CHAPTER 810 – HYDROLOGY

Topic 811 – General

Index 811.1 – Introduction

Hydrology is often defined as: "A science dealing with the properties, distribution, and circulation of water on the surface of the land, in the soil and underlying rocks, and in the atmosphere." This is a very broad definition encompassing many disciplines relating to water. The highway engineer is principally concerned with surface hydrology and controlling surface runoff. Controlling runoff includes the hydraulic design of drainage features for both cross highway drainage (Chapter 820) and removal of runoff from the roadway (Chapter 830).

The runoff of water over land has long been studied and some rather sophisticated theories and methods have been proposed and developed for estimating flood flows. Most attempts to describe the process have been only partially successful at best. This is due to the complexity of the process and interactive factors. The random nature of rainfall, snowmelt, and other sources of water further complicate the process.

It should be understood that there are no exact methods for hydrologic analysis. Different methods that are commonly used may produce significantly different results for a specific site and particular situation.

Although hydrology is not an exact science, it is possible to obtain solutions which are functionally acceptable to form the basis for design of highway drainage facilities.

More complete information on the principles and engineering techniques pertaining to hydrology for transportation and highway engineers may be found in FHWA Hydraulic Design Series (HDS) No. 2, Highway Hydrology.

This chapter will focus primarily on the hydrologic analyses that are conducted for peak flow facilities for both transportation facility and cross drainage. In many cases, these peak flow facilities serve dual purposes and receive and convey storm water flows while meeting water quality criteria and other flow criteria independent of Chapter 810. Information related to the designer's responsibility for the hydrologic design of storm water flow facilities is contained in the Department's Project Planning and Design Guide. See: http://www.dot.ca.gov/hq/oppd/stormwtr/ppdg.htm

811.2 Objectives of Hydrologic Analysis

Regardless of the size or cost of the drainage feature the most important step prior to hydraulic design is estimating the discharge (rate of runoff) or volume of runoff that the drainage facility will be required to convey or control.

While some hydrologic analysis is necessary in establishing the quantity of surface water that must be considered in the design of all highway drainage facilities, the extent of such studies are to be commensurate with the importance of the highway, the potential for damage to the highway, loss of property, and hazard to life associated with the facilities.

The choice of analytical method must be a conscious decision made as each problem arises. To make an informed decision, the highway engineer must determine:

- What level of hydrologic analysis is justified.
- What data are available or must be collected.
- What methods of analysis are available including the relative strengths and weaknesses in terms of cost and accuracy.

Cross drainage design, Chapter 820, normally requires more extensive hydrologic analysis than is necessary for roadway drainage design, Chapter 830. The well known and relatively simple "Rational Method" (see Index 819.2) is generally adequate for estimating the rate or volume of runoff for the design of on-site roadway drainage facilities and removal of runoff from highway pavements.

811.3 Peak Discharge

Peak discharge is the maximum rate of flow of water passing a given point during or after a rainfall event. Peak discharge, often called peak flow, occurs at the momentary "peak" of the stream's flood hydrograph. (See Index 816.5, Flood Hydrograph.)

Design discharge, expressed as the quantity (Q) of flow in cubic feet per second (CFS), is the peak discharge that a highway drainage structure is sized to handle. Peak discharge is different for every storm and it is the highway engineer's responsibility to size drainage facilities and structures for the magnitude of the design storm and flood severity. The magnitude of peak discharge varies with the severity of flood events which is based on probability of exceedance (see Index 811.4). The selection of design storm frequency and flood probability are more fully discussed under Topic 818, Flood Probability and Frequency.

811.4 Flood Severity

Flood severity is usually stated in terms of:

- Probability of Exceedance, or
- Frequency of Recurrence.

Modern concepts tend to define a flood in terms of probability. Probability of exceedance, the statistical odds or chance of a flood of given magnitude being exceeded in any year, is generally expressed as a percentage. Frequency of recurrence is expressed in years, on the average, that a flood of given magnitude would be predicted. Refer to Topic 818 for further discussion of flood probability and frequency.

811.5 Factors Affecting Runoff

The highway engineer should become familiar with the many factors or characteristics that affect runoff before making a hydrologic analysis. The effects of many of the factors known to influence surface runoff only exist in empirical form. Extensive field data, empirically determined coefficients, sound judgment, and experience are required for a quantitative analysis of these factors. Relating flood flows to these causative factors has not yet advanced to a level of precise mathematical expression.

Some of the more significant factors which affect the hydraulic character of surface water runoff are categorized and briefly discussed in Topics 812 through 814. It is important to recognize that the factors discussed may exist concurrently within a watershed and their combined effects are very difficult to quantify.

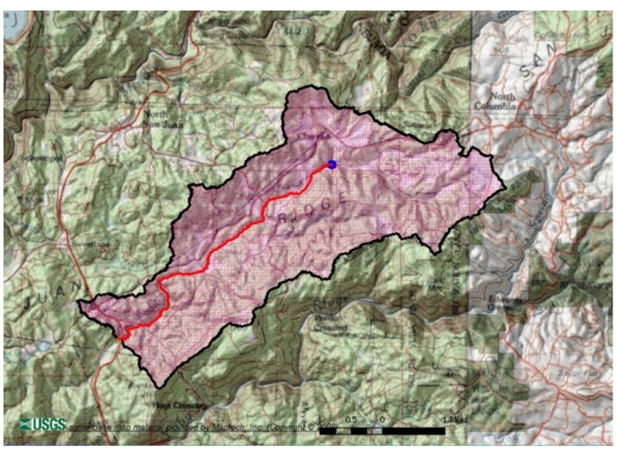
Topic 812 – Basin Characteristics

812.1 Size

The size (area) of a drainage basin is the most important watershed characteristic affecting runoff. Determining the size of the drainage area that contributes to flow at the site of the drainage structure is a basic step in a hydrologic analysis regardless of the method used to evaluate flood flows. The drainage area typically expressed in acres or square miles, is frequently determined from digital elevation maps (DEMs), field surveys, topographic maps, or aerial photographs. Automated watershed delineation is included within several of the software programs indicated under the "Hydrology" column of Table 808.1, e.g., USGS StreamStats and WMS. See Figure 812.1.

Figure 812.1

Automated Watershed Delineation



812.2 Shape

The shape, or outline formed by the basin boundaries, affects the rate at which water is supplied to the main stream as it proceeds along its course from the runoff source to the site of the drainage structure. Long narrow watersheds generally give lower peak discharges than do fan or pear shaped basins.

812.3 Slope

The slope of a drainage basin is one of the major factors affecting the time of overland flow and concentration of rainfall (see Index 816.6,Timeof Concentration). Steep slopes tend to result in shorter response time and increase the discharge while flat slopes tend to result in longer response time and reduce the discharge. Automated basin slope calculation is included within several of the software programs indicated under the "Hydrology" column of Table 808.1, e.g., USGS StreamStats and WMS.

812.4 Land Use

Changes in land use nearly always cause increases in surface water runoff. Of all the land use changes, urbanization is the most dominant factor affecting the hydrology of an area.

Land use studies may be necessary to define present and future conditions with regard to urbanization or other changes expected to take place within the drainage basin.

Valuable information concerning land use trends is available from many sources such as:

- State, regional or municipal planning organizations.
- U.S. Geological Survey.
- U.S. Department of Agriculture Economic Research Service.

Within each District there are various organizations that collect, publish or record land use information. The District Hydraulics Engineer should be familiar with these organizations and the types of information they have available.

A criterion of good drainage design is that future development and land use changes which can reasonably be anticipated to occur during the design life of the drainage facility be considered in the hydraulic analysis and estimation of design discharge.

812.5 Soil and Geology

The type of surface soil which is characteristic of an area is an important consideration for any hydrologic analysis and is a basic input to the National Resources Conservation Service (NRCS) method. Rock formations underlying the surface soil and other geophysical characteristics such as volcanic, glacial, and river deposits can have a significant effect on run-off.

The major source of soil information is the National Resources Conservation Service (NRCS) of the U.S. Department of Agriculture.

Use the following link to access soil information at the NRCS Web Soil Survey website: http://websoilsurvey.nrcs.usda.gov/app/.

812.6 Storage

Interception and depression storage are generally not important considerations in highway drainage design and may be ignored in most hydrologic analysis. Interception storage is rainfall intercepted by vegetation and never becomes run-off. Depression storage is rainfall lost in filling small depressions in the ground surface, storage in transit (overland or channel flow), and storage in ponds, lakes or swamps.

Detention storage can have a significant effect in reducing the peak rate of discharge, but this is not always the case. There have been rare instances where artificial storage radically redistributes the discharges and higher peak discharges have resulted than would occur had the storage not been added.

The effect of flood-control reservoirs should be considered in evaluating downstream conditions, flood peaks, and river stages for design of highway structures. The controlling public agency or the owner should be contacted for helpful information on determining the effects, if any, on downstream highway drainage structures.

It is not uncommon for flood control projects to be authorized but never constructed because funds are not appropriated. Therefore a flood control project should exist or be under construction if its effects on a drainage system are to be considered.

812.7 Elevation

The mean elevation of a drainage basin and significant variations in elevation within a drainage basin may be important characteristics affecting run-off, particularly with respect to precipitation falling as snow. Elevation is a basic input to some of the USGS Regional Regression Equations (see Index 819.2(2).

812.8 Orientation

The amount of runoff can be affected by the orientation of the basin. Where the general slope of the drainage basin is to the south it will receive more exposure to the heat of the sun than will a slope to the north. Such orientation affects transpiration, evaporation, and infiltration losses. Snowpack and the rate at which snow melts will also be affected. A basin's orientation with respect to the direction of storm movement can affect a flood peak. Storms moving upstream produce lower peaks than storms tending to move in the general direction of stream flow.

Topic 813 – Channel and Floodplain Characteristics

813.1 General

Streams are formed by the gathering together of surface waters into channels that are usually well defined. The natural or altered condition of the channels can materially affect the volume and rate of runoff and is a significant consideration in the hydrological analysis for cross drainage design.

A useful reference relative to issues associated with transverse and longitudinal highway encroachments upon river channels and floodplains is the FHWA Hydraulic Design Series (HDS) No. 6 "River Engineering for Highway Encroachments."

813.2 Length and Slope

The longer the channel the more time it takes for water to flow from the beginning of the channel to the site under consideration. Channel length and effective channel slope are important parameters in determining the response time of a watershed to precipitation events of given frequency.

In the case of a wide floodplain with a meandering main channel the effective channel length will be reduced during flood stages when the banks are overtopped and flow tends more toward a straight line.

