HYDROLOGY MANUAL

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Water Resources Division
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CHAPTER

1

Introduction

1.1 PURPOSE AND SCOPE

This manual establishes the Los Angeles County Department of Public Works' hydrologic design procedures and serves as a reference and training guide. This manual contains charts, graphs, and tables necessary to conduct a hydrologic study within the County of Los Angeles. Examples provide guidance on using the hydrologic methods.

The primary purpose of this manual is to explain the steps involved in converting rainfall to runoff flow rates and volumes using Public Works' standards. This manual contains procedures and standards developed and revised by the Water Resources Division based on historic rainfall and runoff data collected within the county. The hydrologic techniques in this manual apply to the design of local storm drains, retention and detention basins, pump stations, and major channel projects. The techniques also apply to storm drain deficiency and flood hazard evaluations. Low flow hydrology methods related to water quality standards are also discussed.

This manual compiles information from previous editions of the County of Los Angeles Hydrology Manual, the 2002 Hydrology Manual Addendum, and other reference materials. The standards set forth in this manual govern all hydrology calculations done under Public Works' jurisdiction. Hydrologic procedures in manuals prepared for use by other Divisions within Public Works must be compatible with this manual.

1.2 OVERVIEW OF HYDROLOGIC METHOD

The Los Angeles County Flood Control District initiated its Comprehensive Plan in 1931. Engineers determined that the runoff data within the District was insufficient to create empirical runoff calculations due to limited stream flow data. Lack of stream flow data made it difficult to establish standards and a hydrologic method based on runoff observations. Therefore, the engineers decided that computing design flows based on rainfall was necessary. A rainfall based hydrologic method was deemed more acceptable due to the availability of rainfall data. Figure 1.2.1 shows a rain gage used to collect rainfall data for hydrologic analysis.

Figure 1.2.1
Rain Gage #47D Located at Clear Creek School

Using rainfall-runoff relationships, methods are developed to compute flow rates and define hydrographs based on a design storm event. The two rainfall-runoff methods that apply to hydrology studies within the County of Los Angeles are the Rational and Modified Rational Methods. The use of these rainfall-runoff methods depends on the study requirements.

The Rational Method, Q = CIA, is used for simple hydrology studies within the County of Los Angeles. This method produces a peak flow rate and is only applicable to small areas. The Rational Method applies to development of small areas when no storage volume information is required and overland flow is the primary collection method.

The primary method, in use since the 1930's, is the Modified Rational Method (MODRAT). MODRAT is based on the Rational Method, but uses a time of concentration and a design storm to determine intensities throughout the storm period. The intensities are used to determine the soil runoff coefficient. The rational formula then provides a flow rate for a specific time. Plotting the time specific flow rate provides a hydrograph and an associated flow volume. MODRAT is the standard method for hydrologic studies within the county. Computer programs implement MODRAT to compute runoff data from input parameters.

MODRAT relies on a design storm defined by a time-intensity relationship and a spatial precipitation pattern. The temporal and spatial distributions of rainfall used with MODRAT have changed over the years based on analysis of historic rainfall records. A dimensionless design storm represents rainfall events commonly observed during major extratropical storms in the Los Angeles area. The storm duration is four days. The maximum rainfall quantity occurs on the fourth day.

Rainfall isohyets show the spatial distribution of rainfall over the county. The isohyets represent the depth of rainfall for a standard design frequency over a specified period of time. Multiplying the unit hyetograph by the rainfall isohyetal depth produces the design storm for a specific area. Figure 1.2.2 shows rainfall isohyets in the County of Los Angeles. This area-specific design storm and an area-specific time of concentration define the time-intensity relationship for a particular subarea. Each subarea requires an area specific time of concentration and design storm.

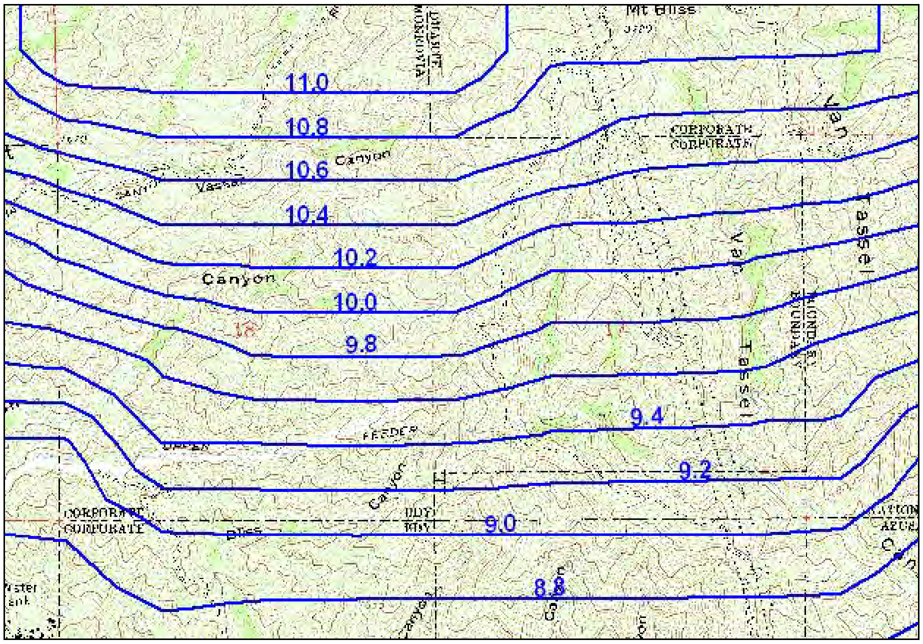

Figure 1.2.2

50-year, 24-hour Rainfall Isohyets in the County of Los Angeles

Calculation of the time of concentration has evolved over time. Currently, time of concentration calculations rely on a regression equation based on the kinematic wave theory.

Reservoir routing of hydrographs for storage uses the Modified Puls method. This method is based on a finite difference approximation of the continuity equation coupled with an empirical representation of the momentum equation.[1] This method is widely used for reservoir routing in hydrologic studies and is the approved method for use within the County of Los Angeles.

Figure 1.2.3 shows Morris Reservoir located in the San Gabriel Mountains.

Figure 1.2.3
Morris Reservoir

[1] US Army Corps of Engineers. Hydrologic Modeling System HEC-HMS Technical Reference Manual. Washington, D.C. 2002

CHAPTER

2

Physical Factors Affecting Hydrology

2.1 TOPOGRAPHY

The County of Los Angeles covers 4,083 square miles and measures approximately 66 miles from east to west and 73 miles from north to south. The topography within the county is 25 percent mountains, 10 percent coastal plain, and 65 percent foothills, valley, or desert. Elevations range from sea level to a maximum of 10,064 feet at the summit of Mount San Antonio. The county is divided into five principal drainage systems: Los Angeles River Basin, San Gabriel River Basin, Santa Clara River Basin, Coastal Basin, and Antelope Valley.

The coastal plain slopes mildly and contains relatively few depressions or natural ponding areas. The slopes of the main river systems crossing the coastal plain, such as San Gabriel River, Los Angeles River, and Ballona Creek, range from 4 to 14 feet per mile.

The mountain ranges within the County of Los Angeles are generally aligned in an east-west direction and are part of the Transverse Ranges. The major range in the county is the San Gabriel Mountains. Most of the mountainous areas lie below 5,000 feet with only 210 square miles above this elevation. The mountainous area is rugged. The deep “V”-shaped canyons with steep walls are separated by sharp dividing ridges. The average slope of the canyon floors ranges from 150 to 850 feet/mile in the San Gabriel Mountains.

2.2 GEOLOGY AND SOILS

The geologic setting of the County of Los Angeles is largely the result of the tectonic plate boundary between the North American and Pacific plates that runs along the northern edge of the county. The San Andreas Fault forms the boundary between these plates and bisects the state in a northwest to southeast direction. In the Los Angeles area, the fault bends to an east-west orientation before returning to its former course. Crustal forces resulting from this change in geometry are uplifting the San Gabriel Mountains. The San Gabriel Mountains experience a high rate of uplift that is being counteracted by high erosion rates. As a result, the county's valleys contain deep deposits of alluvial sediments.[1]

Igneous, sedimentary, and metamorphic rock groups are present within the county. The San Gabriel Mountains and Verdugo Hills are composed primarily of highly fractured igneous rock, with large formations of granitic rock exposed above coarse and porous alluvial soils. Faulting and deep weathering have produced pervious zones in the rock formations. These rock masses have a comparatively shallow soil mantle caused by accelerated erosion on the steep slopes. Figure 2.2.1 illustrates a weathered igneous rock outcrop along Highway 39 in San Gabriel Canyon.

Figure 2.2.1
Weathered Igneous Rock Outcrop Along Highway 39 in San Gabriel Canyon

Other mountainous and hilly areas within the county are composed primarily of folded and faulted sedimentary rocks, including shale, sandstone, and conglomerate. Residual soils in these areas are shallow and are generally less pervious than those of the San Gabriel Mountains.

Valley and desert surface soils are alluvial and grade from coarse sand and gravel near canyon mouths to silty clay and clay in the lower valleys and coastal plain. The alluvium builds up through repeated deposition of debris and reaches depths as great as 2,000 feet. Where there is little clay, this material is quite porous. Impervious lenses and irregularities divide the alluvium into several distinct groundwater basins. Valley soils are generally well drained with relatively few perched water or artesian areas.

2.3 VEGETATIVE COVER AND LAND USE

The principal vegetative cover of upper mountain areas consists of various species of brush and shrubs known as chaparral. Most trees found on mountain slopes are oak. Figure 2.3.1 shows oak trees along a stream in the San Gabriel Mountains. Pine, cedar, and juniper are found in ravines at higher elevations and along high mountain summits. Alder, willow, and sycamore are found along streambeds at lower elevations.

Figure 2.3.1
Oaks Trees Along a Stream in the San Gabriel Mountains

The chaparral is extremely flammable, and extensive burning of the mountain vegetation frequently occurs during dry, windy weather. Chaparral depends on fire to germinate and has the ability to sprout quickly after fire, reestablishing the watershed cover within a period of five to ten years. Figure 2.3.2 shows the revegetation of chaparral after a fire.

Figure 2.3.2
Revegetation of Chaparral After Fire

Grasses are the principal vegetation on the low elevation hills. Most of the hills and valleys have been converted to urban and suburban use in the portion of the county south of the San Gabriel Mountains. Development of the desert areas north of the San Gabriel Mountains and in the Santa Clarita Valley has increased in recent years and is proceeding at an accelerated rate.

2.4 CLIMATE

The climate within the county varies greatly. The windward side of the San Gabriel Mountain range is subtropical while the leeward side in the Mojave Desert is arid. Seasonal, normal precipitation totals for representative areas are shown in Table 2.4.1.

Location	Average Annual Precipitation (in)
Coastal Plain	15.5
San Gabriel Mountains	32.9
Desert – Antelope Valley	7.8

Table 2.4.1
Seasonal Normal Precipitation for Various Climate Zones

Most precipitation occurs between December and March. Precipitation during summer months is infrequent, and rainless periods of several months are common.

Snow rarely falls on the coastal plain. Snowfall at elevations above 5,000 feet frequently occurs during winter storms. This snow melts rapidly except on the higher peaks and north facing slopes.

January and July are the coldest and warmest months of the year, respectively. Table 2.4.2 illustrates the seasonal variation of temperature in the mountain and coastal plain areas.

	Los Angeles (Coastal Plain)	Mt Wilson (San Gabriel Mts)
Average January Minimum Temperature	48°	35°
Average July Maximum Temperature	84°	80°
Record High	112°	99°
Record Low	28°	9°

Table 2.4.2
Characteristic Temperatures of the Mountain and Coastal Plain Areas

2.5 HYDROMETEOROLOGIC CHARACTERISTICS

Hydrometeorological characteristics are greatly influenced by the mountains within the county. Winter storms affect the coastal areas while convective storms affect the desert areas.

Coastal and Mountain Areas

Most precipitation in the Los Angeles area occurs in the winter due to extratropical cyclones from the North Pacific. Major storms consist of one or more frontal systems, extending 500 to 1,000 miles in length. The frontal systems produce rainfall simultaneously throughout the county, occasionally lasting four days or longer.

These storms approach Southern California from the west or southwest with southerly winds that continue until the front passes. The mountain ranges lie directly across the path of the inflowing warm, moist air. The coastal and inland ranges cause the warm air to rise. As it rises, precipitation forms and falls. This orographic effect intensifies rainfall along the mountains and coastal areas. As a result, rainfall intensities and totals in these areas increase. The effect of snow melt on flood runoff is significant only in the few cases where warm spring rains from southerly storms fall on a snow pack. Temperatures throughout the county usually remain above freezing during major storms. Figure 2.5.1 is a view of the coastal area within the County of Los Angeles.

Figure 2.5.1
Coastal Area

Desert Areas

Orographic precipitation over the mountains produces a rain shadow on the leeward side of the mountains. As a result, the northern San Gabriel Mountains and the Mojave Desert regions experience primarily summer convective rainfall. The most serious floods in many desert areas may result from convective summer storms. Figure 2.5.2 shows a view of the desert area within the County of Los Angeles.

Figure 2.5.2
Desert Area Near Lancaster

2.6 RUNOFF CHARACTERISTICS

Runoff characteristics are influenced by soil type, slope, vegetation, and many other conditions. General regions behave differently based on these factors and runoff varies greatly between mountain and valley areas.

Mountain Areas

Steep canyon walls and channel slopes rapidly concentrate storm runoff in mountainous areas. Depression and detention storage effects are minor in this rugged terrain.

The moisture content of mountain soils has a pronounced effect on runoff during a storm. Precipitation during periods of low soil moisture is almost entirely absorbed by the porous soils. Soil moisture is lowest at the beginning of the rainy season due to evapotranspiration during the preceding summer months. Significant surface runoff does not occur until soil moisture is near field capacity, except during extremely intense rainfall. Consequently, in certain areas, significant runoff occurs as subsurface flow, or interflow, rather than direct runoff. Most streams in the county are intermittent. Natural year-round perennial discharge is mostly limited to springs in portions of the San Gabriel Mountains.

Hill and Valley Areas

Runoff concentrates rapidly below the generally steep slopes in hilly areas. Runoff rates from undeveloped hilly areas are normally smaller than those from mountain areas of the same size. Development in hilly areas decreases runoff concentration times considerably due to increased channelization. Runoff volumes and rates increase due to increased imperviousness.

Debris production from undeveloped hilly areas is normally less than debris production from mountainous areas of the same size. Increased development reduces erosion and limits debris in storm flow.

Figure 2.6.1 shows a hilly area located in the Santa Clara River Watershed.

Figure 2.6.1
Hills in Santa Clara River Watershed

Runoff in the valleys and coastal plain is affected by ponding and spreading of flows. Valley areas are affected by development. In highly developed valley areas, local runoff volumes increase as impervious materials replace the soil. Peak runoff rates for valley areas increase due to the elimination of natural ponding areas and improved hydraulic efficiency. Conveyances, such as streets and storm drain systems carry the water to the ocean more rapidly and do not allow infiltration. Figure 2.6.2 shows a view of the Los Angeles basin from the San Gabriel Mountains.

Figure 2.6.2
Los Angeles Basin from the San Gabriel Mountains

[1] *San Gabriel River Corridor Master Plan, March 2004.*

CHAPTER

3

Major Watersheds and Tributaries

There are five major watersheds within the County of Los Angeles. Four of these drain to the ocean and the fifth enters dry lakes in the desert. The watersheds are unique and are developed to different extents. Watershed descriptions and a location map shown in Figure 3.1 are provided to help understand the hydrologic conditions within each watershed.

Figure 3.1
Major Watersheds in the County of Los Angeles

3.1 LOS ANGELES RIVER[1]

The Los Angeles River Watershed covers over 830 square miles. The watershed includes the western portion of the San Gabriel Mountains, the Santa Susana Mountains, the Verdugo Hills, and the northern slope of the Santa Monica Mountains. The river flows from the headwaters in the western San Fernando Valley and outlets in San Pedro Bay near Long Beach. The river crosses the San Fernando Valley and the central portion of the Los Angeles Basin. The watershed terrain consists of mountains, foothills, valleys, and the coastal plain.

The Los Angeles River and many of its tributaries have been the subject of extensive engineering work to reduce the impacts of flood events. Prior to development, the Los Angeles River system was typical of other streams in the southwest. The river's channel was broad and often shifted location within the flood plain due to the high sediment loads. The stream location within the coastal plain has varied greatly over the years. Between 1815 and 1825, the river changed course completely. Breaking its banks in what is now Downtown Los Angeles, the river followed the course of Ballona Creek, reaching the ocean at a location 20 miles from its current outlet.

Numerous flood control facilities were constructed in the early 20th century, as development began to take place on this wide flood plain. The concrete sections of the Los Angeles River were constructed between the late 1930's and the 1950's. Channel improvements and extensive watershed development decrease times of concentration and increase runoff flow rates and volumes. The Los Angeles County Flood Control district constructed three major dams during this period: Pacoima, Big Tujunga and Devil's Gate. The dams were built to reduce downstream flow rates and conserve water for ground water recharge purposes. In the Rio Hondo drainage area, several dams were constructed including Eaton Wash, Sierra Madre, Santa Anita and Sawpit. Additionally, the U.S. Army Corps of Engineers operates four major dams in the watershed to assist in flood control. The four dams are Hansen, Lopez, Sepulveda and Whittier Narrows. Figure 3.1.1 is a view of Big Tujunga Dam after the January 2005 storms.

Figure 3.1.1
Big Tujunga Dam
January 11, 2005

The parts of the San Gabriel Mountains tributary to the Los Angeles River contain some of the most prolific sediment producing streams in the world. Intense rainfall, coupled with highly erodible sediment, produces damaging debris discharges. Numerous debris basins have been constructed along the foothills of the San Gabriel Mountains to remove sediment from the flow.

The Los Angeles River Watershed has a diverse land use pattern. The upper portions of the watershed are covered by Angeles National Forest and other rural areas. The remainder of the watershed is highly developed. The watershed has large areas of commercial, residential, and industrial development. Few parks or natural areas exist in the lower watershed.

The major tributaries of the Los Angeles River include Burbank Western Channel, Pacoima Wash, Tujunga Wash, and Verdugo Wash in the San Fernando Valley; and the Arroyo Seco, Compton Creek, and Rio Hondo in the Los Angeles Basin. Much of this tributary network has also been lined with concrete to meet flood control needs. Figure 3.1.2 shows a view of the Los Angeles River at Willow Street.

