02	1611.78
35	630	525	3.52	0.94	3.32	0.04	1.28	0.32	0.02	1695.96
33	645	495	3.43	0.94	3.24	0.04	1.32	0.34	0.02	1783.57
32	660	480	3.39	0.94	3.19	0.04	1.36	0.36	0.02	1874.63
30	675	450	3.30	0.94	3.11	0.04	1.40	0.39	0.03	1969.49
29	690	435	3.26	0.94	3.07	0.04	1.45	0.42	0.03	2068.46
27	705	405	3.16	0.94	2.98	0.05	1.49	0.45	0.03	2172.15
26	720	390	3.12	0.94	2.93	0.05	1.54	0.48	0.03	2280.97
24	735	360	3.02	0.94	2.84	0.05	1.59	0.51	0.04	2398.61
23	750	345	2.97	0.94	2.78	0.05	1.65	0.55	0.04	2528.51
21	765	315	2.86	0.93	2.67	0.06	1.71	0.59	0.04	2674.85
20	780	300	2.80	0.93	2.61	0.06	1.76	0.63	0.04	2840.40
18	795	270	2.69	0.93	2.49	0.06	1.83	0.67	0.04	3022.15
17	810	255	2.63	0.93	2.43	0.06	1.89	0.72	0.05	3216.07
15	825	225	2.50	0.92	2.30	0.07	1.96	0.76	0.05	3420.66
14	840	210	2.43	0.92	2.24	0.07	2.03	0.82	0.05	3635.46
12	855	180	2.29	0.92	2.09	0.10	2.13	0.89	0.08	3881.93
11	870	165	2.20	0.90	1.99	0.10	2.23	0.97	0.08	4181.16
9	885	135	2.02	0.88	1.78	0.11	2.34	1.05	0.08	4561.80
8	900	120	1.92	0.87	1.67	0.12	2.46	1.15	0.09	5046.92
6	915	90	1.68	0.85	1.43	0.13	2.59	1.25	0.10	5622.13
5	930	75	1.55	0.84	1.30	0.14	2.73	1.37	0.11	6271.59
3	945	45	1.20	0.78	0.94	0.23	2.96	1.55	0.19	7050.95
2	960	30	0.98	0.73	0.71	0.20	3.16	1.72	0.17	7988.11
1	975	15	0.70	0.73	0.51	0.51	3.67	2.17	0.44	9379.20
4	990	60	1.40	0.83	1.16	0.22	3.89	2.36	0.19	11226.08
7	1005	105	1.80	0.86	1.55	0.12	4.01	2.47	0.11	13367.84
10	1020	150	2.11	0.89	1.89	0.11	4.12	2.57	0.10	15517.41
13	1035	195	2.36	0.92	2.17	0.07	4.19	2.63	0.07	16886.61
16	1050	240	2.57	0.92	2.37	0.07	4.26	2.69	0.06	17218.51
19	1065	285	2.75	0.93	2.55	0.06	4.32	2.74	0.05	16653.53
22	1080	330	2.91	0.94	2.73	0.06	4.37	2.80	0.05	15405.59

Appendix WB.C
SDHydroTools Exported SDUH Results Table for NRCS Hydrologic Method Example #2
(with formatting added)