813.3 Cross Section

Flood peaks may be estimated by using data from stream gaging stations and natural channel cross section information.

Although channel storage is usually ignored in the hydrologic analysis for the design of highway drainage structures, channel cross section may significantly affect discharge, particularly in wide floodplains with heavy vegetation.

If channel storage is considered to be a significant factor, the assistance of an expert in combining the analysis of basin hydrology and stream hydraulics should be sought. The U.S. Army Corps of Engineers has developed HEC-HMS Flood Hydrograph Package and HEC-RAS, Water Surface Profiles, for this type of analysis. For modeling complex water surface profiles, where one-dimensional models fail, the Finite Element Surface Water Modeling System Two Dimensional Flow in a Horizontal Plane (SMS) was developed by others.

813.4 Hydraulic Roughness

Hydraulic roughness represents the resistance to flows in natural channels and floodplains. It affects both the time response of a drainage channel and channel storage characteristics. The lower the roughness, the higher the peak discharge and the shorter the time of the resulting hydrograph. The total volume of runoff however is virtually independent of hydraulic roughness.

Streamflow is frequently indirectly computed by using Manning's equation, see Index 866.3(4). Procedures for selecting an appropriate coefficient of hydraulic roughness, Manning's "n", may be found in the FHWA report, "Guide for Selecting Manning's Roughness Coefficient for Natural Channels and Flood Plains". See http://www.fhwa.dot.gov/bridge/wsp2339.pdf

813.5 Natural and Man-made Constrictions

Natural constrictions, such as gravel bars, rock outcrops and debris jams as well as artificial constrictions such as diversion and storage dams, grade-control structures, and other water-use facilities may control or regulate flow. Their effect on the flood peak may be an important consideration in the hydrologic analysis.

813.6 Channel Modifications

Channel improvements such as channel-straightening, flood control levees, dredging, bank clearing and removal of obstructions tend to reduce natural attenuation and increase downstream flood peaks.

813.7 Aggradation - Degradation

Aggradation, deposited sediments, may lessen channel capacity and increase flood heights causing overflow at a lower discharge. Degradation, the lowering of the bed of a stream or channel, may increase channel capacity and result in a higher peak discharge.

The validity of hydrologic analysis using observed historical highwater marks may be affected by aggradation or degradation of the streambed. The effects of aggradation and degradation are considerations in selecting an effective drainage system design to protect highways and adjacent properties from damage. For more information refer to the FHWA report entitled, "Stream Channel Degradation and Aggradation: Causes and Consequences to Highways." See http://isddc.dot.gov/OLPFiles/FHWA/009471.pdf.

813.8 Debris

The quantity and size of solid matter carried by a stream may affect the hydrologic analysis of a drainage basin. Bulking due to mud, suspended sediment and other debris transported by storm runoff may significantly increase the volume of flow, affect flow characteristics, and can be a major consideration in the hydraulic design of drainage structures. In particular, bulking factors are typically a consideration in determining design discharges for facilities with watersheds that are located within mountainous regions subject to fire and subsequent soil erosion (see Figure 813.1), or in arid regions when the facility is in the vicinity of alluvial fans (see Index 819.7(2) and Index 872.3(5) for special considerations given to highways located across desert washes).

Debris control methods, structures, and design considerations are discussed in Topic 822, Debris Control.

The District Hydraulics Engineer should be consulted for any local studies that may be available. If both stream gage data and local studies are available, a determination of whether post-fire peak flows are included within the data record should be made. Consideration should be given to treating a significant post-fire peak as the design discharge in lieu of the peak discharge obtained through gage analysis for a given probability flood event. Records of stream discharge from burned and long-unburned (unburned for 40 years or more years) areas have showed peak discharge increases from 2 to 30 times in the first year after burning. In mountainous regions subject to fire with no local studies available, the U.S. Forest Service should be contacted for fire history in order to determine if there is a significant post-fire peak within the stream records.

Figure 813.1

Post-Fire Debris

Alamos Canyon, Ventura County, post-fire debris and plugged culvert barrels (Highway 118)

Topic 814 – Meteorological Characteristics

814.1 General

Meteorology is the science dealing with the earth's atmosphere, especially the weather. As applied to hydrology for the highway designer the following elements of meteorological phenomena are considered the more important factors affecting runoff and flood predictions.

814.2 Rainfall

Rainfall is the most common factor used to predict design discharge. Unfortunately, due to the many interactive factors involved, the relationship between rainfall and runoff is not all that well defined. Intuitively, engineers know and studies confirm, that runoff increases in proportion to the rainfall on a drainage basin. Highway design engineers are cautioned about assuming that a given frequency storm always produces a flood of the same frequency. There are analytical techniques for ungaged watersheds that are based on this assumption.

A statistical analysis of extensive past rainfall records should be made before such a correlation is accepted.

Rainfall event characteristics which are important to highway drainage design are:

- Intensity (rate of rainfall)
- Duration (time rainfall lasts)
- Frequency (statistical probability of how often rainfall will occur)
- Time Distribution (intensity hyetograph)
- Storm Type (orographic, convective or cyclonic)
- Storm Size (localized or broad areal extent)
- Storm Movement (direction of storm)

814.3 Snow

Much of the precipitation that falls in the mountainous areas of the state falls as frozen water in the form of snow, hail, and sleet. Since frozen precipitation cannot become part of the runoff until melting occurs it is stored as snowpack until thawed by warmer weather.

Rain upon an accumulation of snow can cause a much higher peak discharge than would occur from rainfall alone. The parameters of snow which may need to be considered in quantifying peak flood runoff are:

- Mean annual snowfall
- Water content of snowpack
- Snowmelt rate

814.4 Evapo-transpiration

Evaporation and transpiration are two natural processes by which water reaching the earth's surface is returned to the atmosphere as vapor. The losses due to both phenomena are important to long term hydrology and water balance in the watershed and are usually ignored in the hydrologic analysis for the design of highway drainage facilities.

814.5 Tides and Waves

The combined effect of upland runoff and tidal action is a primary consideration in the design of highway drainage structures and shore protection facilities along the coastlines, on estuaries, and in river delta systems.

The time and height of high and low water caused by the gravitational attraction of the sun and moon upon the earth's oceans are precisely predictable. Information on gravitational tides and tidal bench marks for the California Coastline is available from the following report: http://www.slc.ca.gov/reports/ca_marine_boundary_program_final_report.pdf or from the following web-site: http://co-ops.nos.noaa.gov/sitemap.html.

One of the most devastating forces affecting the coastline occurs when an astronomical high tide and a storm of hurricane proportion arrive on the land at the same time. This is also true of the effect of a tsunami. A tsunami is a wave caused by an earthquake at sea. If shore protection were designed to withstand the forces of a tsunami, it would be extremely costly to construct. Since it would be so costly and the probability of occurrence is so slight, such a design may not be justified.

Wind-waves directly affect coastal structures and cause dynamic changes in coastal morphology. The U.S. Corps of Engineers collects and publishes data which may be used to predict size of Pacific Coast wind-waves. Information pertaining to the California coastline from the Mexican border north to Cape San Martin can be obtained from:

U.S. Army Corps of Engineers
Los Angeles District
915 Wilshire Blvd., Suite 1101
Los Angeles, CA 90017
(213) 452-3333

For information from Cape San Martin to the Oregon border from:

U.S. Army Corps of Engineers
San Francisco District
1455 Market Street
San Francisco, CA 94103-1398
(415) 503-6804

Also see the following website for USGS Coastal Storm Modeling System (CoSMoS) for detailed predictions of storm-induced coastal flooding, erosion and cliff failures over large geographic scales: http://walrus.wr.usgs.gov/coastal_processes/cosmos/

Wind-waves are also generated on large inland bodies of water and their effect should be considered in the design of shoreline highway facilities.

Topic 815 – Hydrologic Data

815.1 General

The purpose for which a hydrologic study is to be made will determine the type and amount of hydrologic data needed. The accuracy necessary for preliminary studies is usually not as critical as the desirable accuracy of a hydrologic analysis to be used for the final design of highway drainage structures. If data needs can be clearly identified, data collection and compilation efforts can be tailored to the importance of the project.

Data needs vary with the methods of hydrologic analysis. Highway engineers should remember that there is no single method applicable to all design problems. They should make use of whatever hydrologic data that has been developed by others whenever it is available and applicable to their needs.

Frequently there is little or no data available in the right form for the project location. For a few locations in the State, so much data has been compiled that it is difficult to manage, store, and retrieve the information that is applicable to the project site.

815.2 Categories

For most highway drainage design purposes there are three primary categories of hydrologic data:

(1) *Surface Water Runoff.* This includes daily and annual averages, peak discharges, instantaneous values, and highwater marks.

(2) *Precipitation.* Includes rainfall, snowfall, hail, and sleet.

(3) *Drainage Basin Characteristics.* Adequate information may not be readily available but can generally be estimated or measured from maps, field reviews or surveys. See Topic 812 for a discussion of basin characteristics.

Other special purpose categories of hydrologic data which may be important to specific problems associated with a highway project are:

- Sediment and debris transport
- Snowpack variations
- Groundwater levels and quantity
- Water quality

815.3 Sources

Hydrologic data necessary for the design of cross drainage (stream crossings) are usually obtained from a combination of sources.

(1) *Field Investigations.* A great deal of the essential information can only be obtained by visiting the site. Except for extremely simple designs or the most preliminary analysis, a field survey or site investigation should always be made.

To optimize the amount and quality of the hydrologic data collected, the field survey should be well planned and conducted by an engineer with general knowledge of drainage design. Data collected are to be documented. When there is reason to believe that sensitive resources or unusual site conditions may exist, preparation of a written report with maps and photographs may be appropriate. See Topic 804 for Floodplain Encroachments. Index 3.1.1 of HDS No. 2 discusses site investigations and field surveys. Typical data collected in a field survey are:

- Highwater marks
- Performance and condition of existing drainage structures
- Stream alignment
- Stream stability and scour potential
- Land use and potential development
- Location and nature of physical and cultural features
- Vegetative cover
- Upstream constraints on headwater elevation
- Downstream constraints
- Debris potential

(2) *Federal Agencies.* The following agencies collect and disseminate stream flow data:

- Geological Survey (USGS)
- Corps of Engineers (COE)
- Bureau of Reclamation (USBR)
- National Resources Conservation Service (NRCS)
- Forest Service (USFS)
- Bureau of Land Management (BLM)
- Federal Emergency Management Agency (FEMA)
- Environmental Protection Agency (EPA)

The USGS is the primary federal agency charged with collecting and maintaining water related data. Stream-gaging station data and other water related information collected by the USGS is published in Water Supply Papers and through the USGS Office of Surface Water website. The USGS web-based tool StreamStats provides streamflow statistics, drainage-basin characteristics, and other information for user-selected sites on streams. See http://water.usgs.gov/osw/streamstats.