Figure 3.1.2
Los Angeles River
At Willow Street

3.2 SAN GABRIEL RIVER

The San Gabriel River Watershed is located in the eastern portion of the county. The river drains the San Gabriel Mountains to the north and is bounded by the Los Angeles River Watershed and Santa Ana River Watersheds. The watershed drains 640 square miles. The watershed outlets into the Pacific Ocean between Long Beach and Seal Beach after passing through the Alamitos Bay estuary. Tributaries to the San Gabriel River include: Walnut Creek, San Jose Creek, and Coyote Creek.

The upper portions of the watershed are contained almost entirely within the Angeles National Forest and are nearly untouched by development. The mountains in this area are extremely rugged with steep V-shaped canyons. The vegetation is dominated by chaparral and coastal sage scrub with patches of oak woodlands. Conifers are dominant at higher elevations. The streambeds in the area contain sycamore and alder woodlands.[2]

In contrast, the lower part of the watershed is mostly developed below the mouth of the San Gabriel Canyon. The developments include commercial, residential, and industrial use. The developed area in the San Gabriel Valley and Los Angeles Basin comprises 26% of the total watershed area. Figure 3.2.1 shows the upper natural portion of the San Gabriel River.

Figure 3.2.1
Upper Portion of the San Gabriel River

Similar to the Los Angeles River, the San Gabriel River once occupied a wide floodplain and shifted course to accommodate large flows and sediment loads. Development of the floodplain required changing the character of the river dramatically since periodic inundation of the floodplain was not compatible with the new land uses.

Several major dams and debris basins impound floodwaters and prevent debris flows originating in the San Gabriel Mountains. These include Cogswell Dam, San Gabriel Dam, Morris Dam, Big Dalton Dam, San Dimas Dam, Live Oak Dam, and Thompson Creek Dam. Many of these facilities were constructed in the 1930's and have proven their worth by preventing significant damage from large flood events. Major flood events occurred in 1938, 1969, 1978, 1983, 1998, and 2005. Additionally, the U.S. Army Corps of Engineers operates the Santa Fe Dam and Whittier Narrows Dam in the watershed to assist in flood control. Figure 3.2.2 shows the San Gabriel Dam at full capacity.

Figure 3.2.2
San Gabriel Dam at Full Capacity

The San Gabriel River has been channelized below Santa Fe Dam to aid in flood prevention. However, the channel invert was left unlined for much of its length between Santa Fe Dam and Florence Avenue in Downey. The unlined bottom promotes infiltration of flood waters released from upstream dams. Public Works installed rubber dams to further utilize the river bottom for ground water recharge. Figure 3.2.3 is a rubber dam located in the lower portion of the river.

Figure 3.2.3
Rubber Dam Located in the Lower Portion of the San Gabriel River

The most significant spreading ground facilities in the county are located in the San Gabriel River watershed. Runoff resulting from storm events is diverted into the spreading facilities and allowed to recharge groundwater. Major spreading grounds are located at the mouth of San Gabriel Canyon and in the Montebello area downstream of the Whittier Narrows Dam.

3.3 SANTA CLARA RIVER

The Santa Clara River originates in the northern slopes of the San Gabriel Mountains at Pacifico Mountain and travels west into Ventura County, discharging into the Pacific Ocean near the City of Ventura. The river runs approximately 100 miles from the headwaters near Acton, California, to the ocean. The river drains an area of approximately 1,600 square miles.

The upper portion of the river, within the County of Los Angeles, has a watershed area of approximately 644 square miles. Ninety percent of this area is mountainous with steep canyons; while the remaining ten percent is alluvial valleys.[3] The area is mostly undeveloped with a large portion in the Angeles National Forest. There are some mixed-use developed areas concentrated in or near the City of Santa Clarita. The watershed is currently experiencing an accelerated rate of development in areas adjacent to the river. Figure 3.3.1 shows the Santa Clara River after the 1978 storms.

Figure 3.3.1
Santa Clara River Downstream of Magic Mountain Parkway
March 4, 1978

The Santa Clara River and its tributaries are ephemeral streams characterized by alluvial soils. Discharge occurs quickly during rainfall events and diminishes quickly after rainfall has ceased. As in other county watersheds, the mountain and foothill areas are susceptible to debris-laden flows during intense rainfall, especially when a watershed is recovering from fire.[4]

The river remains in a generally natural state with some modifications related to the development of the floodplain. The expected population increase will continue to produce floodplain encroachment, requiring additional bank protection, channelization, and channel crossings. The expected population increase, as well as increased imperviousness, will impact the hydrologic characteristics of the river and the sediment balance.

Some of the major tributaries in the county's portion of the Santa Clara River watershed include: Castaic Creek, San Francisquito Canyon, Bouquet Canyon, Sand Canyon, Mint Canyon, and the South Fork of the Santa Clara River.

3.4 COASTAL[5]

The Coastal watershed is comprised of a number of individual watersheds that outlet into Santa Monica and San Pedro Bays. These include the major watersheds of Malibu Creek, Topanga Creek, Ballona Creek, and the Dominguez Channel. These watersheds have unique topographic and hydrologic characteristics ranging from undeveloped to highly urbanized. For simplicity, these coastal watersheds are grouped together due to their relatively small sizes.

The Malibu Creek Watershed is comprised of 109 square miles at the western end of the County of Los Angeles and extends into Ventura County. Most of the watershed is undeveloped public land. There is sporadic but increasing development throughout the area. The most extensive development is centered along US Highway 101. The northern portion is hilly while the southern portion, near the ocean, is rugged mountain terrain. Malibu Creek drains into the Pacific Ocean near the Malibu Civic Center. A portion of Malibu Creek is shown in Figure 3.4.1.

Figure 3.4.1
Malibu Creek

Topanga Creek drains 18 square miles in the central Santa Monica Mountains. The watershed is primarily rural with widely scattered residential and commercial development. The creek flows unobstructed along its course and empties into the Santa Monica Bay in an unincorporated portion of the county east of Malibu.

Ballona Creek is a flood control channel that drains the western Los Angeles basin. The watershed area is bounded by the Santa Monica Mountains on the north and the Baldwin Hills on the south. It extends east nearly to downtown Los Angeles. The total watershed area is roughly 130 square miles. The area is primarily developed but includes undeveloped areas on the south slope of the Santa Monica Mountains. The land use is 64%

residential, 8% commercial, 4% industrial, and 17% open space. The major tributaries to Ballona Creek include: Centinela Creek, Sepulveda Canyon Channel, Benedict Canyon Channel, and numerous storm drains. The watershed drains into Santa Monica Bay at Marina del Rey.

Figure 3.4.2 is a view of the concrete lined portion of Ballona Creek.

Figure 3.4.2
Ballona Creek

The Dominguez Watershed is comprised of approximately 133 square miles in the southern portion of the county. The watershed extends from near the Los Angeles International Airport to the Los Angeles Harbor. The area is almost completely developed with regions of residential, commercial, and industrial land use. The storm drains and flood control channel network, as opposed to natural drainage features, define the watershed.

There are many other smaller watersheds in the Coastal drainage area that drain developed and undeveloped areas directly to the Pacific Ocean.

3.5 ANTELOPE VALLEY

The Antelope Valley encompasses approximately 1,200 square miles in the northern portion of the County of Los Angeles. The valley is bounded on the north by the Tehachapi Mountains and on the south by the Sierra Pelona and the San Gabriel Mountains. Numerous streams from the mountains and foothills flow across the valley floor. The valley lacks defined drainage channels outside of the foothills and is subject to unpredictable drainage patterns.

Nearly all the surface water runoff from the Los Angeles portion of the Antelope Valley accumulates on Rosamond Dry Lake near the Kern County Line. A small portion is tributary to other dry lakes in the area. This 20 square mile playa is dry during most of the year, but is likely to be flooded during prolonged periods of winter precipitation. Surface runoff, as well as discharges from groundwater, remain on the dry lake until removed by infiltration and evaporation. Anecdotal evidence indicates that at times the playa may be underwater for up to five months at a time, as occurred during the winter of 1965-66.

The valley contains the developed areas of Lancaster and Palmdale. The remainder of the valley is sparsely developed. However, the valley is one of the most rapidly developing areas in the county. Rapid development is likely to continue for some time. This development will significantly alter the hydrologic characteristics of the basin.

A view of Antelope Valley is shown in Figure 3.5.1.

Figure 3.5.1
Antelope Valley

1 *The Los Angeles River Master Plan.* "Flood Management and Water Conservation". Los Angeles County Department of Public Works. Approved June 13, 1996.

2 *San Gabriel River Corridor Master Plan,* March 2004, pages 2-4.

3 "Hydrologic Model of the Santa Clara River and its Tributaries". David Ford Consulting. December 1999.

4 "Hydrologic Model of the Santa Clara River and its Tributaries". David Ford Consulting. December 1999.

5 See North Santa Monica Bay Watersheds White Paper, November 6, 2003; Dominguez Watershed Management Master Plan, April 2004

6 Dettling, C., R.H. French, J.J. Miller, and J. Carr (2004). An Approach to Estimating the Frequency of Playa Lake Flooding.

CHAPTER

4

Policy on Levels of Protection

4.1 DEPARTMENT POLICY MEMORANDUM

A Department of Public Works memorandum dated March 31, 1986, General Files No. 2-15.321, established the policy on levels of flood protection. This policy describes degrees of flooding and which design storms should be used for certain conditions and structures. Chapter 5 defines the design storms for use in the County of Los Angeles.

4.2 CAPITAL FLOOD PROTECTION

The Capital Flood is the runoff produced by a 50-year frequency design storm falling on a saturated watershed (soil moisture at field capacity). A 50-year frequency design storm has a probability of 1/50 of being equaled or exceeded in any year. Capital Flood protection also requires adding the effects of fires and erosion under certain conditions. This section describes specific criteria for applying the burning and bulking requirements for Capital Flood protection.

The following sections describe facilities and structures required to meet the Capital Flood level of protection.

Natural Watercourses

The Capital Flood level of protection applies to all facilities, including open channels, closed conduits, bridges, dams, and debris basins not under State of California jurisdiction. These facilities must also be constructed in or intercept flood waters from natural watercourses. Facilities under the State of California jurisdiction must also meet the state's criteria, which may include the Probable Maximum Flood criteria described in Section 4.4.

A natural watercourse is a path along which water flows due to natural topographic features. For definition purposes, a natural watercourse drains a watershed greater than 100 acres. Natural watercourses have not been subject to major engineering works such as channel realignment or bank protection. The watercourse must also meet one or more of the following conditions during a Capital Flood:

1. Flow velocities greater than 5 ft/sec.

2. Flow depths greater than 1.5 feet.

Replacement of the natural watercourse with flood control facilities that do not provide the Capital Flood level of protection requires water surface elevation analysis. The water surface elevation must be at least one foot below the base of existing dwellings adjacent to the channel. The construction must also meet the requirement of the National Flood Insurance Program described in Section 4.6. An example of a natural watercourse in Bouquet Canyon is shown in Figure 4.2.1.

Figure 4.2.1
Bouquet Canyon
Natural Watercourse
in June 2005

Floodways

The Capital Flood applies to all areas mapped as floodways. See Section 4.6 for more information on floodways.

Natural Depressions or Sumps

The Capital Flood level of protection applies to all facilities constructed to drain natural depressions or sumps. These facilities include channels, closed conduits, retention basins, detention basins, pump stations, and highway underpasses. A depression or sump is an area from which there is no surface flow outlet and must meet one or more of the following conditions during a Capital Flood:

1. Ponded depth of 3 feet or greater.

2. Ponded water surface elevations within one foot below the base of adjacent dwellings resulting from construction of facilities with less than the Capital Flood capacity. This condition does not apply if ponded water can escape as surface flow before reaching the base of adjacent dwellings during the Capital Flood.

Figure 4.2.2 shows an example of a flooded sump at the intersection of San Fernando Road and Tuxford Street in Sun Valley.

Figure 4.2.2
Flooded Sump at Intersection of San Fernando Road and Tuxford Street
January 9, 2005

Sumps with drainage from roadways require special care. If flows reach the sump by following the roadway from upstream, use the Capital Flood on all areas upstream of the sump that drain to the roadway. The roadway must carry the Capital Flood capacity with a water surface elevation below the private property line. Otherwise, drainage facilities must be added beneath the roadway. See the Los Angeles County Highway Design Manual[1], and Chapter 44 of the Land Development Division Guidelines.

Culverts

The Capital Flood level of protection applies to all culverts under major and secondary highways.

Tributary Areas Subject to Burning

Canyons and mountainous areas within the County of Los Angeles are subject to burning. The Capital Flood applies to all areas likely to remain in a natural state, regardless of size. Burned canyons and mountainous areas also add debris to the runoff. Therefore, flow from "burned" areas must be "bulked." Bulking reflects increases in runoff volumes and peak flows related to inclusion and transport of sediment and debris.

Section 6.3 discusses the development of burned watershed hydrology. Section 3.3 of the Public Works' Sedimentation Manual contains information on bulking flows.

4.3 URBAN FLOOD PROTECTION

All drainage facilities in developed areas not covered under the Capital Flood protection conditions must meet the Urban Flood level of protection. The Urban Flood is runoff from a 25-year frequency design storm falling on a saturated watershed. A 25-year frequency design storm has a probability of 1/25 of being equaled or exceeded in any year.

Street flow due to the urban flood may not exceed the private property line elevation. However, runoff can be conveyed in drains under the street and on the street surface. Urban Flood runoff is allowed to flow in the street to the point where the flow reaches the street capacity at the property line. Depth analysis is to be started at the upstream end of the watershed. The flow should be split to allow conveyance in the street and in a drain below the street when flows exceed street capacity. Drains must at least carry flow from the 10-year frequency design storm. See the Los Angeles County Highway Design Manual[1] and Chapter 44 of the Land Development Division Guidelines for road design requirements.

The street or highway must carry the balance of the 25-year frequency design storm below the property line. The drain may carry more flow to lower the water surface on the street to below the private property line or meet other requirements for vehicular or pedestrian traffic. See the Los Angeles County Highway Design Manual for the traffic requirements[1]. The maximum allowable pipe diameter for hydrology studies is 96 inches. Beyond this size, choose a rectangular channel conveyance. Figure 4.3.1 provides an example of street flow.

Figure 4.3.1
Street Flow After 1938 Storm

4.4 PROBABLE MAXIMUM FLOOD PROTECTION

The Probable Maximum Flood (PMF) results from the most severe combination of critical meteorological and hydrologic conditions that are reasonably possible in the region[2]. The Probable Maximum Precipitation[3] (PMP) represents the greatest depth of rainfall theoretically possible for a given duration over a given drainage basin. The PMF occurs when the PMP falls over watersheds that have reached field capacity (saturated) conditions.

California's Division of Safety of Dams (DSOD) requires a PMF analysis for dams and debris basins that hold at least 1,000 acre-feet, are 50 feet or higher, would require at least 1,000 people to be evacuated, and have a damage potential of $25,000,000 or more. Most dams and debris basins (earth embankment, concrete, or other materials) in the County of Los Angeles must safely pass the PMF[4]. Figure 4.4.1 shows a chart of the State's height and storage parameters that define dam jurisdiction[5]:

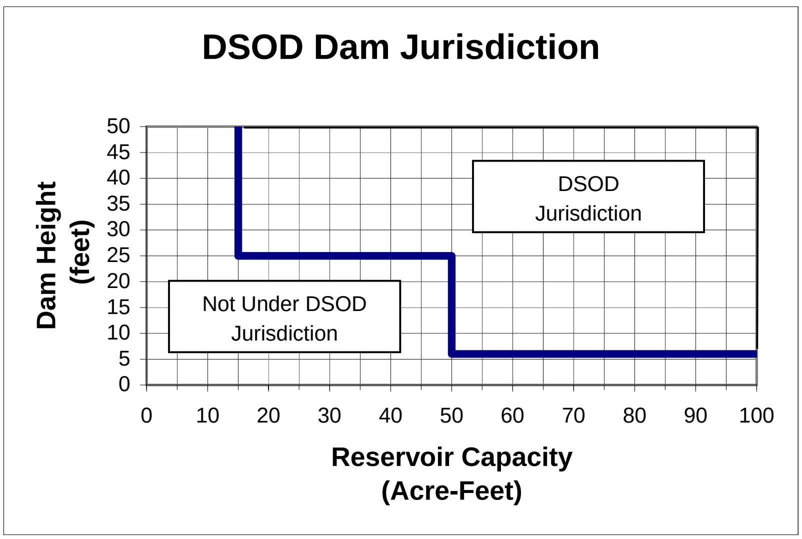

Figure 4.4.1
Dam Jurisdiction Chart

Spillway sizing requirements for dams and debris basins is available through the California Department of Water Resources, Division of Safety of Dams[4]. Figure 4.4.2 is a picture of Morris Dam, constructed in 1932, which falls under DSOD jurisdiction.

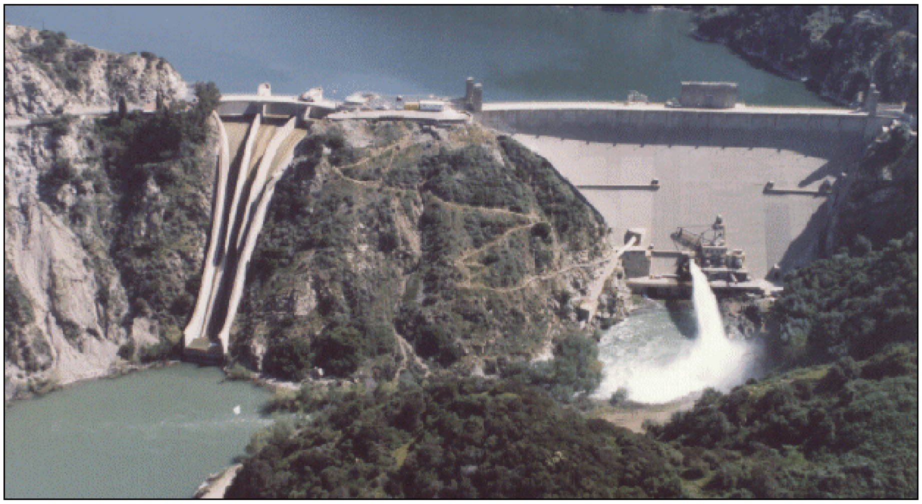

Figure 4.4.2
Morris Dam
1993

4.5 NATIONAL FLOOD INSURANCE PROGRAM

The National Flood Insurance Program (NFIP) set the 100-year flood as the standard for flood insurance protection. The 100-year flood relies on historic runoff records for definition. The standard makes no allowance for future urbanization or the possible inclusion of debris in the flow. In flood hazard areas, the federal standard requires the finished floor elevation of proposed dwellings to be at least 1 foot above the water surface elevation of the 100-year flood[5]. The Base Flood Elevation (BFE) refers to the water surface elevation of the 100-year flood on the pre-developed condition.