N	Time (min)	Duration (min)	Depth Area		Incremental Hyetograph		Cummulative	Incremental	Hydrograph	
			Precipitation for D	Adjustment (in)	Depth Area Adjusted P (in)	Ordinate (R) (in)	Excess Rain (in)	Excess Rain (in)	Ordinate (QN) (cfs)	
25	1095	375	3.07	0.94	2.88	0.05	4.42	2.84	0.04	13797.99
28	1110	420	3.21	0.94	3.02	0.05	4.47	2.88	0.04	11991.09
31	1125	465	3.35	0.94	3.15	0.04	4.51	2.92	0.04	10438.91
34	1140	510	3.48	0.94	3.28	0.04	4.55	2.96	0.04	9140.33
37	1155	555	3.60	0.94	3.39	0.04	4.59	2.99	0.04	8076.59
40	1170	600	3.72	0.94	3.51	0.04	4.63	3.03	0.03	7178.91
43	1185	645	3.83	0.94	3.61	0.04	4.66	3.06	0.03	6425.47
46	1200	690	3.93	0.95	3.72	0.03	4.70	3.09	0.03	5799.60
49	1215	735	4.04	0.95	3.82	0.03	4.73	3.12	0.03	5277.45
52	1230	780	4.14	0.95	3.92	0.03	4.77	3.15	0.03	4846.33
55	1245	825	4.24	0.95	4.02	0.03	4.80	3.18	0.03	4496.96
58	1260	870	4.34	0.95	4.12	0.03	4.83	3.21	0.03	4209.59
61	1275	915	4.43	0.95	4.21	0.03	4.86	3.24	0.03	3970.41
64	1290	960	4.53	0.95	4.30	0.03	4.89	3.27	0.03	3769.37
67	1305	1005	4.62	0.95	4.39	0.03	4.92	3.29	0.03	3599.09
70	1320	1050	4.70	0.95	4.48	0.03	4.95	3.32	0.03	3451.95
73	1335	1095	4.79	0.95	4.56	0.03	4.98	3.35	0.03	3327.58
76	1350	1140	4.87	0.95	4.64	0.03	5.00	3.37	0.03	3209.66
79	1365	1185	4.95	0.95	4.72	0.03	5.03	3.40	0.02	3097.14
82	1380	1230	5.03	0.95	4.80	0.03	5.06	3.42	0.02	2994.41
85	1395	1275	5.11	0.96	4.88	0.03	5.08	3.44	0.02	2904.99
88	1410	1320	5.19	0.96	4.96	0.03	5.11	3.47	0.02	2826.81
91	1425	1365	5.26	0.96	5.03	0.03	5.13	3.49	0.02	2759.26
94	1440	1410	5.34	0.96	5.11	0.02	5.16	3.51	0.02	2698.30
	1455		0.00	0.00	0.00	0.00	0.00	0.00	0.00	2619.73
	1470		0.00	0.00	0.00	0.00	0.00	0.00	0.00	2500.92
	1485		0.00	0.00	0.00	0.00	0.00	0.00	0.00	2316.71
	1500		0.00	0.00	0.00	0.00	0.00	0.00	0.00	2051.22
	1515		0.00	0.00	0.00	0.00	0.00	0.00	0.00	1738.22
	1530		0.00	0.00	0.00	0.00	0.00	0.00	0.00	1416.83
	1545		0.00	0.00	0.00	0.00	0.00	0.00	0.00	1115.91
	1560		0.00	0.00	0.00	0.00	0.00	0.00	0.00	854.29
	1575		0.00	0.00	0.00	0.00	0.00	0.00	0.00	641.87
	1590		0.00	0.00	0.00	0.00	0.00	0.00	0.00	487.40
	1605		0.00	0.00	0.00	0.00	0.00	0.00	0.00	371.44
	1620		0.00	0.00	0.00	0.00	0.00	0.00	0.00	283.64

Appendix WB.C
 SDHydroTools Exported SDUH Results Table for NRCS Hydrologic Method Example #2
 (with formatting added)

N	Time (min)	Duration (min)	Precipitation for D	Depth Area		Incremental Hyetograph	Cummulative Rainfall (in)	Cummulative Excess Rain (in)	Incremental Excess Rain (in)	Hydrograph Ordinate (QN) (cfs)
				Adjustment (in)	Depth Area Adjusted P (in)	Ordinate (R) (in)				
	1635		0.00	0.00	0.00	0.00	0.00	0.00	0.00	215.04
	1650		0.00	0.00	0.00	0.00	0.00	0.00	0.00	162.79
	1665		0.00	0.00	0.00	0.00	0.00	0.00	0.00	123.08
	1680		0.00	0.00	0.00	0.00	0.00	0.00	0.00	92.78
	1695		0.00	0.00	0.00	0.00	0.00	0.00	0.00	69.89
	1710		0.00	0.00	0.00	0.00	0.00	0.00	0.00	52.72
	1725		0.00	0.00	0.00	0.00	0.00	0.00	0.00	39.49
	1740		0.00	0.00	0.00	0.00	0.00	0.00	0.00	29.33
	1755		0.00	0.00	0.00	0.00	0.00	0.00	0.00	21.58
	1770		0.00	0.00	0.00	0.00	0.00	0.00	0.00	15.71
	1785		0.00	0.00	0.00	0.00	0.00	0.00	0.00	11.29
	1800		0.00	0.00	0.00	0.00	0.00	0.00	0.00	7.91
	1815		0.00	0.00	0.00	0.00	0.00	0.00	0.00	4.83
	1830		0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.43
	1845		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.94
	1860		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.25
	1875		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	1890		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