(3) *State Agencies*. The primary state agency collecting stream-gaging and precipitation (rain-gage and snowfall) data is the California Department of Water Resources (DWR). The California Data Exchange Center (CDEC) installs, maintains, and operates an extensive hydrologic data collection network including automatic snow reporting gages and precipitation and river stage sensors. See http://cdec.water.ca.gov/index.html.

(4) *Local Agencies.* Entities such as cities, counties, flood control districts, or local improvement districts study local drainage conditions and are often a valuable source of hydrologic data.

(5) *Private Sector.* Water using industries or utilities, railroads and local consultants frequently have pertinent hydrologic records and studies available.

815.4 Stream Flow

Once surface runoff water enters into a stream, it becomes "stream flow". Stream flow is the only portion of the hydrologic cycle in which water is so confined as to make possible reasonably accurate measurements of the discharges or volumes involved.

The two most common types of stream flow data are:

- Gaging Stations – data generally based on recording gage station observations with detailed information about the stream channel cross section. Current meter measurements of transverse channel velocities are made to more accurately reflect stream flow rates.
- Historic – data based on observed high water mark and indirect stream flow measurements.

Stream flow data are usually available as mean daily flow or peak daily flow. Daily flow is a measurement of the rate of flow in cubic feet per second (CFS) for the 24-hour period from midnight to midnight.

"Paleoflood" (ancient flood) data has been found useful in extending stream gaging station records. (See Topic 817 for further discussion on measuring stream flow)

815.5 Precipitation

Precipitation data is collected by recording and non-recording rain gages. Precipitation collected by vertical cylindrical rain gages is designated as "point rainfall".

Regardless of the care and precision used, precipitation measurements from rain gages have inherent and unavoidable shortcomings. Snow and wind problems frequently interrupt rainfall records. Extreme precipitation data from recording rain gage charts are generally underestimated.

Rain gage measurements are seldom used directly by highway engineers. The statistical analysis which must be done with precipitation measurements is nearly always performed by qualified hydrologists and meteorologists.

NOAA's Atlas 14 is an example of precipitation data that has been converted into formats usable by designers. See http://hdsc.nws.noaa.gov/hdsc/pfds/.

815.6 Adequacy of Data

All hydrologic data that has been collected must be evaluated and compiled into a usable format. Experience, knowledge and judgment are an important part of data evaluation. It must be ascertained whether the data contains inconsistencies or other unexplained anomalies which might lead to erroneous calculations and conclusions that could result in the over design or under design of drainage structures.

Topic 816 – Runoff

816.1 General

The process of surface runoff begins when precipitation exceeds the requirements of:

- Vegetal interception.
- Infiltration into the soil.
- Filling surface depressions (puddles, swamps and ponds). As rain continues to fall, surface waters flow down slope toward an established channel or stream.

816.2 Overland Flow

Overland flow is surface waters which travel over the ground as sheet flow, in rivulets and in small channels to a watercourse.

816.3 Subsurface Flow

Waters which move laterally through the upper soil surface to streams are called "interflow" or "subsurface flow". For the purpose of highway drainage hydrology, where peak design discharge (flood peaks) are the primary interest, subsurface flows are considered to be insignificant. Subsurface flows travel slower than overland flow.

While groundwater and subsurface water may be ignored for runoff estimates, their detrimental effect upon highway structural section stability cannot be overstated. See Chapter 840, Subsurface Drainage.

816.4 Detention and Retention

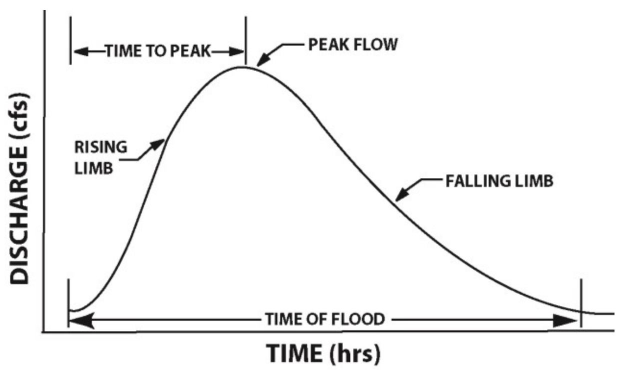
Water which accumulates and ponds in low points or depressions in the soil surface with no possibility for escape as runoff is in retention storage. Where water is moving over the land it is in detention storage. Detained water, as opposed to retained water, contributes to runoff.

816.5 Flood Hydrograph and Flood Volume

In response to a rainstorm the quantity of water flowing in a stream increases. The water level rises and may continue to do so after rainfall ceases. The response of an affected stream, during and after a storm event, can be pictured by plotting discharge against time to produce a flood hydrograph. The principal elements of a typical flood hydrograph are shown in Figure 816.5.

Figure 816.5

Typical Flood Hydrograph



Flood volume is the area under the flood hydrograph. Although flood volume is not considered in the design for all highway drainage facilities, it is an essential design parameter when storage must be evaluated.

Comprehensive guidance on flood hydrographs and methods to estimate the hydrograph may be found in Chapters 6, 7 and 8 of HDS No. 2, Hydrology, and in Chapters 4, 5, 6, 7 and 8 of the user guide for HEC-HMS. See: http://www.hec.usace.army.mil/software/hec-hms/documentation/HEC-HMS_Technical%20Reference%20Manual_(CPD-74B).pdf

See Index 819.4 for a general discussion of hydrograph methods.

816.6 Time of Concentration (Tc) and Travel Time (Tt)

Time of concentration is defined as the time required for storm runoff to travel from the hydraulically most remote point of the drainage basin to the point of interest.

An assumption made in some of the hydrologic methods for estimating peak discharge, such as the Rational and NRCS Methods (Index 819.2), is that maximum flow results when rainfall of uniform intensity falls over the entire watershed area and the duration of that rainfall is equal to the time of concentration. Time of concentration (T_C) is typically the cumulative sum of three travel times, including:

- Sheet flow
- Shallow concentrated flow
- Channel flow

For all-paved watersheds (e.g., parking lots, roadway travel lanes and shoulders, etc.) it is not necessary to calculate a separate shallow concentrated flow travel time segment. Such flows will typically transition directly from sheet flow to channel flow or be intercepted at inlets with either no, or inconsequential lengths of, shallow concentrated flow.

In many cases a minimum time of concentration will have to be assumed as extremely short travel times will lead to calculated rainfall intensities that are overly conservative for design purposes. For all-paved areas, slopes steeper than 10H:1V, or where there is a limited opportunity for surface storage, a minimum T_C of 5 minutes should be assumed. For rural or undeveloped areas, it is recommended that a minimum T_C of 10 minutes be used for most situations.

Designers should be aware that maximum runoff estimates are not always obtained using rainfall intensities determined by the time of concentration for the total area. Peak runoff estimates may be obtained by applying higher rainfall intensities from storms of short duration over a portion of the watershed.

(1) *Sheet flow travel time.* Sheet flow is flow of uniform depth over plane surfaces and usually occurs for some distance after rain falls on the ground. The maximum flow depth is usually less than 0.8 inches – 1.2 inches. For unpaved areas, sheet flow normally exists for a distance less than 80 feet – 100 feet. An upper limit of 300 feet is recommended for paved areas.

A common method to estimate the travel time of sheet flow is based on kinematic wave theory and uses the Kinematic Wave Equation:

$$T_t = \frac{0.93 L^{3/5} n^{3/5}}{i^{2/5} S^{3/10}}$$

where

Tt = Travel time in minutes.

L = Length of flow path in feet.

S = Slope of flow in feet per feet.

n = Manning's roughness coefficient for sheet flow (see Table 816.6A).

i = Design storm rainfall intensity in inches per hour.

If T_t is used (as part of T_C) to determine the intensity of the design storm from the IDF curves, application of the Kinematic Wave Equation becomes an iterative process: an assumed value of T_t is used to determine i from the IDF curve; then the equation is used to calculate a new value of T_t which in turn yields an updated i. The process is repeated until the calculated T_t is the same in two successive iterations.

To eliminate the iterations, use the following simplified form of the Manning's kinematic solution:

$$T_t = \frac{0.42L^{4/5}n^{4/5}}{P_2^{1/2}s^{2/5}}$$

where P_2 is the 2-year, 24-hour rainfall depth in inches (ref. NOAA Atlas 14, http://hdsc.nws.noaa.gov/hdsc/pfds/).

The use of flow length alone as a limiting factor for the Kinematic wave equation can lead to circumstances where the underlying assumptions are no longer valid. Over prediction of travel time can occur for conditions with significant amounts of depression storage, where there is a high Manning's n-value or for flat slopes. One study suggests that the upper limit of applicability of the Kinematic wave equation is a function of flow length, slope and Manning's roughness coefficient. This study used both field and laboratory data to propose an upper limit of 100 for the composite parameter of $nL/s^{1/2}$. It is recommended that this criteria be used as a check where the designer has uncertainty on the maximum flow length to which the Kinematic wave equation can be applied to project conditions.

Where sheet flow travel distance cannot be determined, a conservative alternative is to assume shallow concentrated flow conditions without an independent sheet flow travel time conditions. See Index 816.6(2).

Table 816.6A

Roughness Coefficients For Sheet Flow

Surface Description	*n*
Hot Mix Asphalt	0.011-0.016
Concrete	0.012-0.014
Brick with cement mortar	0.014
Cement rubble	0.024
Fallow (no residue)	0.05
Grass	
Short grass prairie	0.15
Dense grass	0.24
Bermuda Grass	0.41
Woods[(1)]	
Light underbrush	0.40
Dense underbrush	0.80

(1) Woods cover is considered up to a height of 1 inch, which is the maximum depth obstructing sheet flow.

(2) *Shallow concentrated flow travel time.* After short distances, sheet flow tends to concentrate in rills and gullies, or the depth exceeds the range where use of the Kinematic wave equation applies. At that point the flow becomes defined as shallow concentrated flow. The Upland Method is commonly used when calculating flow velocity for shallow concentrated flow. This method may also be used to calculate the total travel time for both the sheet flow and the shallow concentrated flow segments under certain conditions (e.g., where use of the Kinematic wave equation to predict sheet flow travel time is questionable, or where the designer cannot reasonably identify the point where sheet flow transitions to shallow concentrated flow).