Public Works uses the Capital Flood peak flow rate for Los Angeles County floodway mapping standards. FEMA Flood Insurance Rate Maps (FIRM Maps) are available at: http://www.ladpw.org/apps/wmd/floodzone. More information about the NFIP level of protection requirements are available at the www.fema.gov/nfip/ website.

The floodway is determined using the 1-foot rise criterion. Some misinterpret this to mean that development in a floodway is permitted if it does not raise the BFE more than one foot. Floodplain management regulations dictate that any rise in the BFE, as a result of a floodway encroachment, is unacceptable without a Conditional Letter of Map Revision[6]. FEMA provides guidelines and standards for flood hazard mapping and requirements to meet the NFIP level of protection. More information on the FEMA requirements is found at http://www.fema.com/fhm/gs_main.shtm.

4.6 COMPATIBILITY WITH EXISTING SYSTEMS

The level of protection standards may require modification if the receiving system has limited capacity at the proposed drain's outlet. If the receiving drain will be replaced or relieved in the future, size the proposed drain for the appropriate level of protection. The proposed drain capacity is restricted to match the capacity available in the downstream drain when no future relief is planned.

Solutions to the situations with restricted capacities require project specific decisions. The Design Division of Public Works should review the proposed drainage system and the outlet conditions to determine the compatible level of protection.

4.7 EXISTING LEVEL OF FLOOD PROTECTION

Sub-surface drainage often replaces surface drainage when land is developed. Replacing or modifying surface drainage systems requires maintaining or increasing the original level of flood protection. The total capacity, sub-surface and surface, must equal or exceed the original surface capacity. Adequate surface drainage capacity must be retained if the proposed sub-surface drain provides a lower level of protection than the original surface drainage system.

4.8 MULTIPLE LEVELS OF FLOOD PROTECTION

There are numerous instances where a drainage system must provide more than a single level of flood protection. Drainage systems must meet the criteria described in this chapter of the Hydrology Manual.

For example, there may be a natural canyon area tributary to a proposed drainage system that drains an urban area containing a sump. The proposed drainage system must convey the burned and bulked Capital Flood flow from the canyon area, protect the sump from a Capital Flood, and protect the developed area from the Urban Flood. Refer to Table 4.1.1 of the Sedimentation Manual to determine if a structure, such as a debris basin, is needed for the natural canyon. If a structure is needed, then only the burned flow is carried through the drainage system.

Figure 4.8.1 is an example of a debris basin.

Figure 4.8.1
Sawpit Debris Basin
January 11, 2005
(Courtesy of Leopoldo A. Herrera)

1 Los Angeles County Highway Design Manual 5th edition. 2001.
2 US Army Corps of Engineers. Flood-Runoff Analysis (EM 1110-2-1417). page 13-7. Washington, D.C. 1994.
3 US Department of Commerce, National Oceanic and Atmospheric Administration, US Army Corps of Engineers. Hydrometeorological Report Number 59. Probable Maximum Precipitation for California. 1999.
4 Calzascia and Fitzpatric. Hydrologic Analysis Within California's Dam Safety Program. California Department of Water Resources, Division of Safety of Dams. http://wwwdsod.water.ca.gov/tech-ref/fitz-paper.pdf
5 National Flood Insurance Program Flood Insurance Manual. Federal Emergency Management Agency. October 2004.
6 Dyhouse, G., J. Hatchett, J. Benn. Floodplain Modeling Using HEC-RAS. Haestad Methods. Connecticut. 2003.

CHAPTER

5

Rainfall and Design Storm Characteristics

The Department of Public Works' hydrologic method uses a design storm derived from historic rainfall data. Observed major extratropical storms in the Los Angeles region provided a pattern for the design storm. The storm does not represent an actual event but is an idealized series of precipitation data that fits a specific design objective. The design storm is a composite determined by analysis of regional rainfall patterns. Three components define the design storm: an Intensity-Duration-Frequency (IDF) equation, a temporal distribution, and a spatial rainfall distribution.

Public Works developed the rainfall distribution and design storms in 2002. A network of approximately 250 rain gages allowed an accurate definition of the spatial and temporal variability of rainfall over the county. The average historic record length for these gages is 75 years.

Data analysis provided the three components needed for the design storm. Analysis of rainfall data within the county provided the IDF equation, which is a relationship between rainfall intensity, duration, and frequency. Then a 24-hour temporal distribution was established using the IDF relationship. The 24-hour temporal distribution is represented by the unit hyetograph, which plots rainfall intensity versus time. Finally, a set of isohyets was established for the county. The isohyets represent rainfall depths for a specific duration and frequency and are applied to the unit hyetograph. The result is a hyetograph for a given location and recurrence interval, which is the design storm for a specific subarea.

5.1 RAINFALL INTENSITY-DURATION-FREQUENCY

The fundamental unit of rainfall is depth. Rain gages directly measure depth. Measuring depth and time provides intensity. Intensity is the amount of rain that has fallen per unit of time. The average intensity is calculated by dividing a rainfall depth by the duration, the time over which the rainfall accumulated. The average intensity is:

Equation 5.1.1

$$\text{Intensity} = \frac{\text{Rain Depth}}{\text{Duration}}$$

The peak intensity produces the largest runoff rate. If rainfall were constant throughout a storm, any duration less than the storm duration would produce the same intensity. However, rainfall is rarely constant for the storm duration and intensity varies.

Table 5.1.1 shows the calculated intensity for various durations. Intensities are calculated using the rainfall depth and storm times in the first two rows. Each of the duration rows show intensities calculated based on different durations. For example, I_5 is the intensity calculated over a period of 5 minutes starting at t = 0 and ending at t = 5 minutes, or starting at t = 5 and ending at t = 10 minutes, etc. Bold text denotes the maximum intensity for each intensity duration. The table shows a decrease of maximum intensity as duration increases for a storm with non-uniform precipitation.

Table 5.1.1
Rainfall Intensity Calculations for Various Durations

	Storm Time (minutes)	0	5	10	15	20	25	30	35	40	45	50	55	60
	Cumulative Precipitation (in)	0	0.5	1.5	2.0	2.25	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
Durations	I_5 (in/hr)	-	6.0	12.0	6.0	3.0	3.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	I_{10} (in/hr)	-	-	9.0	9.0	4.5	3.0	1.5	0.0	0.0	0.0	0.0	0.0	0.0
	I_{30} (in/hr)	-	-	-	-	-	-	5.0	4.0	2.0	1.0	0.5	0.0	0.0
	I_{60} (in/hr)	-	-	-	-	-	-	-	-	-	-	-	-	2.5

Design decisions often require assigning a probability of occurrence to the rainfall event. Statistical analysis of rainfall intensity data yields a probability that such a rainfall will occur in a given year. The reciprocal of this probability is the frequency. The frequency represents the time between two occurrences of a specific rainfall event. The rainfall frequency is inversely proportional to the size of the event. Large rainfall events are much less common than small rainfall events.[1]

A study of rain gage data provided relationships between intensity, duration, and frequency within the County of Los Angeles. The study analyzed historic records for 107 rain gages and determined the maximum intensities for rainfall durations of 5, 10, 15, 30, 60, 120, 180, 240, 300, 720, and 1440 minutes. The analysis looked at the frequencies associated with the various intensities. Each intensity was assigned frequencies of 2-, 5-, 10-, 25-, 50-, 100-, and 500-years based on the Gumbel extreme value distribution of each gage.

The 1440 minute, or 24-hour duration, was a primary focus of this analysis. Sets of factors were developed to relate the rainfall depths of various frequencies to the 50-year rainfall frequency. Section 5.3 details the development of these factors.

The normalized intensity equation relates the intensity, duration, and frequency (IDF). The Hydrologic Method authorization memorandum outlines development of the equation.[2] Equation 5.1.2 provides the normalized IDF relationship:

$$\frac{I_t}{I_{1440}} = \left(\frac{1440}{t}\right)^{0.47}$$ **Equation 5.1.2**

Where:

	t	= Duration in minutes
	I_t	= Rainfall intensity for the duration in in/hr
	I_{1440}	= 24-hour rainfall intensity in in/hr
	$\frac{I_t}{I_{1440}}$	= Peak normalized intensity, dimensionless

Equation 5.1.2 allows calculation of the peak-normalized intensity for durations from 5 to 1440 minutes. For durations less than 5 minutes, I_t / I_{1440} = 14.32. Figure 5.1.1 graphically presents the peak-normalized intensity for durations of 5 minutes to 30 minutes.

In addition to its role in defining the design storm, Equation 5.1.2 is used to calculate the peak intensity for time of concentration calculations described in Section 7.3. The equation calculates the intensity for any duration when the 24-hour rainfall intensity is known. Section 5.4 contains an example that illustrates the use of Equation 5.1.2 and Table 5.1.1 to determine the 25-year, 10-minute intensity from the 50-year, 24-hour rainfall isohyetal data.

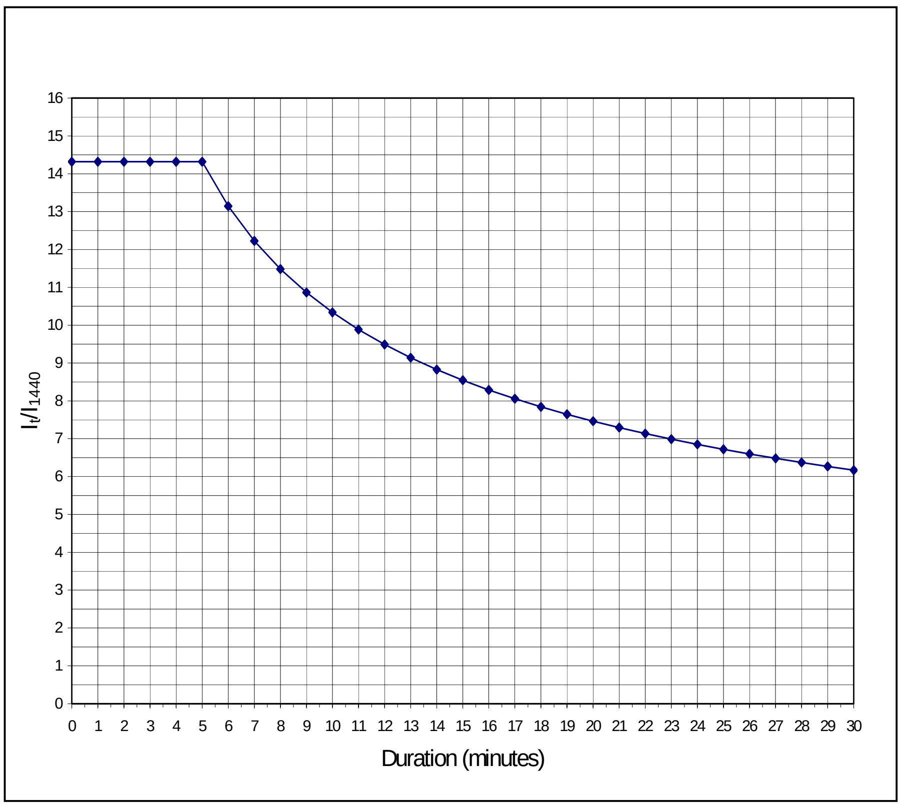

Figure 5.1.1
Normalized Intensity Curve

5.2 UNIT HYETOGRAPH

The definition of a design storm requires a description of how rainfall occurs over time. Public Works' design storm uses a 24-hour cumulative unit hyetograph to describe the temporal distribution of precipitation. The unit hyetograph provides the temporal distribution of one inch of rainfall occurring over a 24-hour period. Figure 5.2.1 shows an example of a cumulative hyetograph and its accompanying incremental hyetograph.

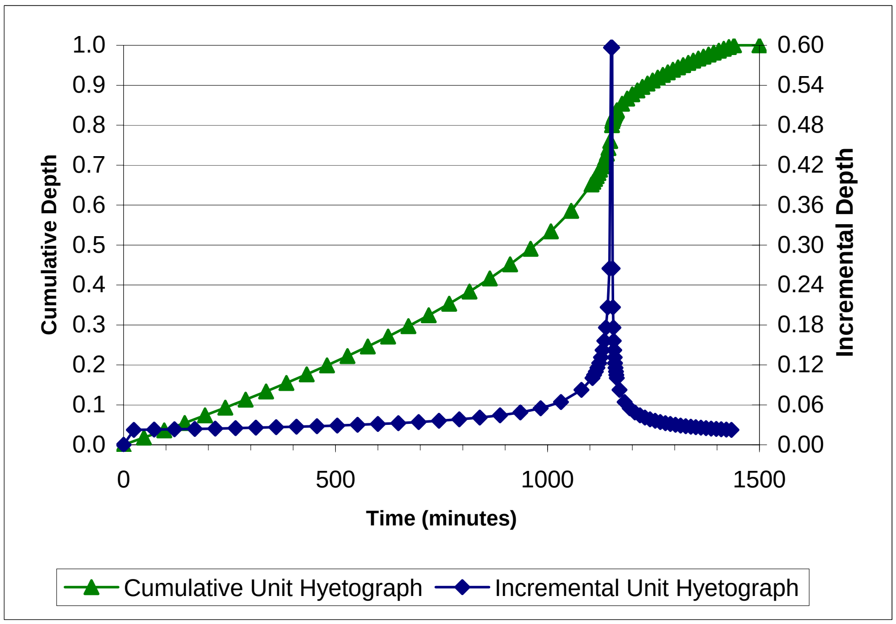

Figure 5.2.1
Relationship Between Cumulative and Incremental Unit Hyetographs

The unit hyetograph is scaled to match design rainfall depths. Design storm rainfall depths are determined from isohyets based on hydrologic design standards. Construction of the hyetograph used the normalized intensity equation solutions with an assumption about where the inflection point of the cumulative hyetograph occurs.

Development of the rainfall hyetograph used a modified alternating block method. See *Applied Hydrology* for a description and example of the alternating block method.[3] Modifications resulted from the use of the normalized intensity curve, instead of a traditional IDF curve, and the regionally specific location of the inflection point. This process produces an incremental unit rainfall distribution for a 24-hour period. The cumulative distribution is developed by summing the incremental distribution at each time step.

Developing the unit hyetograph using the IDF equation required an assumption about the timing of the most intense rainfall. The inflection point of the cumulative unit hyetograph represents the highest intensity. An analysis of the hourly distribution of large historical 24-hour events showed rainfall intensities increasing during the first 70 to 90 percent of the period and decreasing for the remaining time. Approximately 80 percent of the total 24-hour rainfall occurs within the same 70 to 90 percent of the period.

The unit hyetograph assumes the rainfall inflection point occurs when 80 percent of the 24-hour rainfall total has fallen and 80 percent of the 24-hour period has elapsed. Ratios of the depth at a given time relative to the total 24-hour depth were derived from the intensity equation. These ratios were then used to define the unit hyetograph curve. The depth ratios shown in Figure 5.2.1 were calculated at 5-minute time steps from 5 to 60 minutes and 60-minute time steps between 60 and 1440 minutes.

The rainfall depth ratios for each intensity were placed on either side of the inflection point. The alternating blocks were placed around the inflection point. However, instead of alternating the blocks on either side with decreasing intensity, the depth ratios for each time step were split with 20 percent of depth for each time step after the inflection point and 80 percent before the inflection point. The distribution of the time steps was similarly divided using 80 percent before the time of inflection and 20 percent after. Table 5.2.1 illustrates the first few intervals in this process:

Table 5.2.1
Rainfall Distribution Around Hyetograph Inflection Point

t	(D_t/D_{1440})	t*20%	$0.8+(D_t/D_{1440})$*20%	t*80%	$0.8-(D_t/D_{1440})$*80%
5	0.0497	1	0.8099	4	0.7602
10	0.0717	2	0.8143	8	0.7425
15	0.0890	3	0.8178	12	0.7287

With the inflection point at 80 percent of the time (1152 minutes) and 80 percent of the rainfall depth (0.8), the t = 5 time step contributes a point above the inflection point at 1153 minutes, 0.8099 and below the inflection point at 1148 minutes, 0.7602. Continuing this process provides the points that define the entire design unit hyetograph.

As described in Section 2.5.1, most major precipitation events in the county are the result of extratropical winter storms. Significant runoff tends to occur when these storms last several days and are comprised of several individual bands of intense precipitation. In the case of a multiple day storm, the most intense rainfall tends to occur on the last day. These observations form the basis for Public Works' 4-day design storm.

The unit hyetograph is multiplied by the 24-hour rainfall depth to produce a rainfall hyetograph for the fourth day. The first through third days have respectively 10, 40, and 35 percent of the fourth day's rainfall. Appendix A contains the unit hyetograph in tabular form. Multiplying the unit hyetograph by the depth for each day results in the daily hyetograph.

5.3 RAINFALL ISOHYETS

Historical data indicates that spatial distribution of precipitation across the county is not uniform during storm events. To account for this spatial variability of rainfall, Public Works developed rainfall isohyetal maps for the County of Los Angeles.

Isohyetal maps show the 24-hour rainfall depths expected for the 50-year storm frequency. The rainfall pattern depicted on these maps shows the influence of topography on rainfall.

The isohyetal maps incorporate information from Public Works' rain gages and the National Oceanic and Atmospheric Administration's (NOAA) gridded rainfall maps of the area. The process used NOAA's *Atlas 2,* 2-year, 24-hour isohyetal data to provide the spatial rainfall pattern. NOAA is a widely accepted source for meteorological data, and NOAA *Atlas 2* is a recognized standard for spatial rainfall distribution data.

Detailed rain gage analysis was performed to determine the various rainfall depth and frequency relationships. Table 5.3.1 summarizes the relationship between various frequencies as factors of the 50-year frequency depths. The factors are normalized to the 50-year event because this event is used for Capital Flood Hydrology.

Table 5.3.1
Rainfall Frequency Multiplication Factors

Frequency	Multiplication Factor
2-yr	0.387
5-yr	0.584
10-yr	0.714
25-yr	0.878
50-yr	1.000
100-yr	1.122
500-yr	1.402

Appendix B contains isohyetal maps for the 50-year, 24-hour rainfall depth. The isohyetal contour lines are spaced at intervals of two-tenths of an inch. The spatial rainfall distributions for the county design storms were converted to grid data for use with Geographic Information System (GIS) compatible hydrologic models.