APPENDIX WB.D
HEC-HMS INPUT AND OUTPUT FOR NRCS HYDROLOGIC METHOD
EXAMPLE #2

The screenshot displays the HEC-HMS 4.1 software interface. The main window title is "HEC-HMS 4.1 [C:\SCRATCH\HECHMS\18164-U\WB326\WB326.hms]". The menu bar includes File, Edit, View, Components, Parameters, Compute, Results, Tools, and Help. The toolbar contains various icons for file operations and navigation. The left sidebar shows a project tree with folders: WB326, Basin Models (containing Basin 1), Meteorologic Models (containing Met 1), and Control Specifications (containing Control 1). Under Basin 1, there is a subbasin named "Subbasin-1".

The "Basin Model [Basin 1]" window is open, showing a central workspace with a "Subbasin-1" icon. Below this window, the "Subbasin" configuration panel is visible, with tabs for "Subbasin", "Loss", "Transform", and "Options". The configuration details are as follows:

- Basin Name:** Basin 1
- Element Name:** Subbasin-1
- Description:** Single Subbasin Watershed
- Downstream:** --None--
- *Area (MI2):** 40.0
- Latitude Degrees:** [Empty]
- Latitude Minutes:** [Empty]
- Latitude Seconds:** [Empty]
- Longitude Degrees:** [Empty]
- Longitude Minutes:** [Empty]
- Longitude Seconds:** [Empty]
- Canopy Method:** --None--
- Surface Method:** --None--
- Loss Method:** SCS Curve Number
- Transform Method:** SCS Unit Hydrograph
- Baseflow Method:** --None--

At the bottom of the interface, there are two status notes:

- NOTE 10008: Begin opening project "WB326" in directory "C:\SCRATCH\HECHMS\18164-U\WB326" at time 12Nov2020, 15:26:02.
- NOTE 10019: Finished opening project "WB326" in directory "C:\SCRATCH\HECHMS\18164-U\WB326" at time 12Nov2020, 15:26:02.

The screenshot displays the HEC-HMS 4.1 software interface. The main window title is "HEC-HMS 4.1 [C:\SCRATCH\HECHMS\18164-U\WB326\WB326.hms]". The menu bar includes File, Edit, View, Components, Parameters, Compute, Results, Tools, and Help. The toolbar contains various icons for file operations and navigation. On the left, a project tree shows the following structure:

- WB326
 - Basin Models
 - Basin 1
 - Subbasin-1
 - Meteorologic Models
 - Met 1
 - Frequency Storm
 - Subbasin-1
 - Control Specifications
 - Control 1

The "Components" tab is active, showing sub-tabs for Subbasin, Loss, Transform, and Options. The "Subbasin" sub-tab is selected, displaying the following parameters:

- Basin Name: Basin 1
- Element Name: Subbasin-1
- Initial Abstraction (IN):
- *Curve Number: 85
- *Impervious (%): 0.0

The "Basin Model [Basin 1]" window is open, showing a central area with a "Subbasin-1" icon. Below this window, a log window contains the following text:

NOTE 10008: Begin opening project "WB326" in directory "C:\SCRATCH\HECHMS\18164-U\WB326" at time 12Nov2020, 15:26:02.
NOTE 10019: Finished opening project "WB326" in directory "C:\SCRATCH\HECHMS\18164-U\WB326" at time 12Nov2020, 15:26:02.

The screenshot displays the HEC-HMS 4.1 software interface. The main window title is "HEC-HMS 4.1 [C:\SCRATCH\HECHMS\18164-U\WB326\WB326.hms]". The menu bar includes File, Edit, View, Components, Parameters, Compute, Results, Tools, and Help. The toolbar contains various icons for file operations and navigation. On the left, a project tree shows the following structure:

- WB326
 - Basin Models
 - Basin 1
 - Subbasin-1
 - Meteorologic Models
 - Met 1
 - Frequency Storm
 - Subbasin-1
 - Control Specifications
 - Control 1