Average velocities for the Upland Method can be taken directly from Figure 816.6 (Source NRCS, National Engineering Handbook part 650) or may be calculated from the following equation:

$$V = (3.28)\ kS^{1/2}$$

Where S is the slope in percent and k is an intercept coefficient depending on land cover as shown in Table 816.6B. It is assumed that the depth range is 0.1 to 0.2 feet, except for grassed waterways, where the depth range is 0.1 to 0.4 feet,

Table 816.6B

Intercept Coefficients for Shallow Concentrated Flow

Land cover/Flow regime	k
Forest with heavy ground litter; hay meadow	0.076
Trash fallow or minimum tillage cultivation; contour or strip cropped; woodland	0.152
Short grass pasture	0.213
Cultivated straight row	0.274
Nearly bare and untilled alluvial fans	0.305
Grassed waterway	0.457
Pavement and small upland gullies	0.620

The travel time can be calculated from:

$$T_t = \frac{L}{60\ V}$$

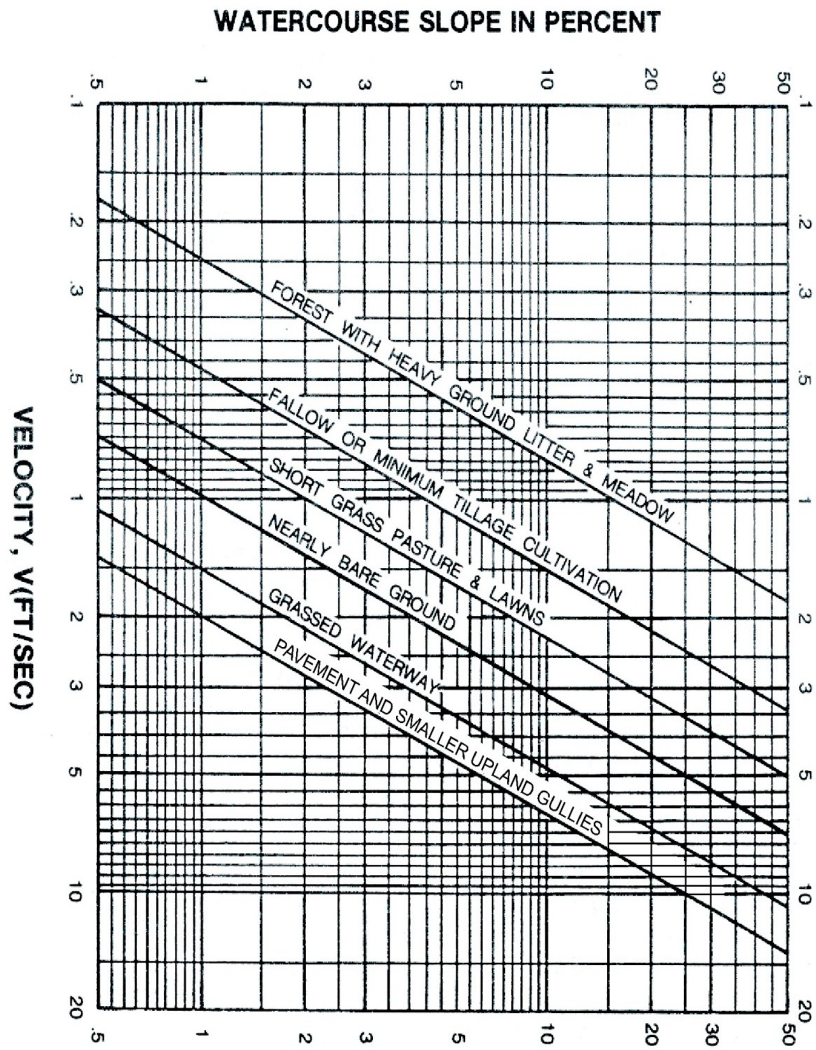
where Tt is the travel time in minutes, L the length in feet, and V the flow velocity in feet per second.

(3) *Channel flow travel time.* When the channel characteristics and geometry are known the preferred method of estimating channel flow time is to divide the channel length by the channel velocity obtained by using the Manning's equation, assuming bankfull conditions. See Index 866.3(4), for further discussion of Manning's equation.

Appropriate values for "n", the coefficient of roughness in the Manning's equation, may be found in most hydrology or hydraulics texts and reference books. Table 866.3A gives some "n" values for lined and unlined channels, gutters, and medians. Procedures for selecting an appropriate hydraulic roughness coefficient may be found in the FHWA report, "Guide for Selecting Manning's Roughness Coefficient for Natural Channels and Flood Plains." See http://www.fhwa.dot.gov/bridge/wsp2339.pdf. Generally, the channel roughness factor will be much lower than the values for overland flow with similar surface appearance.

Figure 816.6

Velocities for Upland Method of Estimating Travel Time for Shallow Concentrated Flow

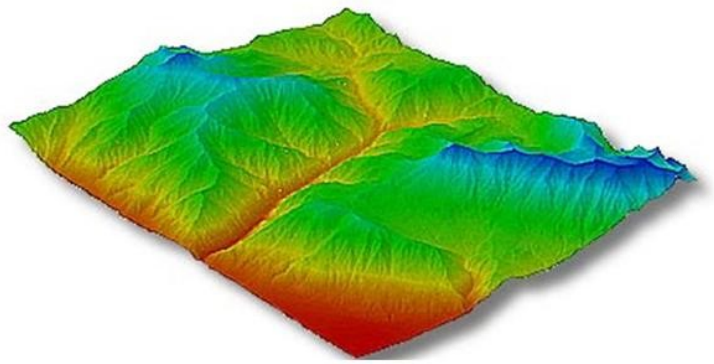


(4) *Culvert or Storm Drain Flow*. Flow velocities in a short culvert are generally higher than they would be in the same length of natural channel and comparable to those in a lined channel. In most cases, including short runs of culvert in the channel, flow time calculation will not materially affect the overall time of concentration (T_C). When it is appropriate to separate flow time calculations, such as for urban storm drains, Manning's equation may be used to obtain flow velocities within pipes.

The TR-55 library of equations for sheet flow, shallow concentrated flow and open channel flow is incorporated into the Watershed Modeling System (WMS) for Time of Concentration Calculations using Triangulated Irregular Networks (TINs) and Digital Elevation Maps (DEMs). See Figure 816.7.

Figure 816.7

Digital Elevation Map (DEM)



Topic 817 – Flood Magnitude

817.1 General

The determination of flood magnitude from either measurements made during a flood or after peak flow has subsided requires knowledge of open-channel hydraulics and flood water behavior. There are USGS Publications and other technical references available which outline the procedures for measuring flood flow. However, it is only through experience that accurate measurements can be obtained and/or correctly interpreted.

817.2 Measurements

(1) *Direct.* Direct flood flow measurements are those made during flood stage. The area and average velocity can be approximated and the estimated discharge can be calculated, from measurements of flow depth and velocity made simultaneously at a number of points in a cross section.

Discharges calculated from continuous records of stage gaging stations are the primary basis for estimating the recurrence interval or frequency of floods. See Figure 817.2.

(2) *Indirect*. Indirect flood flow measurements are those made after the flood subsides. From channel geometry measurements and high water marks the magnitude of a flood can be calculated using basic open channel hydraulic equations given in Chapter 860. This method of determining flood discharges for given events is a valuable tool to the highway engineer possessing a thorough knowledge and understanding of the techniques involved. See Figure 817.3.

Figure 817.2

Gaging Station

Smith River Stage Gaging Station at Dr. Fine Bridge

Figure 817.3

High Water Marks

Topic 818 – Flood Probability And Frequency

818.1 General

The estimation of peak discharges of various recurrence intervals is the most common and important problem encountered in highway engineering hydrology. Since the hydrology for the sizing of highway drainage facilities is concerned with future events, the time and magnitude of which cannot be precisely forecast, the highway engineer must resort to probability statistics to define the design discharge.

Modern hydrologists tend to define floods in terms of probability, as expressed in percentage rather than in terms of return period (recurrence interval). Return period, the "N-year flood", and probability (p) are reciprocals, that is, p = 1/N. Therefore, a flood having a 50-year return frequency (Q_{50}) is now commonly expressed as a flood with the probability of recurrence of 0.02 (2 percent chance of being exceeded) in any given year.

There are certain other terminologies which are frequently used and understood by highway engineers but which might have a slight variation in meaning to other engineering branches. For convenience and example, the following definition of terms have been excerpted from Topic 806, Definition of Drainage Terms.

(1) *Base Flood.* "The flood, tide, or a combination of the two having a 1 percent chance of being exceeded in any given year". The "base flood" is used as the standard flood by FEMA and has been adopted by many agencies for flood hazard analysis to comply with regulatory requirements. See Topic 804, Floodplain Encroachments.

(2) *Overtopping Flood.* "The flood described by the probability of exceedance and water surface elevation at which flow occurs over the highway, over the watershed divide, or through structure(s) provided for emergency relief". The "overtopping flood" is of particular interest to highway drainage engineers because it may be the threshold where the relatively low profile of the highway acts as a flood relief mechanism for the purpose of minimizing upstream backwater damages. See Figure 818.1. On Interstate highways, CFR 650 states “The design flood for encroachments by through lanes of Interstate highways shall not be less than the flood with a 2-percent chance of being exceeded in any given year. No minimum design flood is specified for Interstate highway ramps and frontage roads or for other highways.”

(3) *Design Flood.* "The peak discharge (when appropriate, the volume, stage, or wave crest elevation) of the flood associated with the probability of exceedance selected for the design of a highway encroachment". Except for the rare situation where the risks associated with a low water crossing are acceptable, the highway will not be inundated by the "design flood".

(4) *Maximum Historical Flood.* "The maximum flood that has been recorded or experienced at any particular highway location". This information is very desirable and where available is an indication that the flood of this magnitude may be repeated at the project site. Hydrologic analysis may suggest that the probability for recurrence of the "maximum historical flood" is very small, less than 1 percent. Nevertheless consideration should be given to sizing drainage structures to convey the "maximum historical flood". See Figure 818.2.

(5) *Probable Maximum Flood.* "The flood discharge that may be expected from the most severe combination of critical meteorological and hydrological conditions that are reasonably possible in the region". The "probable maximum flood" is generally not applicable to highway projects. The possibility of a flood of such rare magnitude, as used by the Corps of Engineers, is applicable to projects such as major dams, when consideration is to be given to virtually complete security from potential floods.

Figure 818.1

Overtopping Flood

Figure 818.2

Maximum Historic Flood

818.2 Establishing Design Flood Frequency

There are two recognized alternatives to establishing an appropriate highway drainage design frequency. That is, by policy or by economic analysis. Both alternatives have merit and may be applied exclusively or jointly depending upon general conditions or specific constraints.