5.4 DESIGN STORM

The three components of the design storm include the IDF equation, the unit hyetograph curve, and the isohyets. These components are used to define the design storm for a particular location and frequency. As an example, consider the 25-year design storm for the Palmer Canyon watershed in Figure 5.4.1. Subarea 1A of this watershed, shown in Figure 5.4.2, will be used for the sample calculations.

1. Compute the area between successive isohyetal lines and multiply by the average of the isohyet values. Table 5.4.1 shows the areas between isohyets for Subarea 1A.

2. The sum of these precipitation-area values divided by the total subarea area provides the area weighted average rainfall depth. The average rainfall should be calculated to the nearest two-tenths of an inch. Table 5.4.1 contains the calculations for the isohyetal values in this subarea.

It may be noted that for small subareas, the isohyet nearest the centroid of the subarea usually equals the design depth. Selecting the isohyets nearest the subarea centroid is an acceptable method for determining the design rainfall for subareas of approximately 40 acres.

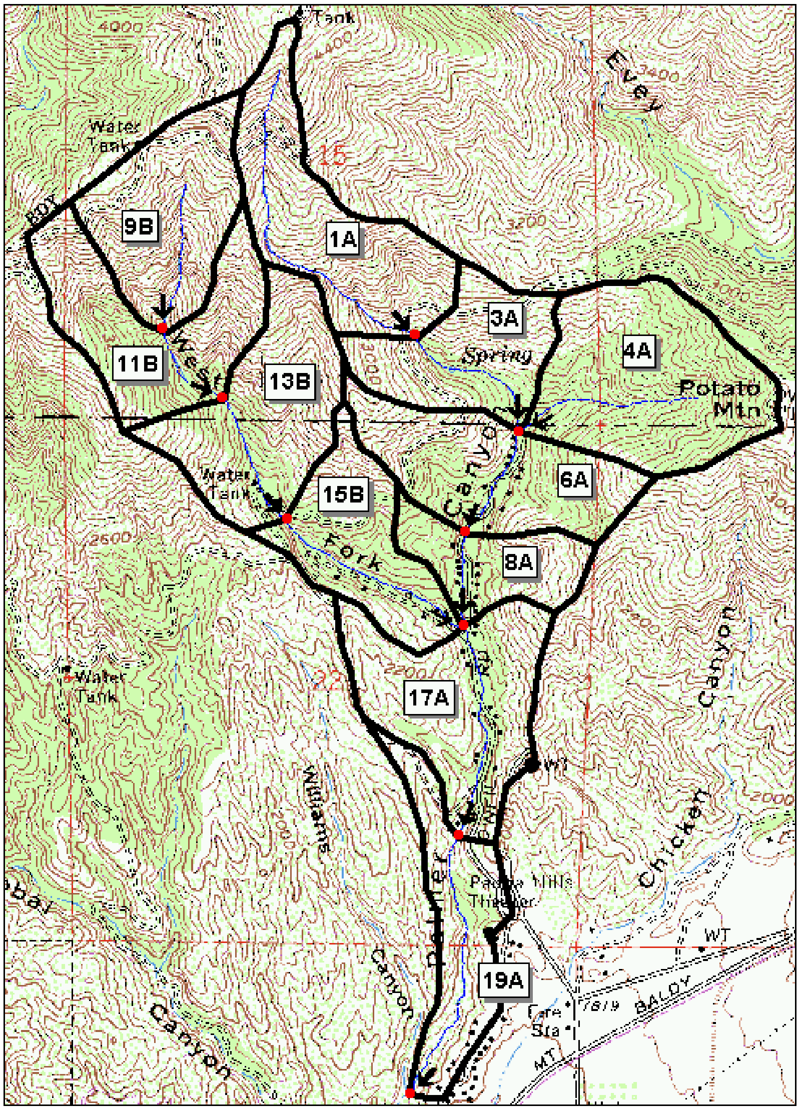

Figure 5.4.1
Palmer Canyon Watershed

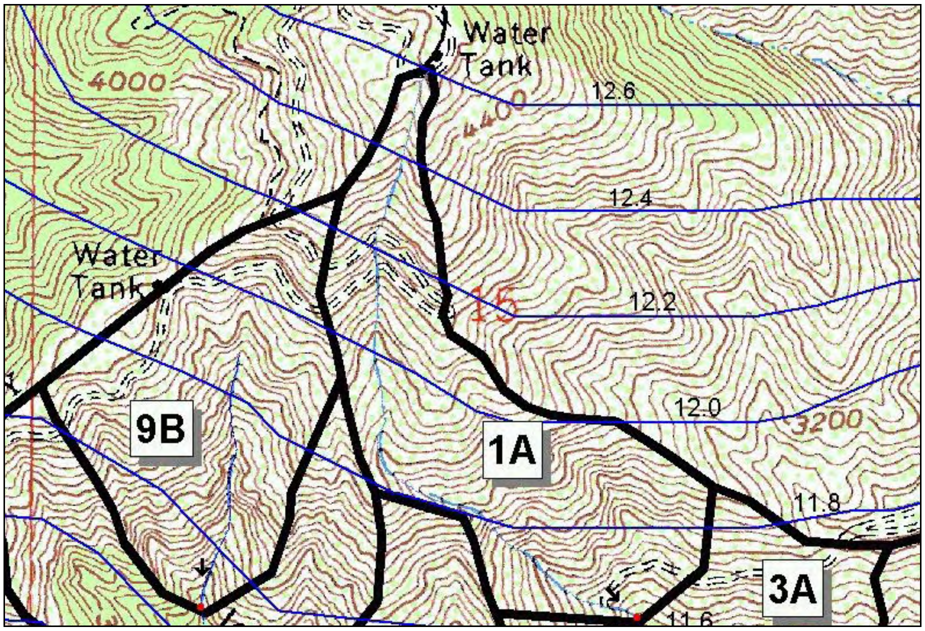

Figure 5.4.2
Subarea 1A with 50-Year, 24-Hour Rainfall Isohyets

Table 5.4.1
Subarea 1A Average Rainfall Depth Calculation

Subarea 1A	Isohyet (in)		Area between Isohyets (acres)		Average Depth (in)		Precipitation * Area (in-acres)
	12.6						
		⟶	2.6	*	12.5	=	32.5
	12.4						
			6.9		12.3		84.9
	12.2						
			13.4		12.1		162.1
	12.0						
			29.7		11.9		353.4
	11.8						
			15.1		11.7		176.7
	11.6						
Total			67.7				809.6
809.6 in-acre / 67.7 acre = 11.96 in → 12.00 in							

Table 5.4.2 shows average rainfall values calculated for the other subareas using the method from steps 1 and 2.

Table 5.4.2
Subarea Average Rainfall Depths

Subarea	Isohyetal Depth (in)
3A	11.4
4A	11.2
6A	11.0
8A	10.8
9B	11.4
11B	11.2
13B	11.0
15B	10.8
17A	10.2
19A	9.4

3. Using the rainfall frequency factor, the 50-year, 24-hour depths are scaled to match the required 25-year, 24-hour depths. The 25-year, 24-hour factor from Table 5.3.1 is 0.878.

Table 5.4.3
Scaling Rainfall Depths

Subarea	50-year depth (in)	50-year to 25-year factor	25-year depth (in)
1A	12.0	* 0.878 =	10.5
3A	11.4	* 0.878 =	10.0
4A	11.2	* 0.878 =	9.8
6A	11.0	* 0.878 =	9.7
8A	10.8	* 0.878 =	9.5
9B	11.4	* 0.878 =	10.0
11B	11.2	* 0.878 =	9.8
13B	11.0	* 0.878 =	9.7
15B	10.8	* 0.878 =	9.5
17A	10.2	* 0.878 =	9.0

4. Next, apply this 25-year, 24-hour depth to the unit hyetograph to produce the design storm hyetograph for the subarea. Multiply each depth on the

unit hyetograph by the 25-year, 24-hour rainfall depth. This produces a cumulative hyetograph for the fourth day. Calculate hyetographs for the first three days by multiplying the unit hyetograph by 10, 40, and 35 percent of the fourth day's rainfall depth. Figure 5.4.3 shows Subarea 1A's temporal rainfall distribution for each day of the design storm.

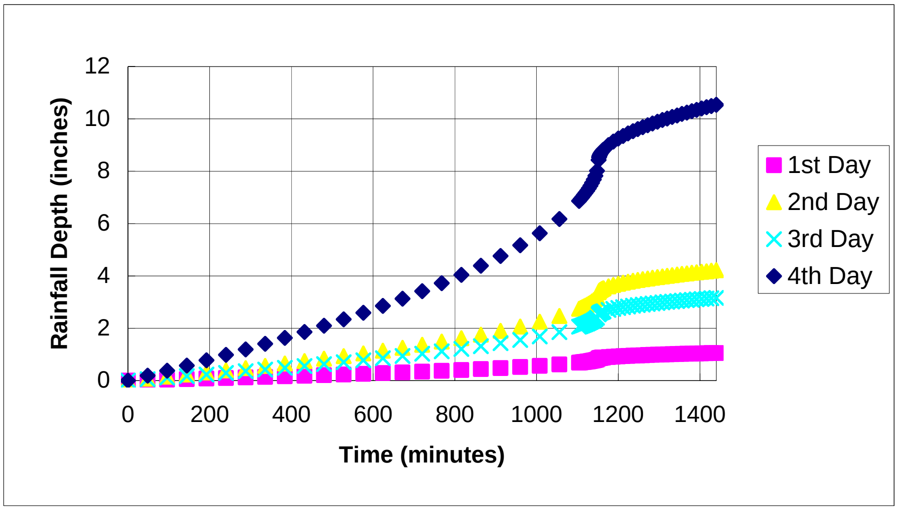

Figure 5.4.3
Hyetographs for Each Storm Day – Subarea 1A

Equation 5.1.2 determines the maximum intensity for the design storm assuming the time of concentration for Subarea 1A is 8 minutes.

$$\frac{I_t}{I_{1440}} = \left(\frac{1440}{t}\right)^{0.47}$$

(Equation 5.1.2)

Where: I_t = Rainfall intensity for the duration given in in/hr
t = 8 minutes
I_{1440} = 10.5 in / 24 hrs = 0.4375 in/hr

$$I_8 = \left(\frac{1440}{8\,\text{min}}\right)^{0.47} \times 0.4375 = 5.02 \ {}^{in}\!/_{hr}$$

The peak 8-minute intensity for the 25-year storm is 5.02 in/hr. If the time of concentration is 8 minutes, the peak flow will be Q = CIA, where I = 5.02 in/hr.

5.5 PROBABLE MAXIMUM PRECIPITATION (PMP)

As noted in Section 4.5, many dam spillways that fall under the State of California jurisdiction must safely pass runoff from the Probable Maximum Precipitation (PMP). The National Weather Service developed PMP design storms for use in the United States.

There are two types of PMP storms: the 3-day general-storm and the 6-hour local-storm. Facilities requiring protection from the Probable Maximum Flood must follow the PMP procedures to develop design storms. The National Weather Service's Hydrometeorological Reports No. 58 and 59 detail procedures for developing the design storm.[4,5] These reports are available at http://www.nws.noaa.gov/oh/hdsc/On-line_reports

Figure 5.5.1
Appian Way in Long Beach
January 21, 1969

1 Applied Hydrology. Chow, Maidment, and Mays. page 466, McGraw-Hill, New York, 1988.
2 Memorandum from Reza Izadi to Brian T. Sasaki, Re: Los Angeles County Hydrologic Method dated March 4, 2002.
3 Applied Hydrology. Chow, Maidment, and Mays. page 466, McGraw-Hill, New York, 1988.
4 Hydrometeorological Report No. 58, Probable Maximum Precipitation for California Calculation Procedures, National Weather Service. October 1998.
5 Hydrometeorological Report No. 59, Probable Maximum Precipitation for California, National Weather Service. February 1999.

CHAPTER

6

Rainfall-Runoff Relationships

Only a portion of the rain that falls on a watershed appears as surface runoff in a stream. This section of the manual describes two methods for estimating the portion of rainfall that becomes runoff. This portion is called the rainfall excess or effective rainfall.

6.1 RAINFALL LOSSES AND RUNOFF PRODUCTION

Rainfall becomes runoff when all loss processes are satisfied. Runoff results from rainfall not lost to infiltration, interception, depression storage, and evaporation.

"Infiltration is the process of water penetrating the ground surface into the soil."[1] Interception loss occurs when water is retained on vegetation and other surfaces. Intercepted water may evaporate or infiltrate. Loss due to depression storage occurs when water accumulates in depressions of all sizes that are not connected to a flow path. Evapotranspiration, a dominant force in the hydrologic cycle, proceeds slowly during a storm.

Different methods have been developed to model rainfall losses. These include runoff coefficients, constant loss parameters, the Horton method, exponential loss calculations, and Green-Ampt losses. The Modified Rational Method uses runoff coefficients. The following sections discuss infiltration and loss methods used within the County of Los Angeles.

6.2 INFILTRATION

Infiltration losses have the greatest effect on surface runoff. The rate of infiltration is a function of the state of the soil and is highly heterogeneous over space and time. Hydraulic conductivity is a measure of the ease with which water can travel through the soil and is a measure of the infiltration rate when the soil is saturated. Similar soils generally have similar hydraulic conductivities. However, the infiltration rate is also affected by the degree of soil saturation. Dry soil allows more infiltration than wet soil. Factors such as ground cover or recent fires within the watershed affect the soil surface and infiltration rates.

Public Works' hydrologic standards assume that watersheds subject to design rainfall are at a field capacity soil moisture condition. This condition is also referred to as a saturated condition. At field capacity, the forces due to gravity and the surface tension on a drop of water in the soil column are in balance. At this point, no water is draining from the soil. Adding water to the soil forces downward movement and allows infiltration to begin.

6.3 MODIFIED RATIONAL LOSS CALCULATIONS

The modified rational method (MODRAT) uses a runoff coefficient that is a function of the rainfall intensity. The runoff coefficient reflects the fraction of rainfall that does not infiltrate and is based on the rainfall intensity for a given time period.

The Modified Rational Method uses the following equation at each time step:

$$Q = C*I*A$$ **Equation 6.3.1**

Where: Q = Volumetric flow rate in cfs
C = Runoff coefficient, dimensionless
I = Rainfall intensity at a given point in time in in/hr
A = Watershed area in acres

The following sections describe development of the unburned soil runoff coefficient, C_u, the developed soil runoff coefficient, C_D, and the burned soil runoff coefficient, C_{ba}. The appropriate coefficient represents runoff for different watershed conditions.

Undeveloped Runoff Coefficient (C_u)

MODRAT uses runoff coefficient curves to model the runoff response of the soil to changing intensity. The 179 undeveloped runoff coefficient curves, plotted in Appendix C, correspond to different soil types within the County of Los Angeles. Figure 6.3.1 shows the shape of a typical runoff coefficient curve.

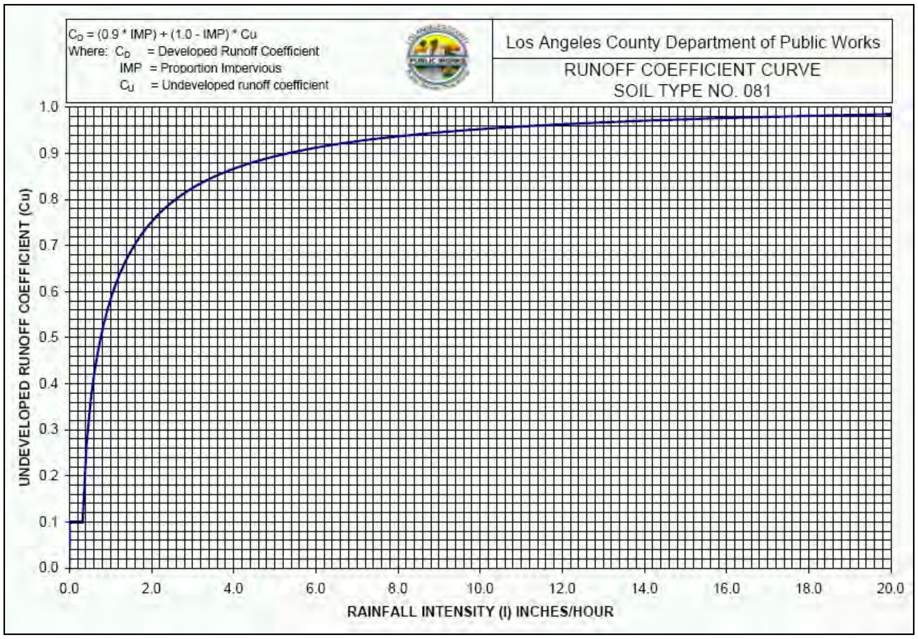

Figure 6.3.1
Runoff Coefficient Curve for Soil 081

Double ring infiltrometer tests provided data for the runoff coefficient curves. The infiltrometer tests used a department-designed, sprinkling–type infiltrometer. Before performing infiltrometer testing, the county was divided into regions of likely hydrologic homogeneity. Areas of homogenous runoff characteristics in the valley and desert areas were based on soil classifications published by the United States Department of Agriculture, Natural Resources Conservation Service. Criteria for homogeneity included topography, rock type, soil type, vegetative cover, and litter. Results from the infiltrometer tests within the homogenous areas determined the infiltration rate.

A series of runoff coefficient-intensity pairs compose each runoff coefficient curve. Each of the curves has a minimum coefficient (C_u) of 0.1 indicating that there is some runoff even at the smallest rainfall intensities. Appendix C contains the runoff coefficient curves for all the soils within the County of Los Angeles.

MODRAT requires assigning a single soil type for each subarea modeled. If a subarea contains more than one soil type, the predominant soil type in the subarea is used.

Developed Soil Runoff Coefficient Curves (C_D)

Each undeveloped runoff coefficient curve represents natural soil conditions. When precipitation occurs over a developed watershed, the rain falls on directly connected impervious areas and pervious areas. Runoff from pervious areas only occurs during heavy rainfall. Directly connected impervious area always produces direct runoff. As impervious area increases, the amount of direct runoff increases. The runoff coefficient curve must be modified to match the developed condition. Equation 6.3.2 accounts for the effects of development based on the undeveloped runoff coefficient and the amount of impervious area.

$$C_d = (0.9 * IMP) + (1 - IMP) * C_u$$

Equation 6.3.2

Where: C_d = Developed area runoff coefficient
IMP = Percent impervious
C_u = Undeveloped area runoff coefficient

The 0.9 in the equation represents the general assumption that no development is completely impervious. This assumption also accounts for initial abstraction losses in developed areas.