The "Components" tab is active, showing a "Subbasin" element. Below it, the "Basin Name" is "Basin 1" and the "Element Name" is "Subbasin-1". The "Graph Type" is set to "Standard (PRF 484)" and the "*Lag Time (MIN)" is 82.5. The "Basin Model [Basin 1]" window is open, showing a central icon for "Subbasin-1". The console at the bottom displays the following notes:

```
NOTE 10008: Begin opening project "WB326" in directory "C:\SCRATCH\HECHMS\18164-U\WB326" at time 12Nov2020, 15:26:02.  
NOTE 10019: Finished opening project "WB326" in directory "C:\SCRATCH\HECHMS\18164-U\WB326" at time 12Nov2020, 15:26:02.
```

The screenshot displays the HEC-HMS 4.1 software interface. The main window title is "HEC-HMS 4.1 [C:\SCRATCH\HECHMS\18164-U\WB326\WB326.hms]". The menu bar includes File, Edit, View, Components, Parameters, Compute, Results, Tools, and Help. The toolbar contains various icons for file operations and navigation. On the left, a project tree shows the following structure:

- WB326
 - Basin Models
 - Basin 1
 - Subbasin-1
 - Meteorologic Models
 - Met 1
 - Frequency Storm
 - Subbasin-1
 - Control Specifications
 - Control 1

The "Components" tab is active, showing a "Subbasin" configuration panel with the following fields:

- Basin Name: Basin 1
- Element Name: Subbasin-1
- Observed Flow: --None--
- Observed Stage: --None--
- Observed SWE: --None--
- Elev-Discharge: --None--
- Ref Flow (CFS):
- Ref Label:

The "Basin Model [Basin 1]" window is open, showing a central area with a "Subbasin-1" icon. Below this window, a log window displays the following messages:

```
NOTE 10008: Begin opening project "WB326" in directory "C:\SCRATCH\HECHMS\18164-U\WB326" at time 12Nov2020, 15:26:02.  
NOTE 10019: Finished opening project "WB326" in directory "C:\SCRATCH\HECHMS\18164-U\WB326" at time 12Nov2020, 15:26:02.
```

HEC-HMS 4.1 [C:\SCRATCH\HECHMS\18164-U\WB326\WB326.hms]

File Edit View Components Parameters Compute Results Tools Help

WB326

- Basin Models
 - Basin 1
 - Subbasin-1
- Meteorologic Models
 - Met 1
 - Frequency Storm
 - Subbasin-1
- Control Specifications
 - Control 1

Components Compute Results

Frequency Storm

Met Name: Met 1

Probability: Other

Input Type: Partial Duration

Output Type: Annual Duration

Intensity Duration: 15 Minutes

Storm Duration: 1 Day

Intensity Position: 67 Percent

Storm Area (MI2): 40.0

Curve: Uniform For All Subbasins

Duration	Partial-Duration Depth (IN)
5 Minutes	
15 Minutes	0.70400
1 Hour	1.4000
2 Hours	1.9200
3 Hours	2.2900
6 Hours	3.0200
12 Hours	4.0000
1 Day	5.3800
2 Days	
4 Days	
7 Days	
10 Days	

Basin Model [Basin 1]

Subbasin-1

NOTE 10008: Begin opening project "WB326" in directory "C:\SCRATCH\HECHMS\18164-U\WB326" at time 12Nov2020, 15:26:02.
NOTE 10019: Finished opening project "WB326" in directory "C:\SCRATCH\HECHMS\18164-U\WB326" at time 12Nov2020, 15:26:02.

The screenshot displays the HEC-HMS 4.1 software interface. The main window title is "HEC-HMS 4.1 [C:\SCRATCH\HECHMS\18164-U\WB326\WB326.hms]". The menu bar includes File, Edit, View, Components, Parameters, Compute, Results, Tools, and Help. The toolbar contains various icons for file operations and navigation. The left sidebar shows a project tree with folders: WB326, Basin Models (Basin 1, Subbasin-1), Meteorologic Models (Met 1, Frequency Storm, Subbasin-1), and Control Specifications (Control 1). The bottom-left panel is titled "Control Specifications" and contains the following fields:

- Name: Control 1
- Description: Control 1
- *Start Date (ddMMYYYY): 01Jan2020
- *Start Time (HH:mm): 00:00
- *End Date (ddMMYYYY): 02Jan2020
- *End Time (HH:mm): 12:00
- Time Interval: 15 Minutes