Application of traditional predetermined design flood frequencies implies that an acceptable level of risk was considered in establishing the design standard. Modern design concepts, on the other hand, recommend that a range of peak flows be considered and that the design flood be established which best satisfies the specific site conditions and associated risks. A preliminary evaluation of the inherent flood-related risks to upstream and downstream properties, the highway facility, and to the traveling public should be made. This evaluation will indicate whether a predetermined design flood frequency is applicable or additional study is warranted.

Highway classification is one of the most important factors, but not the sole factor, in establishing an appropriate design flood frequency. Due consideration should be given to all the other factors listed under Index 801.5. If the analysis is correct, the highway drainage system will occasionally be overtaxed. The alternative of accommodating the worst possible event that could happen is usually so costly that it may not be justified.

Highway engineers should understand that the option to select a predetermined design flood frequency is generally only applicable to new highway locations. Because of existing constraints, the freedom to select a prescribed design flood frequency may not exist for projects involving replacement of existing facilities. Caltrans policy relative to up-grading of existing drainage facilities may be found in Index 803.3.

Although the procedures and methodology presented in HEC 17, Design of Encroachments on Flood Plains Using Risk Analysis, are not fully endorsed by Caltrans, the circular is an available source of information on the theory of "least total expected cost (LTEC) design". Highway engineers are cautioned about applying LTEC methodology and procedures to ordinary drainage design problems. The Headquarters Hydraulics Engineer in the Division of Design should be consulted before committing to design by the LTEC method since its use can only be justified and recommended under extra-ordinary circumstances.

818.3 Stationarity and Climate Variability

In Index 818.1, the assumption behind flood probability and frequency analysis is that climate is stationary. Stationarity assumes that hydrology varies within an unchanging envelope of natural variability, so that the past accurately represents the future. It has been a basic assumption used for many years in the planning and design of bridges and culverts and continues to represent the current state of practice that serves the engineering community well.

Climate change as well as better understanding of climate variability have presented a challenge to the validity of this assumption.

Today, there is growing recognition that, despite its successful application in the past, the assumption of stationarity may not accurately represent the future. However, until a multi-disciplinary consensus is reached on future trends that can be expected, stationarity will continue to be utilized with current procedures.

To minimize uncertainty, designers should continue to utilize existing hydrologic tools with the most current datasets available for rainfall and runoff. Observed trends can then be quantified and placed in the context of the uncertainty associated with the frequency estimates themselves.

(1) *Nonstationarity and Climate Variability.* Changes in land use, changing groundwater levels, and urbanization are examples of nonstationarity within a watershed that can affect hydrologic response. The Intergovernmental Panel on Climate Change (IPCC) has stated that "Climate change challenges the traditional assumption that past hydrological experience provides a good guide to future conditions". Although the assumption of stationarity is being challenged, there is no consensus within the scientific or engineering community on a viable replacement.

Topic 819 – Estimating Design Discharge

819.1 Introduction

Before highway drainage facilities can be hydraulically designed, the quantity of run-off (design Q) that they may reasonably be expected to convey must be established. The estimation of peak discharge for various recurrence intervals is therefore the most important, and often the most difficult, task facing the highway engineer. Refer to Table 819.5A for a summary of methods for estimating design discharge.

In Topic 819, various design recommendations are given for both general and region-specific areas of California.

819.2 Empirical Methods

Because the movement of water is so complex, numerous empirical methods have been used in hydrology. Empirical methods in hydrology have great usefulness to the highway engineer. When correctly applied by engineers knowledgeable in the method being used and its idiosyncrasies, peak discharge estimates can be obtained which are functionally acceptable for the design of highway drainage structures and other features. Some of the more commonly used empirical methods for estimating runoff are as follows.

(1) *Rational Methods*. Undoubtedly, the most popular and most often misused empirical hydrology method is the Rational Formula:

Q = CiA

Q = Design discharge in cubic feet per second.

C = Coefficient of runoff.

i = Average rainfall intensity in inches per hour for the selected frequency and for a duration equal to the time of concentration. See http://hdsc.nws.noaa.gov/hdsc/pfds/

A = Drainage area in acres.

Rational methods are simple to use, and it is this simplicity that has made them so popular among highway drainage design engineers. Design discharge, as computed by these methods, has the same probability of occurrence (design frequency) as the frequency of the rainfall used. Refer to Topic 818 for further information on flood probability and frequency of recurrence.

An assumption that limits applicability is that the rainfall is of equal intensity over the entire watershed. Because of this, Rational Methods should be used only for estimating runoff from small simple watershed areas, preferably no larger than 320 acres. Even where the watershed area is relatively small but complicated by a mainstream fed by one or more significant tributaries, Rational Methods should be applied separately to each tributary stream and the tributary flows then routed down the main channel. Flow routing can best be accomplished through the use of hydrographs discussed in Index 819.4. Since Rational Methods give results that are in terms of instantaneous peak discharge and provide little information relative to runoff rate with respect to time, synthetic hydrographs should be developed for routing significant tributary inflows. Several relatively simple methods have been established for developing hydrographs, such as transposing a hydrograph from another hydrologically homogeneous watershed. The stream hydraulic method, and upland method are described in HDS No. 2. These, and other methods, are adequate for use with Rational Methods for estimating peak discharge and will provide results that are acceptable to form the basis for design of highway drainage facilities.

It is clearly evident upon examination of the assumptions and parameters which form the basis of the equation that much care and judgment must be applied with the use of Rational Methods to obtain reasonable results.

- The runoff coefficient "C" in the equation represents the percent of water which will run off the ground surface during the storm. The remaining amount of precipitation is lost to infiltration, transpiration, evaporation and depression storage. "C" is a volumetric coefficient that relates the peak discharge to the “theoretical peak” or 100 percent runoff, occurring when runoff matches the net rain rate. Hence "C" is also a function of infiltration and other hydrologic abstractions.

  Values of "C" may be determined for un-developed areas from Figure 819.2A by considering the four characteristics of: relief, soil infiltration, vegetal cover, and surface storage.

The designer must use judgment to select the appropriate "C" value within the range. Generally, larger areas with permeable soils, flat slopes and dense vegetation should have the lowest "C" values. Smaller areas with dense soils, moderate to steep slopes, and sparse vegetation should be assigned the highest "C" values.

Some typical values of "C" for developed areas are given in Table 819.2B. Should the basin contain varying amounts of different cover, a weighted runoff coefficient for the entire basin can be determined as:

$$C = \frac{C_1A_1 + C_2A_2 + \dots}{A_1 + A_2 + \dots}$$

- To properly satisfy the assumption that the entire drainage area contributes to the flow; the rainfall intensity, (i) in the equation expressed in inches per hour, requires that the storm duration and the time of concentration (t_C) be equal. Therefore, the first step in estimating (i) is to estimate (t_C). Methods for determining time of concentration are discussed under Index 816.6.
- Once the time of concentration, (t_C), is estimated, the rainfall intensity, (i), corresponding to a storm of equal duration, may be obtained from available sources such as intensity-duration-frequency (IDF) curves. For IDF curve generating software, see http://hdsc.nws.noaa.gov/hdsc/pfds/.

The runoff coefficients given in Figure 819.2A and Table 819.2B are applicable for storms of up to 5 or 10 year frequencies. Less frequent, higher intensity storms usually require modification of the coefficient because infiltration, detention, and other losses have a proportionally smaller effect on the total runoff volume. The adjustment of the rational method for use with major storms can be made by multiplying the coefficient by a frequency factor, C(f). Values of C(f) are given below. Under no circumstances should the product of C(f) times C exceed 1.0.

Frequency (yrs)	C(f)
25	1.1
50	1.2
100	1.25

(2) *Regional Analysis Methods.* Regional analysis methods utilize records for streams or drainage areas in the vicinity of the stream under consideration which would have similar characteristics to develop peak discharge estimates. These methods provide techniques for estimating annual peak stream discharge at any site, gaged or ungaged, for probability of recurrence from 50 percent (2years) to 1 percent (100 years). Application of these methods is convenient, but the procedure is subject to some limitations.

Regional Flood - Frequency equations developed by the U.S. Geological Survey for use in California are given in Table 819.2C and Table 819.7A. These equations are based on regional regression analysis of data from stream gauging stations. The equations in Table 819.2C were derived from data gathered and analyzed through 2006, while the regions covered by Table 819.7A are reflective of a 1994 study of the Southwestern U.S, which has been supplemented by a more recent 2007 Study of California Desert Region Hydrology. Information on use and development of this method may be found in "Methods for Determining Magnitude and Frequency of Floods in California Based on Data through Water Year 2006" by the U.S. Department of the Interior, Geological Survey.

The Regional Flood-Frequency equations are applicable only to sites within the flood-frequency regions for which they were derived and on streams with virtually natural flows. The equations are not directly applicable to streams in urban areas affected substantially by urban development. In urban areas the equations may be used to estimate peak discharge values under natural conditions and then by use of the techniques described in the publication or HDS No. 2, adjust the discharge values to compensate for urbanization. A method for directly estimating design discharges for some gaged and ungaged streams is also provided in HDS No. 2. The method is applicable to streams on or nearby those for which study data are available.

(3) *Flood Frequency Analysis*

(a) If there are two gaged sites with similar watershed characteristics but one has a short record and the other has a longer record of peak flows, a two-station comparison analysis can be conducted to extend the equivalent length of record at the shorter gaged site.

(b) Flood-frequency relations at sites near gaged sites on the same stream (or in a similar watershed) can be estimated using a ratio of drainage area for the ungaged and gaged sites.

(c) At a gaged site, weighted estimates of peak discharges based on the station flood-frequency relation and the regional regression equations are considered the best estimates of flood frequency and are used to reduce the time-sampling error that may occur in a station flood-frequency estimate.