Imperviousness is assigned based on the land use types present in a subarea. Land use information requires existing and/or planned development patterns. If more than one type of development is present within a subarea, a composite impervious value must be determined using an area-weighted average. For example, consider a subarea with the characteristics in Table 6.3.1.

Table 6.3.1
Composite Impervious Values

	Percent Impervious	Area (acres)	Impervious*Area
	91%	20	1820
	42%	5	210
	21%	10	210
	1%	5	5
Total	-	40	2245

To determine the composite impervious value for this subarea, calculate the area weighted average of imperviousness. First, multiply each impervious value by the area it represents. Then sum these products and divide by the total area. The composite area weighted imperviousness for the example subarea is:

$$\text{Composite imperviousness} = {}^{2245}\!/_{40} = 56\%$$

The Southern California Association of Governments (SCAG) land use studies establish the land use patterns within the county. SCAG creates land use maps based on development type. Public Works assigns imperviousness values to each development type and then verifies these values using previous studies and aerial photos. The current land use map is based on SCAG data from 2000.

Representative proportion impervious values have been developed by measuring sample areas for each land use type. Appendix D has a table of these values. For undeveloped rural areas, 1% of the area is assumed impervious. Table 6.3.2 shows the standard range of percent impervious values for different development types.

Type of Development	Percent Impervious
Single-Family	21% to 45%
Multi-Family	40% to 80%
Commercial	48% to 92%
Industrial	60% to 92%
Institutional	70% to 90%

Table 6.3.2
Standard Range of Percent Impervious

Burned Soil Runoff Coefficient Curves (C_{ba})

Wildfires frequently burn undeveloped watersheds within the County of Los Angeles. Infiltrometer tests conducted in burned chaparral-covered mountain watersheds indicate that these watersheds suffer from a decreased infiltration rate after a fire. The decrease results from calcification caused by intense heat, plugging of the soil pores by ash or other fines, and other chemical reactions that produce a hydrophobic condition. A lack of surface cover also promotes the formation of a crust of fine soil due to the impact of raindrops. This crust further impedes infiltration.[2]

Collection of field infiltrometer data in recently burned areas quantified the infiltration rate decrease for all soil types. Tests were done in burned and unburned portions of an area with previously homogenous infiltration.

Figure 6.3.2 is a picture of the 2002 Williams Fire in the San Gabriel Mountains viewed from Santa Fe Dam.

Figure 6.3.2
Williams Fire in the San Gabriel Mountains Viewed From Santa Fe Dam 2002

Burned area runoff calculations use a runoff coefficient curve adjusted for the burned watershed condition. For burned watersheds, the rational equation becomes $Q_{ba} = C_{ba}IA$, in which Q_{ba} and C_{ba} are respectively the peak runoff from a burned area and the statistically adjusted burned soil runoff coefficient. The burned runoff coefficient is adjusted using a fire factor. The fire factor is an index between the natural and completely burned watershed conditions, which ranges from 0 to 1 respectively. An analysis of historic fires in the major watersheds within the County of Los Angeles provided design fire factors for undeveloped watersheds.[3,4] Table 6.3.3 contains the design fire factors.

Watershed	Fire Factor
Santa Clara River Watershed & Antelope Valley	0.34
Los Angeles River Watershed	0.71
San Gabriel River Watershed	0.71
Coastal Watershed	0.83

Table 6.3.3
Design Fire Factors for Use with Burned Watershed Hydrology

Only undeveloped subareas with 15% or less imperviousness require burn calculations. Equation 6.3.3 calculates the burned runoff coefficient.

$$C_{ba} = FF \times [(1-K) \times (1-C_u)] + C_u$$

Equation 6.3.3

Where: C_{ba} = Adjusted burned soil runoff coefficient, dimensionless
FF = Fire Factor, the effectively burned percentage of watershed area, dimensionless
K = Ratio of burned to unburned infiltration rates for I, 0.677 x $I^{-0.102}$, dimensionless
I = Rainfall intensity in in/hr
C_u = Undeveloped runoff coefficient, dimensionless

The K factor represents the ratio of burned to unburned infiltration rates. The ratio varies with the rainfall intensity. Equation 6.3.4 is useful for determining the burned peak flow when an unburned flow and intensity are known.

$$Q_{ba} = FF \times [(0.677 \times I^{-0.102} - 1) \times (Q_u - A \times I)] + Q_u$$

Equation 6.3.4

Where: Q_{ba} = Peak runoff from a burned area in cfs
FF = Fire Factor, the effectively burned percentage of watershed area
I = Rainfall intensity in in/hr
A = Watershed area in acres
Q_u = Peak runoff from an unburned area in cfs

Fires increase runoff and debris production. Higher runoff rates entrain more debris and burned watersheds have more debris available for entrainment. Debris production yields as much as 120,000 cubic yards/square mile of watershed for major storms. Boulders up to eight feet in diameter have been deposited in valley areas at considerable distances from their source. Debris quantities equal in volume to the storm runoff (100 percent bulking) have been recorded in major storms. The Flood Control District and the

Department of Public Works have built many debris control and storage structures in the foothills to minimize the chance of channels clogging with debris.

Peak flows from burned watersheds are “bulked” to account for volume changes caused by debris entrainment. Debris basins remove the sediment so that downstream flows are equal to flows from burned watershed. For more information on debris production, bulking flows, sediment transport, and design of debris retaining structures and basins, see the Department of Public Works Sedimentation Manual.

6.4 CONSTANT LOSS METHOD

The constant loss method is a frequently used and generally accepted rainfall loss method for flood hydrology. The constant loss method models infiltration by allowing all rainfall to infiltrate when the rainfall intensity is below a certain rate. All rainfall exceeding this infiltration rate will run off. Table 6.4.1 contains example calculations of direct runoff using the constant loss method. A constant loss rate of 0.1 in/hr is applied to an incremental rainfall series. Rainfall exceeding the loss rate becomes runoff.

Table 6.4.1
Application of Constant Loss Method

Time (hours)	Incremental Rainfall (in)	Loss (CL=0.10 in/hr)	Runoff (in)
1	0.00	0.00	0.00
2	0.05	0.05	0.00
3	0.08	0.08	0.00
4	0.10	0.10	0.00
5	0.20	0.10	0.10
6	0.12	0.10	0.02
7	0.05	0.05	0.00

Figure 6.4.1 illustrates the relationship between the constant loss rate and the total rainfall. In this example, a total of 0.60 inches of rain fell in 7 hours. Of this rain, a total of 0.48 inches was lost to infiltration while 0.12 inches became runoff. The runoff coefficient for this entire period is 0.2, representing that 20 percent of rainfall becomes runoff.

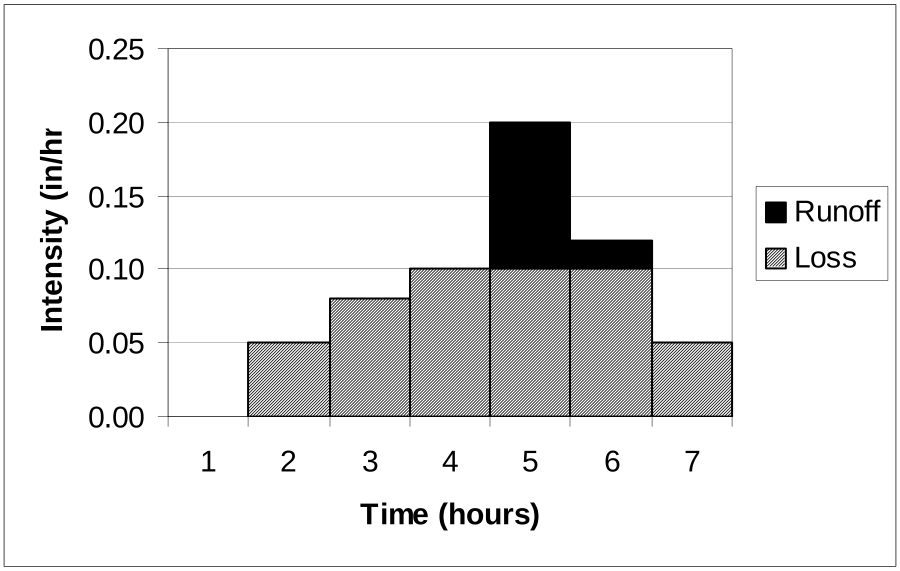

Figure 6.4.1
Rainfall Hyetograph and resulting Constant Loss Runoff

In general, application of a constant loss rate requires model calibration to estimate the loss rate parameters. Constant loss rates are highly variable and depend on the degree of saturation, soil type, storm duration, and rainfall intensity.

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1 *Applied Hydrology*. Chow, Ven Te; David R. Maidment; and Larry W. Mays. page 188. McGraw-Hill, Inc. New York, 1988.

2 *Handbook of Hydrology*. Ed. Maidment, David R. page 5.42. McGraw-Hill. New York, 1993.

3 “Development of Burn Policy Fire Factors.” Los Angeles County Department of Public Works. August 5, 2004.

4 “Development of Burn Policy Methodology (Santa Clara River Pilot Project).” Los Angeles County Department of Public Works. June 2003.

CHAPTER

7

Runoff Calculation Methods

The design of drainage systems for stormwater conveyance within the County of Los Angeles requires converting rainfall into runoff volumes and flow rates. There are many methods available for converting the rainfall to runoff.

The Department of Public Works uses two basic methods for converting rainfall to runoff, depending on the conditions. The methods are facilitated by software for use on a personal computer. The sections in this chapter explain how to select the proper method for hydrologic studies and the theory and application of the two methods.

7.1 SELECTING THE PROPER METHOD

Table 7.1.1 provides a brief description of the uses and limitations of each method.

Table 7.1.1
County of Los Angeles Hydrologic Methods

Method		**Use / Limitations**
Rational Method	Use:	For drainage areas 40 acres or less; finds the peak flow rate for any frequency design storm
	Limitations:	Does not create hydrographs or determine runoff volumes. Area limited to approximately 40 acres.
Modified Rational (MODRAT)	Use:	For any size watershed; for any combination of laterals; for any combination of developed and undeveloped drainage areas; to create hydrographs and runoff volumes at specified locations; to find peak subarea and mainline flow rates; recommended method for systems incorporating pumping or water impoundment.
	Limitations:	Underestimates volumes in rural areas when interflow and baseflow add to the runoff volume.

7.2 RATIONAL METHOD

Mulvaney first outlined the rational method[1], which assumes that a steady, uniform rainfall rate will produce maximum runoff when all parts of the watershed are contributing to outflow[2]. This occurs when the storm event lasts longer than the time of concentration. The time of concentration is the time it takes for rain in the most hydrologically remote part of the watershed to reach the outlet. The method assumes that the runoff coefficient remains constant during a storm. The rational method formula is Q = CIA, previously mentioned in Chapter 6 as Equation 6.3.1. The direct runoff volume is calculated using the following equation:

$$V = C * \left(\frac{P}{12}\right) * A$$

Equation 7.2.1

Where:
- V = Volume in ac-ft
- C = Runoff coefficient, proportion of rainfall that runs off the surface
- P = Rainfall depth in inches
- A = Drainage area in acres

Use of the rational method for drainage system design in small urban areas is appropriate. Use within the County of Los Angeles requires subarea division when[3]:

- Subareas are larger than approximately 40 acres
- There is more than one drainage channel
- Hydrologic properties are different within the area
- The time of concentration is greater than 30 minutes

The following are disadvantages of the classic rational method:

- Does not produce a hydrograph
- Runoff coefficient, C, is usually the same regardless of rainfall intensity
- Results are unreliable for areas greater than 200 acres[8]

The rational method applies to small watersheds where storage routing is not necessary. The method is useful for determining peak flows from small subdivisions and development projects or to determine flows to catch basins.

Section 7.5 describes catch basin hydrology in detail. Section 12.2 contains an example using the rational method to compute runoff.

7.3 MODIFIED RATIONAL METHOD

The modified rational method (MODRAT) uses a design storm and a time of concentration to calculate runoff at different times throughout the storm. Section 5.2 describes the temporal distribution of the design storm. Section 5.3 describes the spatial distribution of design storm rainfall within the County of Los Angeles.

Calculating flows based on the rainfall distribution results in a runoff hydrograph. The volume of runoff equals the area under the hydrograph curve. MODRAT allows users to route hydrographs generated in each subarea through conveyances and combine hydrographs based on time. MODRAT produces peak flows equal to or lower than flows calculated using the rational method. The reduction in peak results from attenuation, channel storage, and combining flows that peak at different times. Figure 7.3.1 shows an example of channel flow and storage.

Figure 7.3.1
Water storage occurring in Bradbury Channel May 28, 1981

Time of Concentration

The time of concentration (T_C) is the time it takes for rain in the most hydrologically remote part of the watershed to reach the outlet. Using a rainfall duration equal to the T_C ensures that the runoff from the entire subarea is contributing flow at the outlet. MODRAT requires a time of concentration in order to calculate intensities for use with the rational equation.

There are several methods for calculating the T_C. Simple relationships use the length of flow multiplied by an assumed flow velocity based on the type of conveyance (overland flow, sheet flow, pipe flow, etc.) Other methods include empirical equations derived through research and the use of the kinematic wave theory. The T_C calculation method for hydrology studies within the County of Los Angeles relies on a regression equation derived from hundreds of studies using the kinematic wave theory.

Time of Concentration - Kinematic Wave Theory[4]

The kinematic wave theory is a method accepted by Public Works, to calculate the time of concentration, T_C. Use of the kinematic wave theory to calculate the T_C requires separating the longest flow path into two parts: overland flow and conveyance flow. Equation 7.3.1 demonstrates this:

$$T_C = t_o + t_c$$ **Equation 7.3.1**

Where: T_C = Time of concentration in minutes
t_o = Overland flow travel time in minutes
t_c = Sum of all conveyance travel times in minutes

Conservation of mass and the momentum equation are used to determine the time associated with overland flow. Equations 7.3.2 and 7.3.3 are used to calculate overland flow time, t_o:

$$t_o = \frac{0.94 * L_o^{0.6} * n_o^{0.6}}{I_x^{0.4} * S_o^{0.3}}$$

Equation 7.3.2

$$I_x = C * I$$

Equation 7.3.3

Where:
- t_o = Overland flow travel time in minutes
- L_o = Overland flow length in feet
- n_o = Roughness for overland flow surface, dimensionless
- I_x = Rainfall excess in in/hr
- S_o = Slope of overland flow in ft/ft
- C = Runoff coefficient, ratio of runoff rate to rainfall intensity in in/in
- I = Rainfall intensity in in/hr

Values for the roughness coefficient of overland flow surfaces are found in Table 7.3.1.

Table 7.3.1
Roughness Coefficients for Overland Flow Computation

Surface Cover[5]	**n_o**
Smooth Asphalt	0.012
Concrete Paving	0.014
Packed Clay	0.030
Light Turf	0.250
Dense Turf	0.350
Industrial/Commercial	0.014
Residential	0.040
Rural	0.060

Table 7.3.2 shows standard values for different types of lots. The kinematic wave method requires evaluation of each subarea to determine the overland flow length and slope.

Table 7.3.2
Standard Values for Overland Flow Computation

Surface Cover[6]	Lot Length (ft)	Range of Lot Slope
Industrial/Commercial	200	0.005 - 0.020
Residential	100	0.010 - 0.050
Rural	200	0.050 - 1.000

Figure 7.3.2 illustrates the different types of lots where overland flow occurs.

INDUSTRIAL/ COMMERCIAL **RESIDENTIAL** **RURAL**

Figure 7.3.2
Different Types of Lots Where Overland Flow Occurs

The kinematic wave approach is applicable to channel flow as well as overland flow. The Manning equation is a form of kinematic wave theory for channels. The Manning equation is used to determine the average velocity in the channel. This velocity is used to determine travel times as shown in equation 7.3.4:

$$t_c = \left(\frac{1}{60}\right)\left(\frac{L_c}{V_{ave}}\right)$$

Equation 7.3.4

Where:
- t_c = Conveyance flow travel time in minutes
- L_c = Conveyance flow length in feet
- V_{ave} = Average conveyance velocity based on Manning equation in ft/sec

Comparison of results from Equation 7.3.1 with Izzard's overland flow experimental results and the results of Yu and McNown showed good correlation[6].

Use of the equations in this section requires an iterative approach since the rainfall excess and T_C are related to each other. An example problem illustrates application of the kinematic wave method for calculating T_C. Figure 7.3.3 shows the subarea that will be analyzed to determine the T_C using the kinematic wave method.

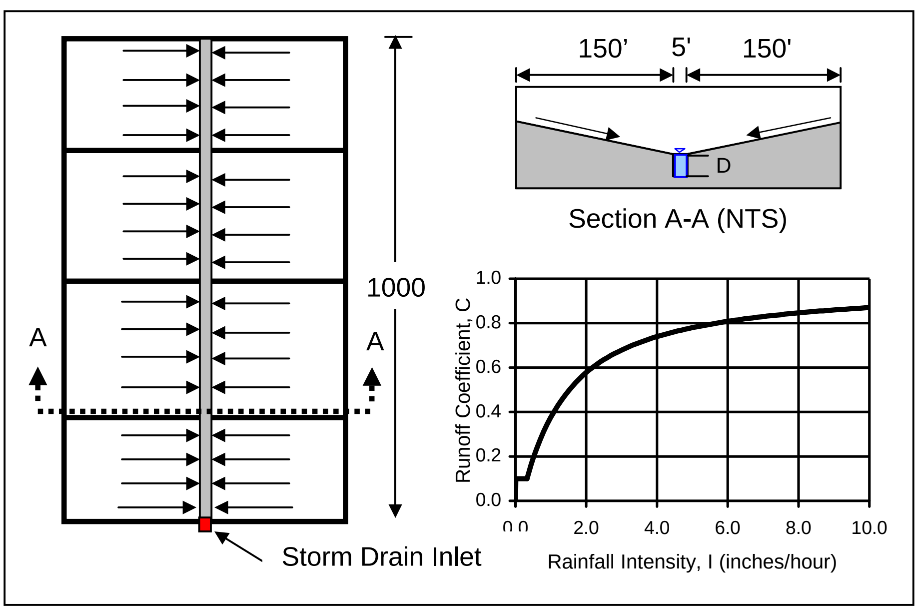

Figure 7.3.3
Example Subarea Demonstrating Kinematic Wave Method

This example shows eight residential lots that drain to a small grassy channel that eventually flows into a storm drain. Table 7.3.3 provides the lot and channel characteristics. The 50-year 24-hour rainfall for this area is 5 inches.