The main workspace is titled "Basin Model [Basin 1]" and shows a diagram with a single element labeled "Subbasin-1". Below the workspace, a log window displays the following messages:

```
NOTE 10008: Begin opening project "WB326" in directory "C:\SCRATCH\HECHMS\18164-U\WB326" at time 12Nov2020, 15:26:02.  
NOTE 10019: Finished opening project "WB326" in directory "C:\SCRATCH\HECHMS\18164-U\WB326" at time 12Nov2020, 15:26:02.  
NOTE 10181: Opened control specifications "Control 1" at time 12Nov2020, 15:27:55.
```

The screenshot displays the HEC-HMS 4.1 software interface. The main window title is "HEC-HMS 4.1 [C:\SCRATCH\HECHMS\18164-U\WB326\WB326.hms]". The menu bar includes File, Edit, View, Components, Parameters, Compute, Results, Tools, and Help. The toolbar contains various icons for file operations and simulation control. The left sidebar shows a project tree with "WB326" containing "Simulation Runs" and "Run 1". Below the sidebar are tabs for "Components", "Compute", and "Results". The "Simulation Run" tab is active, showing details for "Run 1":
Name: Run 1
Description: Basin: Basin 1 & Met: Met 1
DSS File: C:\SCRATCH\HECHMS\18164-U\WB326\WB326.hms
Basin Model: Basin 1
Meteorologic Model: Met 1
Control Specifications: Control 1
A separate window titled "Basin Model [Basin 1] Current Run [Run 1]" is open, showing a map of "Subbasin-1" with a blue water icon and a right-pointing arrow. At the bottom of the main window, a log window displays the following text:
NOTE 10008: Begin opening project "WB326" in directory "C:\SCRATCH\HECHMS\18164-U\WB326" at time 12Nov2020, 15:26:02.
NOTE 10019: Finished opening project "WB326" in directory "C:\SCRATCH\HECHMS\18164-U\WB326" at time 12Nov2020, 15:26:02.
NOTE 10181: Opened control specifications "Control 1" at time 12Nov2020, 15:27:55.
NOTE 15301: Began computing simulation run "Run 1" at time 12Nov2020, 15:28:23.
NOTE 20364: Found no parameter problems in meteorologic model "Met 1".
NOTE 40049: Found no parameter problems in basin model "Basin 1".
NOTE 15302: Finished computing simulation run "Run 1" at time 12Nov2020, 15:28:24.

The screenshot displays the HEC-HMS 4.1 software interface. The main window is titled "Basin Model [Basin 1] Current Run [Run 1]". On the left, a tree view shows the project structure: WB326 > Simulation Runs > Run 1 > Subbasin-1. The "Summary Table" is selected. A "Summary Results for Subbasin 'Subbasin-1'" dialog box is open, showing the following information:

Project: WB326 Simulation Run: Run 1
Subbasin: Subbasin-1

Start of Run:	01Jan2020, 00:00	Basin Model:	Basin 1
End of Run:	02Jan2020, 12:00	Meteorologic Model:	Met 1
Compute Time:	12Nov2020, 15:28:23	Control Specifications:	Control 1

Volume Units: IN AC-FT

Computed Results

Peak Discharge:	16961.2 (CFS)	Date/Time of Peak Discharge:	01Jan2020, 17:30
Precipitation Volume:	5.16 (IN)	Direct Runoff Volume:	3.52 (IN)
Loss Volume:	1.64 (IN)	Baseflow Volume:	0.00 (IN)
Excess Volume:	3.52 (IN)	Discharge Volume:	3.52 (IN)

At the bottom of the interface, a log window displays the following notes:

```
NOTE 10008: Begin opening project "WB326" in directory "C:\SCRATCH\HECHMS\18164-U\WB326" at time 12Nov2020, 15:26:02.  
NOTE 10019: Finished opening project "WB326" in directory "C:\SCRATCH\HECHMS\18164-U\WB326" at time 12Nov2020, 15:26:02.  
NOTE 10181: Opened control specifications "Control 1" at time 12Nov2020, 15:27:55.  
NOTE 15301: Began computing simulation run "Run 1" at time 12Nov2020, 15:28:23.  
NOTE 20364: Found no parameter problems in meteorologic model "Met 1".  
NOTE 40049: Found no parameter problems in basin model "Basin 1".  
NOTE 15302: Finished computing simulation run "Run 1" at time 12Nov2020, 15:28:24.
```