Figure 819.2A

Runoff Coefficients for Undeveloped Areas Watershed Types

	Extreme	High	Normal	Low
Relief	.28 -.35 Steep, rugged terrain with average slopes above 30%	.20 -.28 Hilly, with average slopes of 10 to 30%	.14 -.20 Rolling, with average slopes of 5 to 10%	.08 -.14 Relatively flat land, with average slopes of 0 to 5%
Soil Infiltration	.12 -.16 No effective soil cover, either rock or thin soil mantle of negligible infiltration capacity	.08 -.12 Slow to take up water, clay or shallow loam soils of low infiltration capacity, imperfectly or poorly drained	.06 -.08 Normal; well drained light or medium textured soils, sandy loams, silt and silt loams	.04 -.06 High; deep sand or other soil that takes up water readily, very light well drained soils
Vegetal Cover	.12 -.16 No effective plant cover, bare or very sparse cover	.08 -.12 Poor to fair; clean cultivation crops, or poor natural cover, less than 20% of drainage area over good cover	.06 -.08 Fair to good; about 50% of area in good grassland or woodland, not more than 50% of area in cultivated crops	.04 -.06 Good to excellent; about 90% of drainage area in good grassland, woodland or equivalent cover
Surface Storage	.10 -.12 Negligible surface depression few and shallow; drainageways steep and small, no marshes	.08 -.10 Low; well defined system of small drainageways; no ponds or marshes	.06 -.08 Normal; considerable surface depression storage; lakes and pond marshes	.04 -.06 High; surface storage, high; drainage system not sharply defined; large floodplain storage or large number of ponds or marshes
Given	An undeveloped watershed consisting of; 1) rolling terrain with average slopes of 5%, 2) clay type soils, 3) good grassland area, and 4) normal surface depressions.		Solution: Relief 0.14 Soil Infiltration 0.08 Vegetal Cover 0.04 Surface Storage 0.06 C = 0.32	
Find	The runoff coefficient, C, for the above watershed.			

Table 819.2B

Run off Coefficients for Developed Areas[1]

Type of Drainage Area	Runoff Coefficient
Business:	
Downtown areas	0.70 - 0.95
Neighborhood areas	0.50 - 0.70
Residential:	
Single-family areas	0.30 - 0.50
Multi-units, detached	0.40 - 0.60
Multi-units, attached	0.60 - 0.75
Suburban	0.25 - 0.40
Apartment dwelling areas	0.50 - 0.70
Industrial:	
Light areas	0.50 - 0.80
Heavy areas	0.60 - 0.90
Parks, cemeteries:	0.10 - 0.25
Playgrounds:	0.20 - 0.40
Railroad yard areas:	0.20 - 0.40
Unimproved areas:	0.10 - 0.30
Lawns:	
Sandy soil, flat, 2%	0.05 - 0.10
Sandy soil, average, 2-7%	0.10 - 0.15
Sandy soil, steep, 7%	0.15 - 0.20
Heavy soil, flat, 2%	0.13 - 0.17
Heavy soil, average, 2-7%	0.18 - 0.22
Heavy soil, steep, 7%	0.25 - 0.35
Streets:	
Asphaltic	0.70 - 0.95
Concrete	0.80 - 0.95
Brick	0.70 - 0.85
Drives and walks	0.75 - 0.85
Roofs:	0.75 - 0.95

NOTES:

(1) From HDS No. 2.

Table 819.2C

Regional Flood-Frequency Equations

NORTH COAST (REGION 1)	LAHONTAN (REGION 2)	SIERRA NEVADA (REGION 3)
$Q_2 = 1.82A^{0.904}P^{0.983}$	$Q_2 = 0.0865A^{0.736}P^{1.59}$	$Q_2 = 2.43A^{0.924}E^{-0.646}P^{2.06}$
$Q_5 = 8.11A^{0.887}P^{0.772}$	$Q_5 = 0.182A^{0.733}P^{1.58}$	$Q_5 = 11.6A^{0.907}E^{-0.566}P^{1.70}$
$Q_{10} = 14.8A^{0.880}P^{0.696}$	$Q_{10} = 0.260A^{0.734}P^{1.59}$	$Q_{10} = 17.2A^{0.896}E^{-0.486}P^{1.54}$
$Q_{25} = 26.0A^{0.874}P^{0.628}$	$Q_{25} = 0.394A^{0.733}P^{1.58}$	$Q_{25} = 20.7A^{0.885}E^{-0.386}P^{1.39}$
$Q_{50} = 36.3A^{0.870}P^{0.589}$	$Q_{50} = 0.532A^{0.733}P^{1.58}$	$Q_{50} = 21.1A^{0.879}E^{-0.316}P^{1.31}$
$Q_{100} = 48.5A^{0.866}P^{0.556}$	$Q_{100} = 0.713A^{0.731}P^{1.56}$	$Q_{100} = 20.6A^{0.874}E^{-0.250}P^{1.24}$

CENTRAL COAST (REGION 4)	SOUTH COAST (REGION 5)
$Q_2 = 0.00459A^{0.856}P^{2.58}$	$Q_2 = 3.60A^{0.672}P^{0.753}$
$Q_5 = 0.0984A^{0.852}P^{1.97}$	$Q_5 = 7.43A^{0.739}P^{0.872}$
$Q_{10} = 0.460A^{0.846}P^{1.66}$	$Q_{10} = 6.56A^{0.783}P^{1.07}$
$Q_{25} = 2.13A^{0.842}P^{1.34}$	$Q_{25} = 4.71A^{0.832}P^{1.32}$
$Q_{50} = 5.32A^{0.840}P^{1.15}$	$Q_{50} = 3.84A^{0.864}P^{1.47}$
$Q_{100} = 11.0A^{0.840}P^{0.994}$	$Q_{100} = 3.28A^{0.891}P^{1.59}$

Q = Peak discharge in CFS, subscript indicates recurrence interval, in years
A = Drainage area, in square miles
P = Mean annual precipitation, in inches (Use link to Table 2) http://pubs.usgs.gov/sir/2012/5113/
E = Mean basin elevation, in feet

Region	Drainage Area (A), mi^2	Mean Annual Precipitation (P), in.	Mean Basin Elevation (E), ft.
North Coast	0.04 – 3200	20 – 125	-
Lahontan [(1)]	0.45 – 1500	13 – 85	-
Sierra Nevada	0.07 – 2000	15 – 100	90 – 11,000
Central Coast	0.11 – 4600	7 – 46	-
South Coast	0.04 – 850	10 – 45	-
Desert [(2)]	N/A	N/A	-

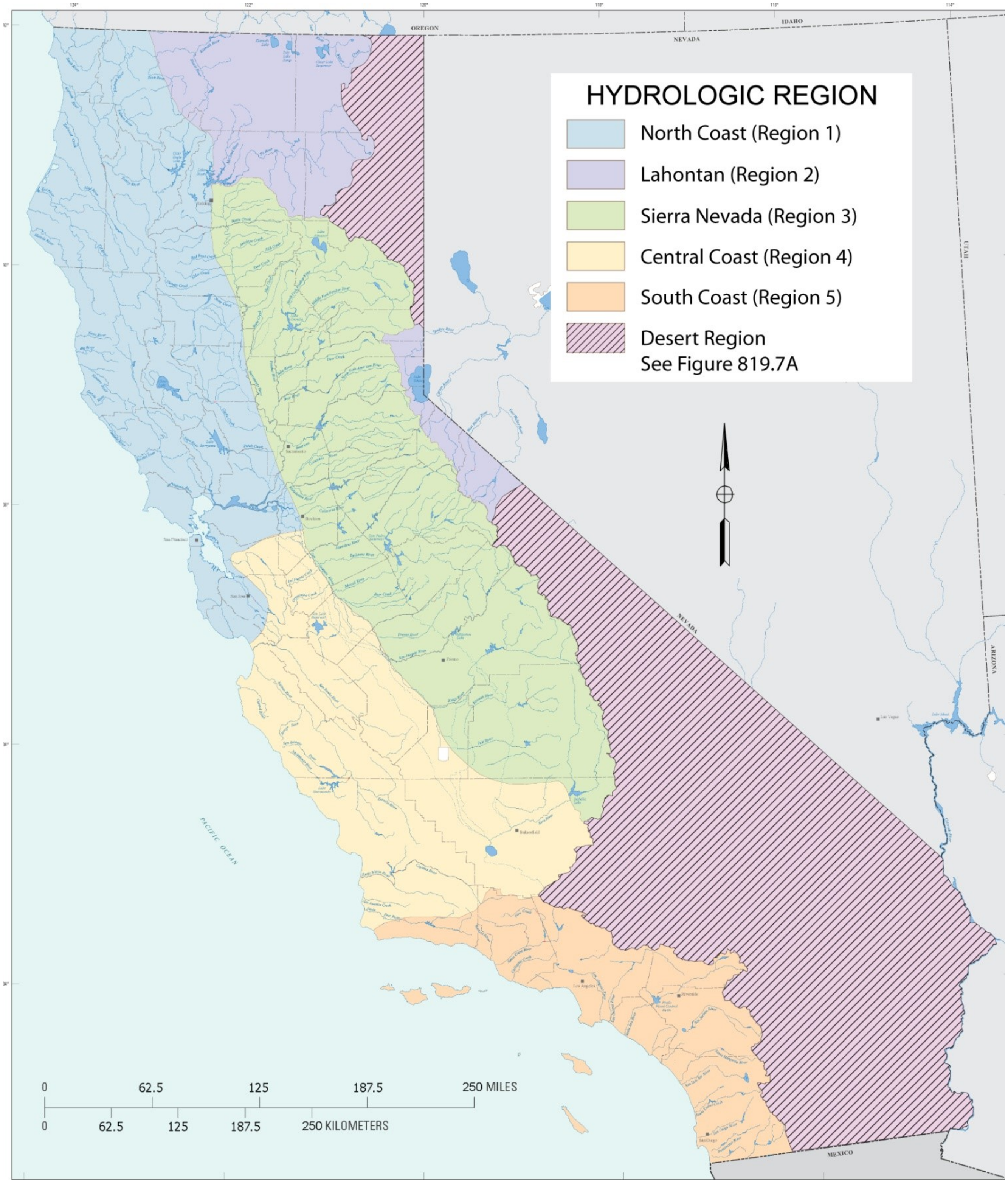
NOTES:

(1) See Index 819.7 for hydrologic procedures for those portions of the Northeast Region classified as desert.

(2) USGS equations not recommended. See Index 819.7.

Figure 819.2C

Regional Flood-Frequency Regions



(d) The flood-frequency flows and the maximum peak discharges at several stations in a region should be used whenever possible for comparison with the peak discharge estimated at an ungaged site using a rainfall-runoff approach or regional regression equation. The watershed characteristics at the ungaged and gaged sites should be similar.

(4) *National Resources Conservation Service (NRCS) Methods.* The Soil Conservation Service's SCS (former title) National Engineering Handbook, 1972, and their 1975, "Urban Hydrology for Small Watersheds", Technical Release 55 (TR-55), present a graphical method for estimating peak discharge. Most NRCS equations and curves provide results in terms of inches of runoff for unit hydrograph development and are not applicable to the estimation of a peak design discharge unless the design hydrograph is first developed in accordance with prescribed NRCS procedures. NRCS methods and procedures are applicable to drainage areas less than 3 square miles (approx. 2,000 acres) and result in a design hydrograph and design discharge that are functionally acceptable to form the basis for the design of highway drainage facilities.