Table 7.3.3
Kinematic Wave Conveyance Data

Flow Path	Length (ft)	Slope (ft/ft)	Manning n	Width (ft)	Max. Depth (ft)
Overland Flow - Lot	150	0.020	0.040	-	-
Concrete Channel	1000	0.005	0.013	5	1

The steps involved in calculating a time of concentration using the kinematic wave method and example calculations are provided:

1. Assume an initial time of concentration

Assume a T_C of 12 minutes for the subarea in Figure 7.3.3

2. Calculate the intensity using Equation 5.1.2 and runoff coefficient using Equation 6.3.2 for overland flow using the time of concentration as the duration

$$I_t = I_{1440} * \left(\frac{1440}{t}\right)^{0.47} \Rightarrow I_{12} = \frac{5\text{ in}}{24\text{ hr}} * \left(\frac{1440}{12\text{ min}}\right)^{0.47} = 1.98 \text{ in/hr}$$

With the 2.0 in/hr intensity, the runoff coefficient is determined from the runoff coefficient curve in Figure 7.3.3. The undeveloped runoff coefficient is 0.58. Assuming a percent impervious of 0.42 for residential land use, the developed runoff coefficient is:

$$\begin{aligned} C_d &= (0.9 * IMP) + (1.0 - IMP) * C_u \\ &= (0.9 * 0.42) + (1.0 - 0.42) * 0.58 = 0.71 \end{aligned}$$

3. Calculate the time required for overland flow to reach the channel using Equation 7.3.2

$$t_o = \frac{0.94 * L_o^{0.6} n_o^{0.6}}{i_x^{0.4} S_o^{0.3}} = \frac{0.94 * (150)^{0.6}(0.040)^{0.6}}{(1.98 * 0.71)^{0.4}(0.020)^{0.3}} = 7.78 \text{ minutes}$$

4. Calculate the average flow in the channel using the rational method

$$\frac{Q}{2} = \frac{C*I*A}{2} = \frac{0.71}{2} * 1.98\,\frac{in}{hr} * \left(\frac{1000\text{ ft} * 305\text{ ft}}{43560\text{ ft}^2/\text{ac}}\right) = 4.92\text{ cfs}$$

5. Determine the velocity for the average channel flow

Solving Manning's Equation for V = 3.39 ft/s

6. Calculate the conveyance flow travel time using Equation 7.3.4

$$t_c = \left(\frac{1}{60}\right)\left(\frac{L_c}{V_{ave}}\right) = \left(\frac{1}{60}\right)\left(\frac{1000}{3.39}\right) = 4.92 \text{ minutes}$$

7. Add the overland flow time and the conveyance flow time to determine the time of concentration using Equation 7.3.1

$$T_C = t_o + t_c = 7.78 + 4.92 = 12.7 \text{ minutes}$$

8. If the value is within 0.5 minutes of the original estimate, use the estimate. If the value is not within 0.5 minutes, round the value from step 7 to the nearest minute and use the value as the new estimate to start the calculations again.

Round the value to 13 minutes and start at step 2. The second iteration provided the values used to find the final T_C:

I	= 1.90 in/hr
t_o	= 7.94 minutes
Q_{ave}	= 4.66 cfs
V_{ave}	= 3.33 ft/s
t_c	= 5.00 minutes
T_C	= 7.94 + 5.00 = 12.94 minutes

Public Works developed a computer program to calculate T_C for hydrologic study subareas. Public Works used the computer program from 1986 until 2001.

Time of Concentration - Regression Equation[7]

Determining the overland flow length and roughness was time consuming and determining the T_C for the conveyance often required solving the Manning equation many times. A 1999 study resulted in the creation of a regression equation for T_C calculations. The regression equation relied on T_C computations from a large number of subareas. The subareas were taken from diverse hydrology studies that used the kinematic wave theory equations to calculate T_C. This representative sample of subarea T_C's came from hydrologic studies performed between 1986 and 1999.

Equation 7.3.5 correlates the T_C to independent hydrologic parameters: flow path length and slope, land use, rainfall intensity, and the soil runoff coefficient. Equation 5.1.2 from Chapter 5 provides the relationship between the 24-hour intensity and the intensity related to the T_C. Equation 6.3.2 from Chapter 6 provides a relationship between the developed and undeveloped soil runoff coefficients.

$$T_C = \frac{0.31 * L^{0.483}}{(C_d * I_t)^{0.519} * S^{0.135}}$$ **Equation 7.3.5**

$$I_t = I_{1440} * \left(\frac{1440}{t}\right)^{0.47}$$ **(Equation 5.1.2)**

$$C_d = (0.9 * IMP) + (1.0 - IMP) * C_u$$ **(Equation 6.3.2)**

Where:
- T_C = Time of concentration in minutes
- L = Longest flow path length from watershed boundary to outlet in feet
- C_d = Developed runoff coefficient, ratio of runoff rate to rainfall intensity in in/in
- I_t = Intensity at time t in in/hr
- S = Slope of longest flow path in ft/ft
- IMP = Percent Impervious, percent expressed as 0.0 to 1.0
- C_u = Undeveloped runoff coefficient, ratio of runoff rate to rainfall intensity in in/in

The regression method still uses an iterative process to calculate the time of concentration. See Section 11.1 for sample time of concentration calculations using the regression equation.

Reviewing the example in Section 11.1 shows that the regression equation calculation is approximately one minute longer than the kinematic wave method calculation for the same example. This difference is explained by the fact that many studies and calculations were used to create the regression equation. The regression equation provides the best fit for all of the studies, but may not match kinematic wave calculations exactly.

Chapter 10 describes the data necessary for watershed modeling and calculation of the time of concentration. Spreadsheet applications and computer programs listed in Chapter 11 automate the iterative process.

Hydrograph Generation

MODRAT relies on the dimensionless temporal rainfall distribution, an isohyetal depth, and the T_C to generate hydrographs. The steps for calculating the runoff are:

1. Determine the rainfall intensity for a time period equal to the T_C
2. Determine the undeveloped soil runoff coefficient for the time period using the intensity
3. Adjust the soil runoff coefficient using Equation 6.3.2 or 6.3.3 to determine C_d or C_{ba}, depending on the subarea conditions
4. Use the rational equation, Equation 7.2.1, to determine the runoff for the time period
5. Repeat steps 1 through 4 for each time period

Figures 7.3.4, 7.3.5, and Table 7.3.4 illustrate how to determine three flow rates based on the design storm for a specific subarea. The following subarea information is needed:

Area:	40 acres
T_C:	30 minutes
Soil:	068
IMP:	20%
Rain:	10 inches

Figure 7.3.4 shows the steepest portion of the rainfall mass curve related to the 50-year 24-hour rainfall depth of 10 inches. The three time segments

represent the intensity at the end of each time period. Figure 7.3.5 shows the soil runoff coefficients for soil 068. Table 7.3.4 shows the intensity, undeveloped runoff coefficient, developed runoff coefficient, the area, and the runoff for each time period. Three time periods are shown to demonstrate the changes in intensity that occur around the inflection point on the mass curve.

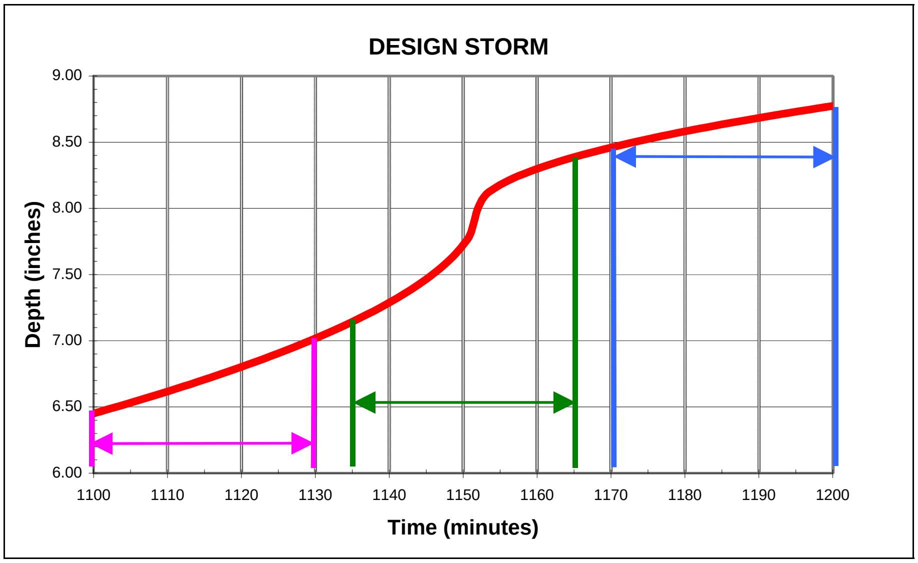

Figure 7.3.4
Three Time Steps for Modified Rational Runoff Calculations

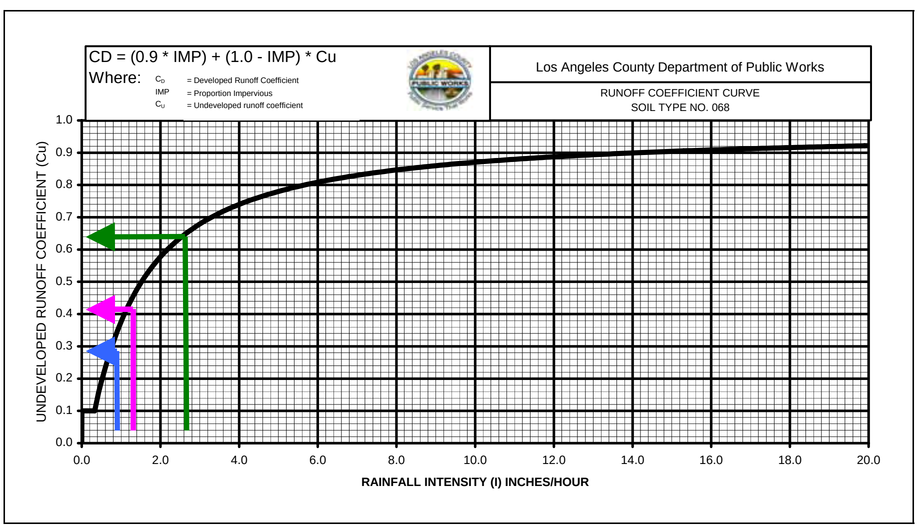

Figure 7.3.5
Undeveloped Runoff Coefficients for 3 Time Steps

Using Figures 7.3.4, 7.3.5 and Equation 6.3.2, Table 7.3.4 shows the runoff calculations for three time steps.

Time (minutes) To	Time (minutes) From	Rainfall (in)	Intensity, I (in/hr)	Undeveloped Runoff Coefficient, C_u Fig. 7.3.3	Developed Runoff Coefficient, C_d Eq. 6.3.2	Area (acres)	Q = C_d*I*A (cfs)
1100	1130	0.567	1.134	0.39	0.492	40	22.3
1135	1165	1.243	2.487	0.62	0.676	40	69.6
1170	1200	0.314	0.627	0.26	0.388	40	9.7

Table 7.3.4
Table of Runoff Calculations

Using the rainfall mass curve, the rainfall depth, and the time of concentration, the runoff value can be calculated for each one-minute increment. This is done by moving the time window forward one step and completing the process shown above. Computer programs or spreadsheets automate this time consuming process. Calculating the runoff at different time increments allows the user to create a hydrograph. Figure 7.3.6 shows the hydrograph for the three points calculated in Table 7.3.4. The figure assumes that at t = 0 and t = 1440 minutes, the flow rate is zero.

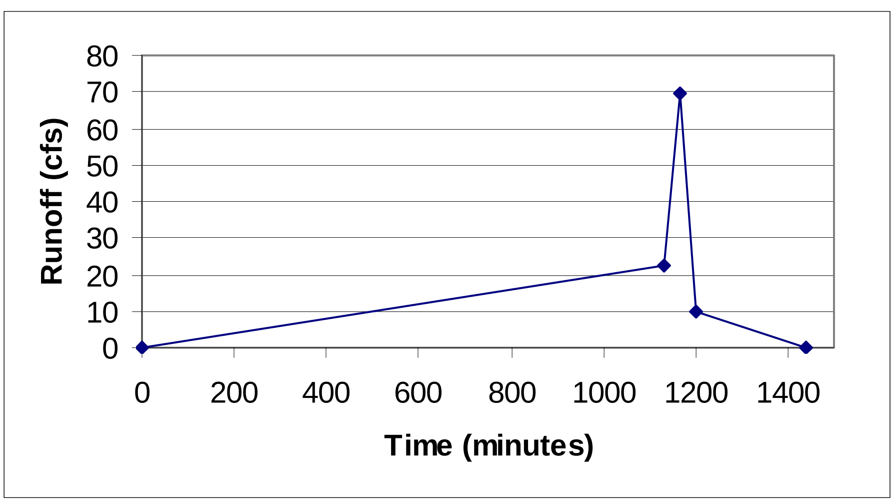

Figure 7.3.6
Hydrograph Generate Using MODRAT Method

The volume of runoff is calculated by summing up the area under the curve. For example, the volume for the first 1130 minutes is equal to the area under the curve. Finding the area of this triangle:

$$\text{Vol} = \frac{1}{2} * \text{b} * \text{h} = \frac{1}{2} * 1130 \text{ minutes} * 22.3 \frac{\text{ft}^3}{\text{sec}} * 60 \frac{\text{sec}}{\text{min}} = 755{,}970 \text{ ft}^3 = 17.35 \text{ ac - ft}$$

Defining the hydrograph with smaller time steps increases the accuracy of the flow rate and volume calculations. Hydrograph routing shows the affects of attenuation and allows superposition of hydrographs. This provides a more realistic evaluation of runoff than adding the peak flow rates calculated using the rational equation.

Channel Routing of Flows

Two types of channel routing exist: hydrologic and hydraulic. Hydrology studies within the County of Los Angeles use hydrologic routing to approximate unsteady flow through channels. Hydrologic routing balances inflow, outflow, and storage volume using the continuity equation. Routing the hydrographs results in outflow hydrographs that are smaller due to peak attenuation and occur later than the inflow due to flood wave translation.

Peak flow attenuation occurs when flows are stored in a channel reach. Figure 7.3.7 shows a graphical representation of peak attenuation. The volume of water stored increases as water fills the channel. Storage continues until the channel depth reaches the maximum water surface elevation. Storage then decreases as the peak flow passes and the water stored in the channel drains.

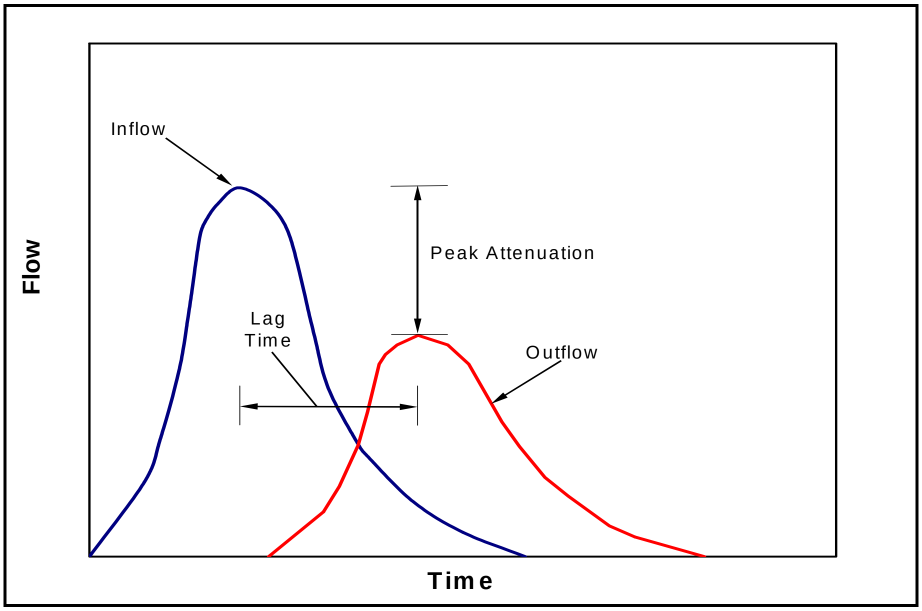

Figure 7.3.7
Peak Attenuation Related to Channel Storage

The water entering the channel must also travel from the upstream end of the section to the downstream end. Hydrologic routing considers this process by shifting the hydrograph in time. The shifting is related to the wave velocity for the specific channel.

There are many methods available for hydrologic routing[8]. The MODRAT method uses the Modified Puls, or level pool, routing method to determine channel storage effects. The method relies on a finite difference approximation of the continuity equation and an empirical representation of the momentum equation. Equation 7.3.8 is the basic equation for the Modified Puls method. The equation allows calculation of the outflow for each time step except the first. Chapter 8 shows another way to write the equation for the Modified Puls method that removes the need to calculate the storage for each time step.

$$\frac{1}{2}\left(I_i + I_{i-1}\right) - \frac{\left(S_i - S_{i-1}\right)}{t_i - t_{i-1}} = \frac{1}{2}\left(O_{i-1} + O_i\right)$$

Equation 7.3.8

Where:
I_{i-1} = Inflow at t_{i-1}
I_i = Inflow at t_i
t_i = Time at step i
t_{i-1} = Time at step i-1
S_{i-1} = Storage at t_{i-1}
S_i = Storage at t_i
O_{i-1} = Outflow at t_{i-1}
O_i = Outflow at t_i

The method ignores wedge storage within the channel reach and assumes that lateral inflow effects are insignificant. A storage-discharge relationship is also required between the inflow rate and storage in the system[9]. The method requires a defined channel storage versus inflow relationship. The relationship is established using the Manning equation to determine depth of flow. Multiplying channel length, water depth, and cross sectional area provides the channel storage for a specific flow value. Using different flow values produces a storage curve. Figure 7.3.8 presents the channel storage relationship for a triangular channel with the following characteristics: slope = 0.001 ft/ft, length = 1000 ft, Manning n = 0.03, side slope = 1:1 ft:ft, and max depth = 6.8 ft.

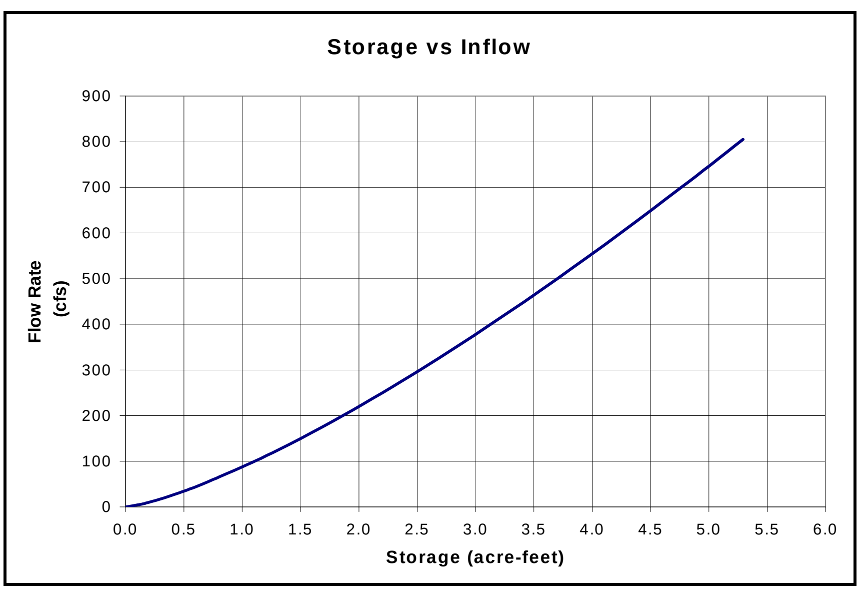

Figure 7.3.8
Storage-Inflow Relationship for a Triangular Channel

Calculation of translation time, the time it takes for the flood wave to travel from one end of the reach to another, requires using wave velocities. Table 7.3.5, Figure 7.3.9, and Figure 7.3.10 located at the end of the section provide more detail on velocity equations used for translation. Table 7.3.5 contains the equations used for translation time calculations. Figure 7.3.9 shows a typical street cross section. Figure 7.3.10 contains information for determining effective slopes of mountain and valley channels. The figure relates map slopes to slopes that match measured flow rates more accurately. The end of the section also contains a list of variables for the equations.

Correct hydrologic routing allows superposition of hydrographs at different locations within the study area. MODRAT starts at the upstream end of the watershed and calculates a runoff hydrograph. The hydrograph is then translated through the downstream channel. The Modified Puls routing then occurs to determine the effects of channel storage and the modified outflow hydrograph is computed. This hydrograph is then combined with the hydrographs from other subareas or is routed through another channel reach.

Computer programs implement this approach to reduce the amount of work required to define these relationships and route flows through the channels. Chapter 8 contains a detailed example of the Modified Puls routing method.

Table 7.3.5
Hydrograph Translation Equations

$T = \frac{L}{60 * V_W}$	Travel Time (minutes)
$V = \frac{Q}{A}$	Average Channel Velocity (ft/s)
$V = 5.6 * Q^{0.333} S_{eff}^{0.500}$	Velocity for Natural Mountain Channels (ft/s)
$V = (7.0 + 8.0 * Q^{0.352}) S_{eff}^{0.500}$	Velocity for Natural Valley Channels (ft/s)
$V_W = 1.5 * V$	Wave Velocity for Natural Mountain and Valley Channels (ft/s)
$D = \frac{B}{2 * \left(\left(Z^2 + 1 \right)^{0.500} - Z \right)}$	Most Efficient Rectangular or Trapezoidal Open Channel Section
$V = \frac{1.486}{n} * R^{0.667} S^{0.500}$	Pipe, Streets, Rectangular, or Trapezoidal Channels (ft/s)
$V_W = V * \left[\frac{\theta * (3 - 5\cos\theta) + \sin\theta}{3 * \theta(1 - \cos\theta)} \right]$	Wave Velocity for Partially Full Pipes (ft/s)
$V_W = V * \left[\frac{5}{3} - \frac{4 * (B + ZD)}{3 * (2 + B) * (B + 2ZD)} \right]$	Wave Velocity for Rectangular and Trapezoidal Channels (ft/s)
$\theta = 4 * \sin^{-1} \left(\frac{D}{d} \right)^{0.500}$	Angle Measurement to Determine Flow Depths in Pipes
$R = \frac{A}{P}$	Hydraulic Radius (ft)
$n = \frac{n_1 B + 2 * n_2 L_w}{B + 2 * L_w}$	Composite Manning's n for Trapezoidal Channels

Variables:	A	= Cross Sectional Area in ft^2
	B	= Channel Bottom Width in feet
	C	= Curb Height in feet
	D	= Flow Depth in feet
	d	= Pipe Diameter in feet
	L	= Length of Channel Reach in feet
	L_w	= Length of Wetted Channel Wall in feet
	n	= Channel Roughness Coefficient
	n_1	= Length of Wetted Channel Wall in feet
	n_2	= Length of Wetted Channel Wall in feet
	P	= Wetted Perimeter in feet
	Q	= Flow Rate in cfs
	R	= Hydraulic Radius in feet
	S	= Slope of channel reach (ft/ft)
	S_{eff}	= Effective channel slope, natural valley and mountain conveyances
	T	= Travel Time in minutes
	V	= Mean Velocity in ft/sec
	V_w	= Wave Velocity in ft/sec
	W_R	= Road Width From Curb to Curb in feet
	Z	= Channel Side Slope Computed as Horizontal Projection of Wall Divided by Depth in ft/ft

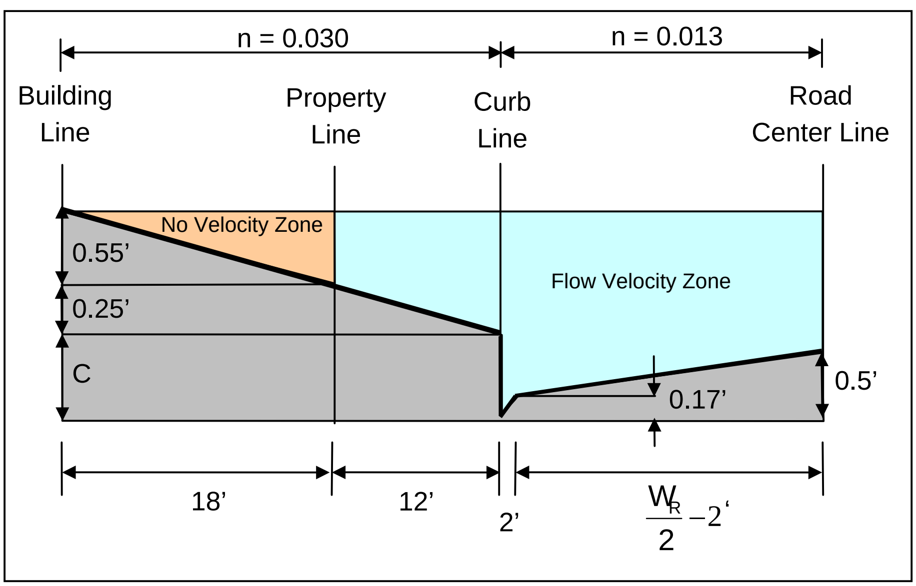

Figure 7.3.9
Typical Street Cross Section

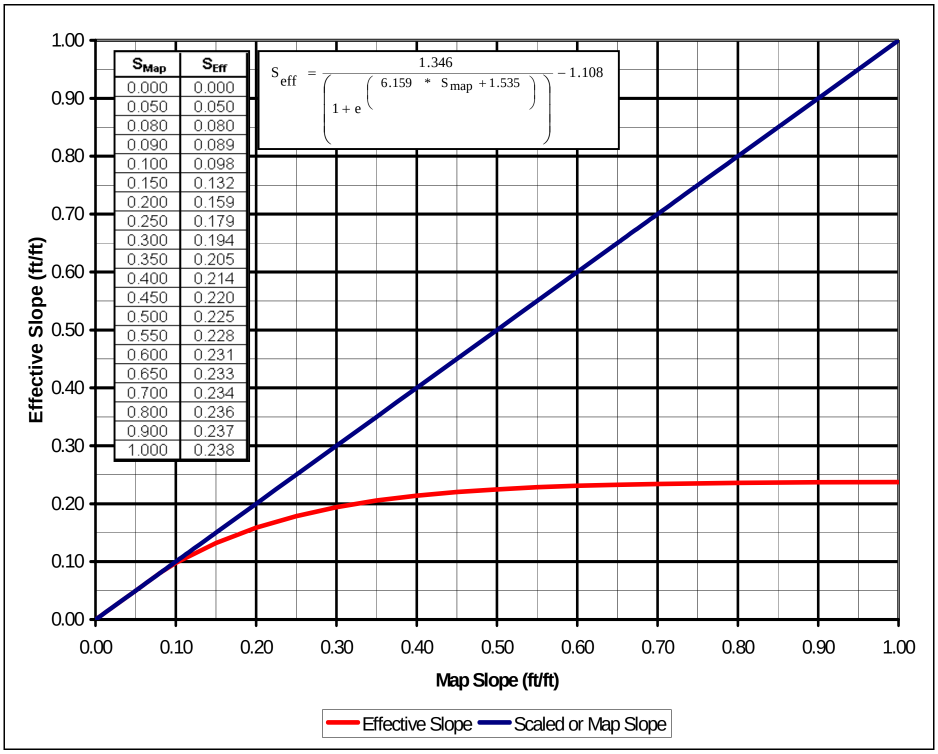

Figure 7.3.10
Effective Slope to Map Slope Relationship

7.4 CATCH BASIN FLOW CALCULATIONS

Flows that drain to catch basins usually come from areas smaller than the 40-acre subareas recommended in the hydrology manual. Determining flow to the catch basins is done by apportioning flow rates from the subarea based on the area draining to individual catch basins. Figure 7.4.1 shows a residential subarea of 35.5 acres that contains nine catch basins.

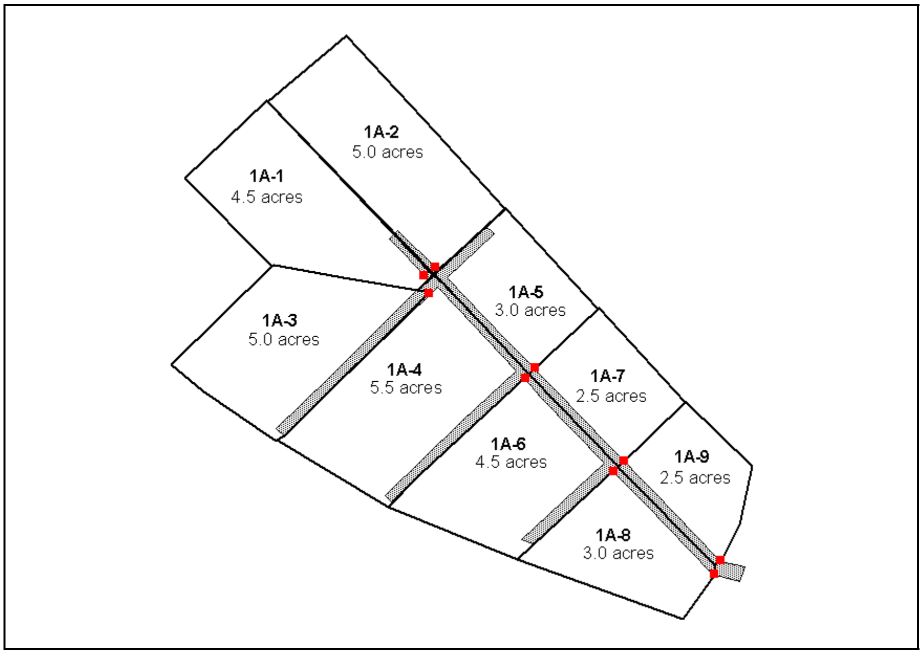

Figure 7.4.1
Catch Basin Flow Allotment

Catch basin allotment relates the peak subarea flow calculated using the MODRAT method to the subareas contributing flow. The steps for determining catch basin flow rates are:

1. Determine the area contributing flow to each proposed catch basin
2. Sum up the subarea areas to determine the total area
3. Divide each catch basin drainage area by the total area to get a weighting factor
4. Multiply the weighting factor by the MODRAT subarea watershed peak flow to get the catch basin peak flow rate for each basin

Table 7.4.1 contains the peak flow calculation for each catch basin in Figure 7.4.1. The total area for the MODRAT subarea 1A is 35.5 acres with a peak flow of 100 cfs.

Catch Basin Drainage Name	Area (A_i) (acres)	Weighting Factor (A_i/A_T)	Subarea Peak Flow (cfs)	Catch Basin Flows (cfs)
1A-1	4.5	0.13	100	13
1A-2	5.0	0.14	100	14
1A-3	5.0	0.14	100	14
1A-4	5.5	0.15	100	15
1A-5	3.0	0.08	100	8
1A-6	4.5	0.13	100	13
1A-7	2.5	0.07	100	7
1A-8	3.0	0.08	100	8
1A-9	2.5	0.07	100	7
Total Area (A_T)	35.5			

Table 7.4.1
Peak Flow Allotment for Catch Basins within Subarea 1A

7.5 REPORTING RUNOFF VALUES

Reporting official peak flow rates on maps and data sheets requires a standard method. This section describes two methods for flow reporting. The first method is used when reporting flow rates from each subarea and is consistent with the United States Geologic Survey (USGS) flow reporting procedures. The second method is for reporting burned and bulked flow rates using the reach grouping method.

Peak Flow Reporting - USGS Method

The USGS is recognized for expertise in flow measurement and reporting. Flow rates reported for subareas and reaches within The County of Los Angeles must use the USGS rounding rules. Table 7.5.1 shows the rules for reporting flow rates using the USGS standard.

Flow Rate (cfs)	Round Flow To Nearest
$0 \leq Q < 1$	0.01 cfs
$1 \leq Q < 10$	0.1 cfs
$10 \leq Q < 100$	1 cfs
$100 \leq Q < 10{,}000$	10 cfs
$10{,}000 \leq Q < 100{,}000$	100 cfs
$Q \geq 100{,}000$	1,000 cfs

Table 7.5.1
USGS Flow Reporting Rounding Rules

Peak Flow Reporting - Reach Grouping

Reporting flow rates for burned and bulked runoff requires grouping flow rates by reach. A reach is a segment of a watercourse between specified collection points. A grouped reach is a collection of reaches grouped together based on rounding rules listed below. Reach grouping reduces the number of calculations required when bulking flow rates.

Reach grouping involves dividing a watercourse into grouped reaches and then bulking each grouped reach individually. This eliminates the need to bulk flow rates at every collection point along a watercourse. Reach grouping must be used to report burned and bulked flow rates for debris-producing watersheds. The following is the procedure for determining grouped reaches used for bulking.

1. List the burned flow rates (Q_{burn}) for all collection points along the desired watercourse
2. Round the burned flow rates according to the rules in Table 7.5.2
3. Group reaches based on rounded burned flow rates of the same value
4. Determine the Debris Production Area (DPA) zone breakup using the most downstream collection point of the grouped reach to account for all DPA zone areas
5. Bulk the largest non-rounded burned flow rate value from the grouped reach
6. When reporting clear flow rates for the grouped reach, use the largest rounded clear flow rate value from the reaches within the grouped reach

When reporting final grouped reach flow rates, if the flow rate decreases downstream along a watercourse, use the flow rate from the immediate upstream grouped reach.

Table 7.5.2
Rounding Rules for Reach Grouping

Flow Rate (cfs)	Round Flow To Nearest
$0 \le Q_{burn} < 20$	0.1 cfs
$20 \le Q_{burn} < 100$	5 cfs
$100 \le Q_{burn} < 1{,}000$	10 cfs
$1{,}000 \le Q_{burn} < 100{,}000$	100 cfs
$Q_{burn} \ge 100{,}000$	1,000 cfs

EXAMPLE – Reach Grouping for Reporting Bulked Flow Rates

Figure 7.5.1 shows a portion of a watercourse that contains three reaches. Table 7.5.3 shows the burned flow rates for these reaches. Each of the burned flow rates is rounded using the rules in Table 7.5.2. Following the reach grouping steps, the burned flow rates for each collection point are listed and rounded. The flow rate at 6A is the largest unrounded burned flow rate and is used in the bulk flow calculations. The DPA zones are calculated from collection point 8A upstream to include the area tributary to the entire grouped reach and the bulked flow is calculated. The burned and bulked flow is then rounded for reporting based on Table 7.5.2. Chapter 3 of the Sedimentation Manual contains more information on bulking flows.

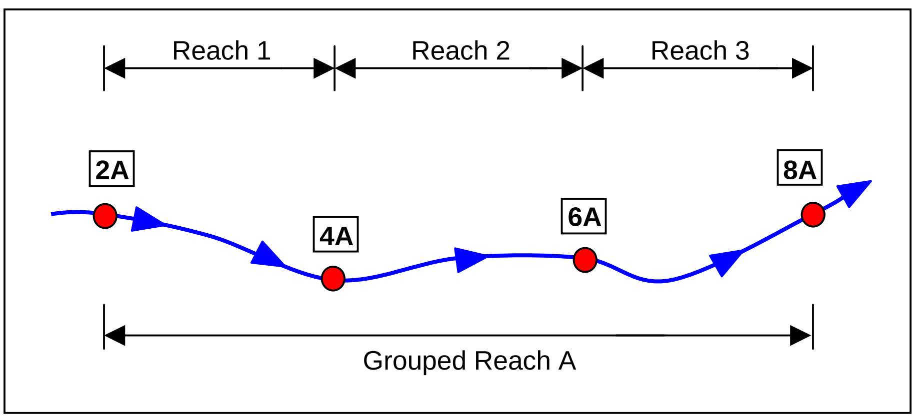

Figure 7.5.1
Grouped Channel Reach Based on Reach Flows

Table 7.5.3
Grouped Reach Flow Rates

Reach	Grouped Reach	Collection Point	50-Year Q_{burn} (cfs)	50-Year Q_{burn} ***Rounded*** (cfs)	50-Year $Q_{burn\ \&\ bulk}$ (cfs)	50-Year $Q_{burn\ \&\ bulk}$ ***Rounded*** (cfs)
1		4A	6,714.7	6,700		
2	A	6A	6,724.6	6,700	8,939.4	8,900
3		8A	6,667.8	6,700		

Figure 7.5.2 shows the aftermath of a bulked flow, downstream of Hook Canyon in Glendora after the January 1969 storm.

Figure 7.5.2
Downstream of Hook Canyon in Glendora
January 26, 1969

1 Mulvaney, T.J. "On the Use of Self-Registering Rain and Flood Gauges. Inst. Civ. Eng. (Ireland) Trans. Vol. 4. pages 1-8. 1851.