819.3 Statistical Methods

Statistical methods of predicting stream discharge utilize numerical data to describe the process. Statistical methods, in general, do not require as much subjective judgment to apply as the previously described deterministic methods. They are usually well documented mathematical procedures which are applied to measured or observed data. The accuracy of statistical methods can also be measured quantitatively. However, to assure that statistical method results are valid, the method and procedures used should be verified by an experienced engineer with a thorough knowledge of engineering statistics.

Analysis of gaged data permits an estimate of the peak discharge in terms of its probability or frequency of recurrence at a given site. This is done by statistical methods provided sufficient data are available at the site to permit a meaningful statistical analysis to be made. Water Resources Council Bulletin 17B, 1981, suggests at least 10 years of record are necessary to warrant a statistical analysis. The techniques of inferential statistics, the branch of statistics dealing with the inference of population characteristics, are described in HDS No. 2.

Before data on the specific characteristics to be examined can be properly analyzed, it must be arranged in a systematic manner. Several computer programs are available which may be used to systematically arrange data and perform the statistical computations.

Some common types of data groupings are as follows:

- Magnitude
- Time of Occurrence
- Geographic Location

Several standard frequency distributions have been studied extensively in the statistical analysis of hydrologic data. Those which have been found to be most useful are:

(1) *Log-Pearson Type III Distribution.* The popularity of the Log-Pearson III distribution is simply based on the fact that it very often fits the available data quite well, and it is flexible enough to be used with a wide variety of distributions. Because of this flexibility, the U.S. Water Resources Council recommends its use by all U.S. Government agencies as the standard distribution for flood frequency studies.

The three parameters necessary to describe the Log-Pearson III distribution are:

- Mean flow
- Standard deviation
- Coefficient of skew

Log-Pearson III distributions are usually plotted on log-normal probability graph paper for convenience even though the plotted frequency distribution may not be a straight line.

It should be noted Log-Pearson III analysis is not typically appropriate for desert regions where flood-frequency analysis is complicated due to short annual peak-flow records (usually less than 20 years) and numerous zero flows and (or) low outliers for many stream gages.

(2) *Log-normal Distribution.* The characteristics of the log-normal distribution are the same as those of the classical normal or Gaussian mathematical distribution except that the flood flow at a specified frequency is replaced with its logarithm and has a positive skew. Positive skew means that the distribution is skewed toward the high flows or extreme values.

(3) *Gumbel Extreme Value Distribution.* The characteristics of the Gumbel extreme value distribution (also known as the double exponential distribution of extreme values) are that the mean flood occurs at the return period of T_r = 2.33 years and that it has a positive skew.

Special probability paper has been developed for plotting log-normal and Gumbel distributions so that sample data, if it is distributed according to prescribed equations, will plot as a straight line.

(4) *L-Moments.* L-moments provide an alternative way of describing frequency distributions to traditional product moments (conventional moments) or maximum likelihood approach. They are less susceptible to the presence of outliers in the data than conventional moments and are well suited for the analysis of data that exhibit significant skewness. See overview of methodology used for NOAA Atlas 14 (Index 4.6.1); http://www.nws.noaa.gov/oh/hdsc/PF_documents/Atlas14_Volume6.pdf

819.4 Hydrograph Methods

Hydrograph methods of estimating design discharge relate runoff rates to time in response to a design storm. When storage must be considered, such as in reservoirs, natural lakes, and detention basins used for drainage or sediment control, the volume of runoff must be known. Since the hydrograph is a plot of flow rate against time, the area under the hydrograph represents volume. If streamflow and precipitation records are available for a particular design site, the development of the design hydrograph is a straight forward procedure. Rainfall records can be readily analyzed to estimate unit durations and the intensity which produces peak flows near the desired design discharge.

It often becomes necessary to develop a hydrograph when watersheds have complex runoff characteristics, such as in urban and desert areas or when storage must be evaluated.

Hydrograph methods apply for watersheds in which the time of concentration is longer than the duration of peak rainfall intensity of the design storm. Precipitation applied to the watershed model is uniform spatially, but varies with time. The hydrograph method accounts for losses (e.g., soil infiltration) and transforms the remaining (excess) rainfall into a runoff hydrograph at the outlet of the watershed. There is no size limitation for watershed area. See HDS No. 2; Figure 2-13, for the relationship of discharge and area and effects of basin characteristics on the flood hydrograph.

Hydrographs are also useful for determining the combined rates of flow for two drainage areas which peak at different times. Hydrographs can also be compounded and lagged to account for complex storms of different duration and varying intensities.

See Index 819.7(1)(d) for a detailed discussion on rainfall-runoff simulation for California's Desert regions. The same four general concepts are applicable elsewhere. Other considerations may include:

- Development of a rainfall hyetograph
- Base flow separation
- Direct runoff hydrograph derivation
- Unit hydrograph derivation; and
- Other synthetic unit hydrographs (e.g., Snyder's or Clark's methods)

Successful application of most hydrograph methods requires the designer to:

- Define the temporal and spatial distribution of the desired design storm.
- Specify appropriate losses within the model to compute the amount of precipitation lost to other processes, such as infiltration that does not run off the watershed.
- Specify appropriate parameters to compute runoff hydrograph resulting from excess (not lost) precipitation.
- If necessary for the application, specify appropriate parameters to compute the lagged and attenuated hydrograph at downstream locations. Basic steps to developing and applying a rainfall-runoff model for predicting the required design flow are illustrated in Figure 819.4A.

Several methods of developing hydrographs are described in HDS No. 2. For basins without data, two of the most widely used methods described in HDS No. 2 for developing synthetic hydrographs are:

- Unit Hydrograph (UH)
- SCS Triangular Hydrograph

Both methods however tend to be somewhat inflexible since storm duration is determined by empirical relations.

For basins with data, HEC-HMS includes the following direct runoff models:

- User specified UH
- Parametric and Synthetic UH
- Snyder's UH
- Clark's UH
- ModClark Model
- Kinematic-wave Model

For more information see; Chapters 4, 5, 6, 7 and 8 of the user guide for HEC-HMS. See: http://www.hec.usace.army.mil/software/hec-hms/documentation/HEC-HMS_Technical%20Reference%20Manual_(CPD-74B).pdf

819.5 Transfer of Data

Often the highway engineer is confronted with the problem where stream flow and rainfall data are not available for a particular site but may exist at points upstream or in an adjacent or nearby watersheds.

Figure 819.4

Basic Steps to Developing and Applying a Rainfall-runoff Model for Predicting the Required Design Flow

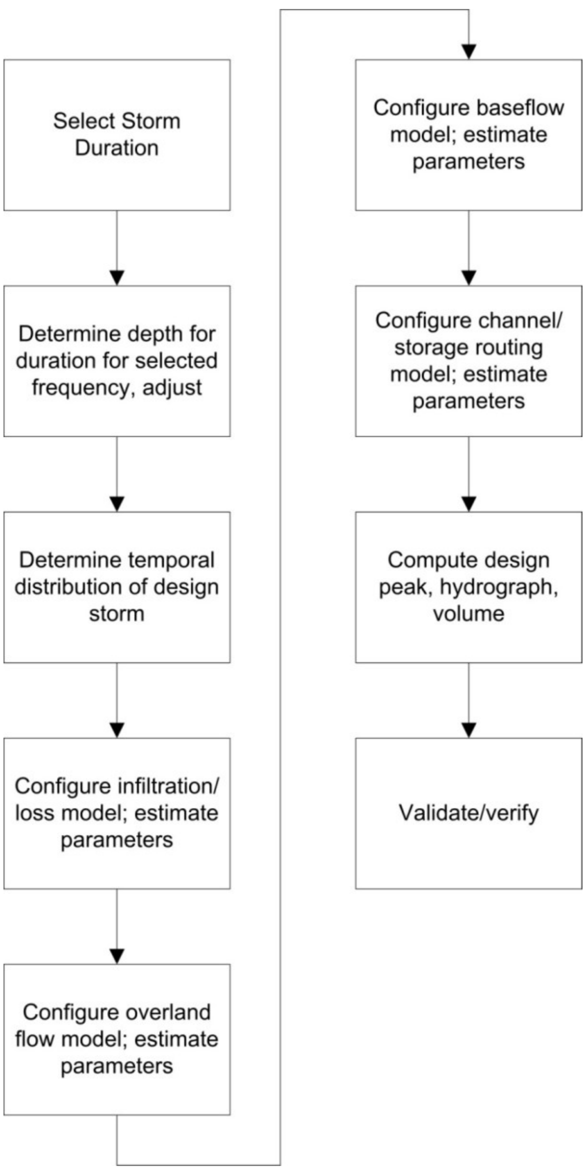


Table 819.5A

Summary of Methods for Estimating Design Discharge

METHOD	ASSUMPTIONS	DATA NEEDS
Rational	• Small catchment (< 320 acres) • Concentration time < 1 hour • Storm duration >or = concentration time • Rainfall uniformly distributed in time and space • Runoff is primarily overland flow • Negligible channel storage	Time of Concentration Drainage area Runoff coefficient Rainfall intensity (http://hdsc.nws.noaa.gov/hdsc/pfds/)
USGS Regional Regression Equations: USGS Water-Resources Investigation 77-21* Improved Highway Design Methods for Desert Storms	• Catchment area limit varies by region • Basin not located on floor of Sacramento or San Joaquin Valleys • Peak discharge value for flow under natural conditions unaffected by urban development and little or no regulation by lakes or reservoirs • Ungaged channel	Drainage area Mean annual precipitation Altitude index
NRCS (TR55)	• Small or midsize catchment (< 3 square miles) • Concentration time range from 0.1-10 hour (tabular hydrograph method limit < 2 hour) • Runoff is overland and channel flow • Simplified channel routing • Negligible channel storage	24-hour rainfall Rainfall distribution Runoff curve number Concentration time Drainage area
Unit Hydrograph (Gaged data) Synthetic Unit Hydrograph SCS Unit Hydrograph S-Graph Unit Hydrograph	• Midsize or large catchment (0.20 square miles to 1,000 square miles) • Uniformity of rainfall intensity and duration • Rainfall-runoff relationship is linear • Duration of direct runoff constant for all uniform-intensity storms of same duration, regardless of differences in the total volume of the direct runoff. • Time distribution of direct runoff from a given storm duration is independent of concurrent runoff from preceding storms • Channel-routing techniques used to connect streamflows	Rainfall hyetograph and direct runoff hydrograph for one or more storm events Drainage area and lengths along main channel to point on watershed divide and opposite watershed centroid (Synthetic Unit Hydrograph)
Statistical (gage data) Log-Pearson Type III Bulletin #17B – U.S. Department of the Interior	• Midsized and large catchments with stream gage data • Appropriate station and/or generalized skew coefficient relationship applied • Channel storage	10 or more years of gaged flood records
Basin Transfer of Gage Data	• Similar hydrologic characteristics • Channel storage	Discharge and area for gaged watershed Area for ungaged watershed

*Magnitude and Frequency of Floods in California

(a) If the site is on the same stream and near a gaging station, peak discharges at the gaging station can be adjusted to the site by drainage area ratio and application of some appropriate power to each drainage area. The USGS may be helpful in suggesting appropriate powers to be used for a specific hydrologic region.