2 Bedient, P.B. and W.C. Huber. Hydrology and Floodplain Analysis, 3rd Ed. Prentice-Hall, Inc. NJ. page 84. 2002.

3 US Army Corps of Engineers. Hydraulic Design of Stream Restoration (ERDC/CHL TR-01-28). page 24, Washington, D.C. 2001.

4 Nasseri, I. Use of Kinematic Wave Theory With the Rational Method. ASCE Engineering Workshop on Peak Reduction for Drainage and Flood Control Projects. Proceedings May 9, 1987.

5 Bedient, P.B. and W.C. Huber. Hydrology and Floodplain Analysis, 3rd Ed. Prentice-Hall, Inc. NJ. page 246. 2002.

6 Nasseri, I. Use of Kinematic Wave Theory With the Rational Method. ASCE Engineering Workshop on Peak Reduction for Drainage and Flood Control Projects. Proceedings May 9, 1987. page 132.

7 Los Angeles County Hydrologic Method Approval Memorandum. Los Angeles County Department of Public Works. March 4, 2002.

8 US Army Corps of Engineers. Hydrologic Modeling System HEC-HMS Technical Reference Manual. CPD-74B. March 2000.

9 Bedient, P.B. and W.C. Huber. Hydrology and Floodplain Analysis, 3rd Ed. Prentice-Hall, Inc. NJ. page 246. 2002.

CHAPTER

8

Reservoir and Basin Routing

Reservoirs and detention ponds are an important aspect of water resources management. Reservoirs and detention ponds change runoff timing and peak runoff rates while storing flows. Hydrologic studies must consider these effects when evaluating existing conditions or planning for future changes within the watershed. Figure 8.1 shows the San Gabriel Reservoir on April 28, 1975.

Figure 8.1
San Gabriel Reservoir
April 28, 1975

Reservoir routing for hydrologic studies within the County of Los Angeles uses the Modified Puls or Level Pool routing method. The method is similar to the method for channel routing, except that no translation is considered. Section 7.3, Channel Routing of Flows discusses the concepts of the

Modified Puls method in more detail. Equation 8.1 is the finite difference form of the continuity equation used for reservoir routing[1]. Equation 8.2 provides a relationship that is used to calculate outflow without actually calculating storage for a given time step. The example problem illustrates use of the equations.

$$\left(I_n + I_{n+1}\right) + \left(\frac{2S_n}{\Delta t} - O_n\right) = \left(\frac{2S_{n+1}}{\Delta t} + O_{n+1}\right)$$

Equation 8.1
Form of the Continuity Equation Used for Reservoir Routing

$$\left(\frac{2S_n}{\Delta t} - O_n\right) = \left(\frac{2S_n}{\Delta t} + O_n\right) - 2O_n$$

Equation 8.2
Relationship Used to Calculate Outflow Without Calculating Storage

Where:
- I_n = Inflow at $time_n$
- I_{n+1} = Inflow at $time_{n+1}$
- Δt = Difference in time, $time_{n+1}$ - $time_n$
- S_n = Storage at $time_n$
- S_{n+1} = Storage at $time_{n+1}$
- O_n = Outflow at $time_n$
- O_{n+1} = Outflow at $time_{n+1}$

Reservoir routing using the Modified Puls method requires a storage-elevation relationship, an outflow-elevation relationship, and an inflow hydrograph. The relationships, the inflow hydrograph, and a known initial storage condition provide the information necessary to calculate outflow. The following example illustrates the use of the Modified Puls reservoir routing method.

EXAMPLE – Modified Puls Routing Through a Reservoir

This example routes an inflow hydrograph through a simple detention basin. Figure 8.2 defines the inflow hydrograph to be routed through the detention basin in this example.

The detention basin has the storage capacity shown in Table 8.1. Outflow from the basin occurs through an 8-inch drain when the water surface elevation is below 6 feet. When the water surface elevation is above 6 feet, outflow occurs through the drainpipe and over a weir. The weir is 20 feet long and has a weir coefficient of 3.5. Equations 8.3 and 8.4 provide the outflow relationships for the weir and drainpipe based on elevation as shown in Table 8.1.

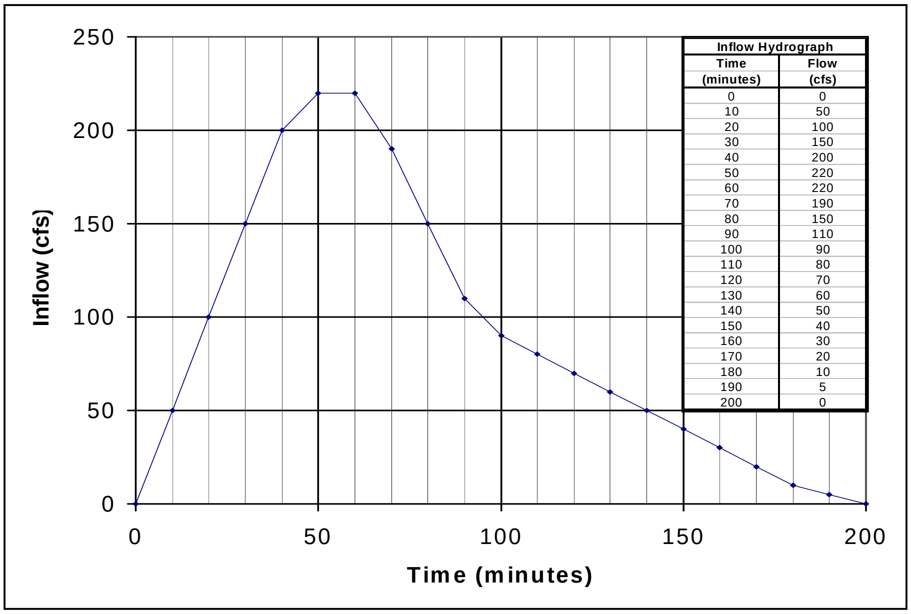

Figure 8.2
Inflow Hydrograph

Table 8.1 contains the storage-elevation and outflow-elevation relationships for this example. When outflow is based only on storage and no inflow is entering the reservoir, these relationships provide enough information to calculate outflow for a specified water surface. If there is inflow occurring at the same time as outflow, the Modified Puls method can be used to calculate outflow. The method requires building a storage indication curve using a specific time interval. The time interval must equal the time interval for the inflow hydrograph. This example uses a 10-minute time interval.

$$Q = CLH^{1.5}$$

Equation 8.3
Weir Flow Equation

$$Q = KA\sqrt{2gH}$$

Equation 8.4
Orifice Flow Equation

Where:
- Q = Outflow in cfs
- C = Weir Coefficient, 3.5
- L = Length of weir crest in feet
- H = Water surface elevation above weir in feet
- K = Orifice flow coefficient, 0.65
- A = Cross sectional area of orifice in ft^2
- g = Gravitational acceleration in ft/sec^2

Water Surface Elevation (ft)	Storage (ft^3)	Orifice Outflow (cfs)	Weir Outflow (cfs)	Total Outflow (cfs)	$2S/\Delta t+O$ (cfs)	$2S/\Delta t-O$ (cfs)
0.0	0	0.0	0.0	0.0	0.0	0.0
0.5	30,000	11.6	0.0	11.6	111.6	88.4
1.0	60,000	16.4	0.0	16.4	216.4	183.6
1.5	80,000	20.1	0.0	20.1	286.7	246.6
2.0	100,000	23.2	0.0	23.2	356.5	310.2
2.5	120,000	25.9	0.0	25.9	425.9	374.1
3.0	140,000	28.4	0.0	28.4	495.1	438.3
3.5	160,000	30.7	0.0	30.7	564.0	502.7
4.0	180,000	32.8	0.0	32.8	632.8	567.2
4.5	200,000	34.8	0.0	34.8	701.4	631.9
5.0	220,000	36.6	0.0	36.6	770.0	696.7
5.5	230,000	38.4	0.0	38.4	805.1	728.2
6.0	240,000	40.1	0.0	40.1	840.1	759.9
6.5	250,000	41.8	24.7	66.5	899.9	766.8
7.0	260,000	43.4	70.0	113.4	980.0	753.3
7.5	300,000	44.9	128.6	173.5	1173.5	826.5
8.0	340,000	46.4	198.0	244.3	1377.7	889.0

Table 8.1
Storage-Elevation and Outflow-Elevation Relationships

Figure 8.3 plots the storage indication curves for this detention pond using the 10-minute time increment. The storage indication curve relates storage to outflow and provides a graphical method for calculating outflow based on the Modified Puls Method. Without the graph, solving for outflow requires interpolation of Table 8.1.

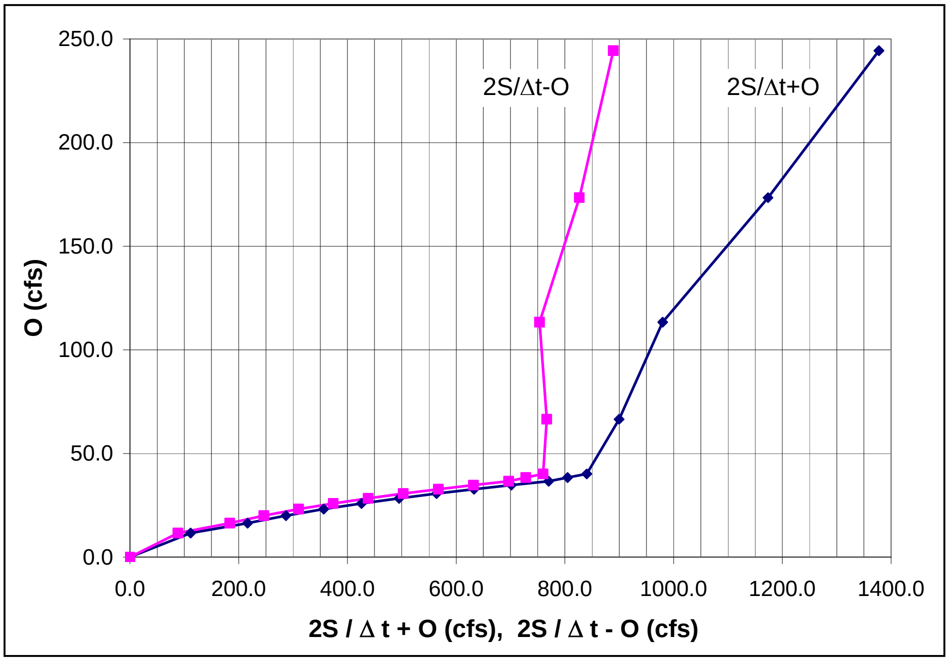

Figure 8.3
Storage-Indication Curve Based on 10-minute Time Interval

The storage-indication curve relates outflow to storage. Routing the flow through a reservoir requires solving graphically, or setting up a spreadsheet or computer program to perform the following steps:

1. Determine the initial storage, inflow, and outflow conditions and the inflow at the first time step (S_n, I_n, O_n, and I_{n+1}). The inflow cannot be greater than the outflow for the first time step.

2. Use the storage-indication curve to determine the storage and outflow for the second time step (S_{n+1} and O_{n+1}).

3. Repeat the steps 1 and 2 until the outflow hydrograph is completed.

The initial values for this example are:

S_1 = 0 ft^3
I_1 = 0 cfs
O_1 = 0 cfs
I_2 = 50 cfs
Δt = (10 minutes)*(60 sec/minute) = 600 sec

The initial values provide a solution to determine the first value on the storage indication curve. This value is calculated as follows:

$$(I_1 + I_2) + \left(\frac{2S_1}{\Delta t} - O_1\right) = \left(\frac{2S_2}{\Delta t} + O_2\right) \Rightarrow$$
$$(0 + 50) + (0) = \left(\frac{2S_2}{\Delta t} + O_2\right) = 50$$

The outflow value for the second time step is found by reading the storage indication curve for 50 cfs along the X-axis and finding the Y-axis value, or by interpolating between the values shown in the last two columns of Table 8.1.

O_2 = 5.2 cfs (from storage indication curve)

The outflow at 10 minutes is 5.2 cfs. This value then provides the information for the next time step.

Equation 8.2 provides the values for $2S_n/\Delta t - O_n$ at time steps after the initial time step:

$$\left(\frac{2S_2}{\Delta t} - O_2\right) = \left(\frac{2S_2}{\Delta t} + O_2\right) - 2O_2$$

The calculation for the second time step value of $2S_n/\Delta t - O_n$ is:

$$\left(\frac{2S_2}{\Delta t} - O_2\right) = (50) - 2(5.2) = 39.6 \text{ cfs}$$

The values for the second iteration are:

I_2	= 50 cfs
O_2	= 5.2 cfs
I_3	= 100 cfs
Δt	= (10 minutes)*(60 sec/minute) = 600 sec

$$(I_2 + I_3) + \left(\frac{2S_2}{\Delta t} - O_2\right) = \left(\frac{2S_3}{\Delta t} + O_3\right) \Rightarrow$$
$$(50 + 100) + (39.6) = \left(\frac{2S_3}{\Delta t} + O_3\right) \Rightarrow 189.6$$

O_3 = 15.2 cfs (from storage indication curve)

Spreadsheets facilitate the Modified Puls calculations for reservoir routing. Table 8.2 provides the rest of the calculations for the detention basin routing problem. Many computer programs use this method to calculate outflow from reservoirs and detention basins.

Table 8.2
Outflow Hydrograph Calculation Using Modified Puls Method

Inflow Hydrograph			Outflow Hydrograph Calculations			
Time Index	Time (min)	Inflow (I_n) (cfs)	I_n+I_{n+1} (cfs)	$2S/\Delta t$-O (cfs)	$2S/\Delta t$+O (cfs)	Outflow O_{n+1} (cfs)
1	0	0.0	0.0	0.0	0.0	0.0
2	10	50.0	50.0	0.0	50.0	5.2
3	20	100.0	150.0	39.6	189.6	15.2
4	30	150.0	250.0	159.3	409.3	25.3
5	40	200.0	350.0	358.8	708.8	35.0
6	50	220.0	420.0	638.9	1058.9	137.9
7	60	220.0	440.0	783.1	1223.1	190.7
8	70	190.0	410.0	841.7	1251.7	200.6
9	80	150.0	340.0	850.5	1190.5	179.4
10	90	110.0	260.0	831.7	1091.7	148.1
11	100	90.0	200.0	795.6	995.6	118.2
12	110	80.0	170.0	759.2	929.2	83.7
13	120	70.0	150.0	761.9	911.9	73.5
14	130	60.0	130.0	764.8	894.8	64.3
15	140	50.0	110.0	766.2	876.2	56.1
16	150	40.0	90.0	764.1	854.1	46.3
17	160	30.0	70.0	761.5	831.5	39.7
18	170	20.0	50.0	752.0	802.0	38.3
19	180	10.0	30.0	725.5	755.5	36.2
20	190	5.0	15.0	683.0	698.0	34.7
21	200	0.0	5.0	628.7	633.7	32.8
22	210	0.0	0.0	568.1	568.1	30.8
23	220	0.0	0.0	506.5	506.5	28.8
24	230	0.0	0.0	449.0	449.0	26.7
25	240	0.0	0.0	395.5	395.5	24.7
26	250	0.0	0.0	346.1	346.1	22.7
27	260	0.0	0.0	300.7	300.7	20.7
28	270	0.0	0.0	259.3	259.3	18.6
29	280	0.0	0.0	222.0	222.0	16.7
30	290	0.0	0.0	188.7	188.7	15.1
31	300	0.0	0.0	158.4	158.4	13.7
32	310	0.0	0.0	131.0	131.0	12.5
33	320	0.0	0.0	106.0	106.0	11.0
34	330	0.0	0.0	84.0	84.0	8.7
35	340	0.0	0.0	66.5	66.5	6.9
36	350	0.0	0.0	52.7	52.7	5.5
37	360	0.0	0.0	41.8	41.8	4.3
38	370	0.0	0.0	33.1	33.1	3.4
39	380	0.0	0.0	26.2	26.2	2.7
40	390	0.0	0.0	20.8	20.8	2.2
41	400	0.0	0.0	16.5	16.5	1.7
42	410	0.0	0.0	13.0	13.0	1.4
43	420	0.0	0.0	10.3	10.3	1.1

[1] Bedient, P.B. and W.C. Huber. Hydrology and Floodplain Analysis, 3rd Ed. Prentice-Hall, Inc. NJ. page 256. 2002.

CHAPTER

9

Water Quality Hydrology

Water quality has been an important aspect of water resources planning and use for many years in Southern California[1]. Regulations protect water quality and seek to limit pollution in part by requiring that new developments meet certain criteria for pollution prevention. Other regulations sometimes result in the retrofitting of existing storm water conveyances to reduce pollution of impaired receiving water bodies. Since problems with the quality of runoff can be associated with common rainfall events, smaller, more frequent storms must be addressed. This section discusses several of the issues that relate hydrology to water quality issues.

9.1 STANDARD URBAN STORMWATER MITIGATION PLANS (SUSMP)[2]

The Standard Urban Stormwater Mitigation Plan (SUSMP) is part of the Development Planning Program of the National Pollution Discharge Elimination System, Phase I, Stormwater Permit for the County of Los Angeles. SUSMP applies to development and redevelopment projects within the County that fall within specific categories. The County of Los Angeles has developed a SUSMP manual that includes the permitting and inspection process for projects required to meet SUSMP regulations. Table 9.1.1 provides a summary of the types of development and activities that fall under SUSMP regulation. The SUSMP manual provides more specific information.

Table 9.1.1
Development or Redevelopment Activities Regulated by SUSMP

Development Type and Activities
• Single-family hillside homes
• Residential development of ten or more units
• Industrial/commercial developments with 1 acre or more of impervious surface area
• Automotive service facilities
• Retail gasoline outlets
• Restaurants
• Parking lots 5,000 ft^2 or more of surface area or with 25 or more parking spaces
• Redevelopment projects in these categories that meet redevelopment thresholds
• Locations within or directly adjacent to or discharging directly to an environmentally sensitive area
• Fueling Areas
• Equipment maintenance, washing and repair areas
• Commercial/Industrial waste handling or storage
• Outdoor hazardous material handling or storage
• Outdoor manufacturing areas
• Outdoor food handling or processing
• Outdoor animal care, confinement, or slaughter
• Outdoor horticultural activities

The objective of SUSMP is to effectively prohibit non-storm water discharges and reduce the discharge of pollutants from storm water conveyance systems to the Maximum Extent Practicable (MEP) statutory standard. SUSMP defines hydrology standards for designing volumetric and flow rate based Best Management Practices (BMPs).

Design of BMPs to meet hydrologic standards for SUSMP must follow the methods outlined in the SUSMP manual. The design must mitigate flows or volumes using one of the required runoff calculations.

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