(b) If a design hydrograph can be developed at an upstream point in the same watershed, the procedure described in HDS No. 2 can be used to route the design hydrograph to the point of interest.

(c) IDF curve generating software, such as NOAA's Atlas 14, have internal routines that provide interstation interpolation that accounts not only for distance from gauge stations, but other factors, such as elevation. No additional effort is required by the designer to address distance/location effects.

819.6 Hydrologic Software

Most simulation models require a significant amount of input data that must be carefully examined by a competent and experienced user with an understanding of the mathematical nuances of the model and the hydrologic nuances of the particular catchment to assure reliable results.

See Table 808.1 for hydrologic software packages that have been reviewed and deemed compatible with Departmental procedures.

A summary of hydrologic software is listed in Table 808.1. Several of those listed are described below.

Watershed Modeling System (WMS) is a comprehensive environment for hydrologic analysis. It was developed by the Engineering Computer Graphics Laboratory of Brigham Young University in cooperation with the U.S. Army Corps of Engineers Waterways Experiment Station (WES).

WMS merges information obtained from terrain models and GIS with industry standard hydrologic analysis models such as HEC-HMS and TR-55.

Terrain models can obtain geometric attributes such as area, slope and runoff distances. Many display options are provided to aid in modeling and understanding the drainage characteristics of terrain surfaces.

WMS uses three primary data sources for model development:

(1) Geographic Information Systems (GIS) Data

(2) Digital Elevation Models (DEMs) published by the U.S. Geological Survey (USGS) at both 1:24,000 and 1:250,000 for the entire U.S. (the 1:24,000 data coverage is not complete)

(3) Triangulated Irregular Networks (TINs)

Automated basin delineation, slope calculation, and basin characteristics are some of the many features available within USGS StreamStats. See; http://water.usgs.gov/osw/streamstats/.

AutoDesk Civil 3D/Hydraflow uses NRCS, Rational and Modified Rational methods to generate runoff hydrographs, however, HEC-HMS provides more comprehensive modeling options for runoff and channel flow.

Two other hydrologic software models that are commonly used are the Army Corps of Engineers' HEC-HMS and the National Resources Conservation Service's TR-20 Method.

The NOAA Atlas 14 product is the preferred IDF tool for State highway projects. See http://hdsc.nws.noaa.gov/hdsc/pfds/.

819.7 Region-Specific Analysis

(1) Desert Hydrology. Figure 819.7A shows the different desert regions in California, each with distinct hydrological characteristics that will be explained in this section.

(a) Storm Type

Summer Convective Storms – In the southern desert regions (Owens Valley/Mono Lake, Mojave Desert, Sonoran Desert and the Colorado Desert), the dominant storm type is the local thunderstorm, specifically summer convective storms. These storms are characterized by their short duration, over a relatively small area (generally less than 20 mi^2), and intense rainfall, which may result in flash floods. These summer convective storms may occur at any time during the year, but are most common and intense during the summer. General summer storms can also occur over these desert regions, but are rare, and usually occur from mid-August to early October. The rainfall intensity can vary from heavy rainfall to heavy thunderstorms.

General Winter Storm – In the Antelope Valley and Northern Basin and Range regions, the dominant storm type is the general winter storm. These storms are characterized by their long duration, 6 hours to 12 hours or more, and possibly intermittently for 3 days to 5 days over a relatively large area. General winter storms produce the majority of large peaks in the northern desert areas; the majority of the largest peaks discharge greater than or equal to 20 cfs/mi^2 occurred during the winter and fall months in the Owens Valley/Mono Lake and Northern Basin and Range regions. At elevations above 6,000 ft, much of the winter precipitation falls as snow; however, snowfall doesn't play a significant role in flood-producing runoff in the southern desert regions (Colorado Desert, Sonoran Desert, Antelope Valley and Mojave Desert). In the northern desert regions (Owens Valley/Mono Lake and Northern Basin and Range), more floods from snowmelt occur at lower elevations; more than 50 percent of runoff events occurred in spring, most likely snowmelt, but did not produce large floods.

(b) Regional Regression

Newly developed equations for California's Desert regions are shown on Table 819.7A.

While the regression equations for the Northern Basin and Range region provide more accurate results than previous USGS developed equations, there is some uncertainty associated with them. Therefore, the development of a rainfall-runoff model may be preferable for ungaged watersheds in this region.

Figure 819.7A

Desert Regions in California

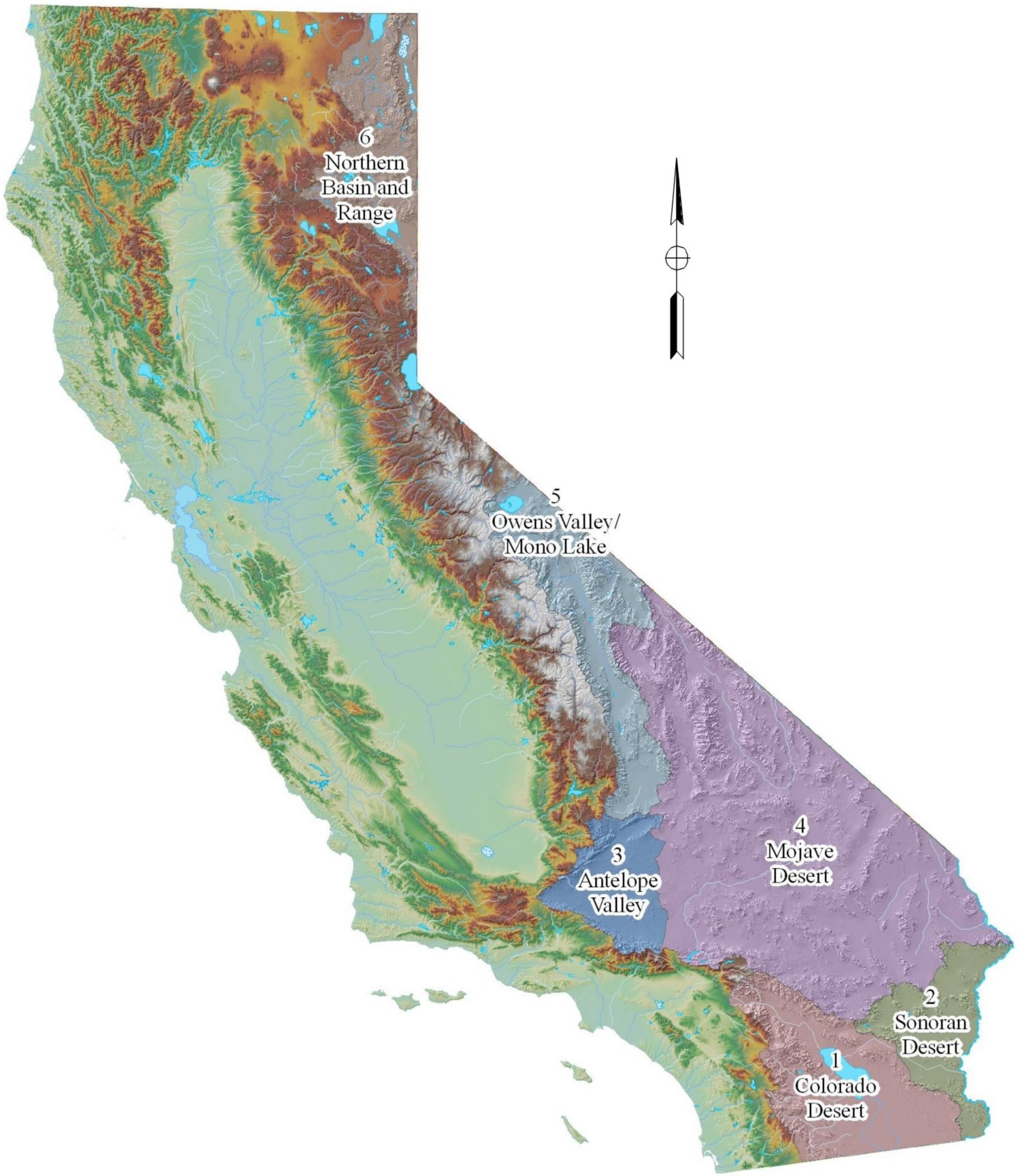


Table 819.7A

Regional Regression Equations for California's Desert Regions

Region(s)	Associated Regression Equations	
Colorado Desert Sonoran Desert Antelope Valley Mojave Desert	$Q_2 = 8.57A^{0.5668}$ $Q_5 = 80.32A^{0.541}$ $Q_{10} = 146.33A^{0.549}$	$Q_{25} = 291.04A^{0.5939}$ $Q_{50} = 397.82A^{0.6189}$ $Q_{100} = 557.31A^{0.6619}$
Owens Valley / Mono Lake	$Q_2 = 0.007A^{1.839}\left[\frac{ELEV}{1000}\right]^{1.485}\left[\frac{LAT-28}{10}\right]^{-0.680}$ $Q_5 = 0.212A^{1.404}\left[\frac{ELEV}{1000}\right]^{0.882}\left[\frac{LAT-28}{10}\right]^{-0.030}$ $Q_{10} = 1.28A^{1.190}\left[\frac{ELEV}{1000}\right]^{0.531}\left[\frac{LAT-28}{10}\right]^{0.525}$ $Q_{25} = 9.70A^{0.962}\left[\frac{ELEV}{1000}\right]^{0.107}\left[\frac{LAT-28}{10}\right]^{1.199}$ $Q_{50} = 34.5A^{0.829}\left[\frac{ELEV}{1000}\right]^{-0.170}\left[\frac{LAT-28}{10}\right]^{1.731}$ $Q_{100} = 111A^{0.707}\left[\frac{ELEV}{1000}\right]^{-0.429}\left[\frac{LAT-28}{10}\right]^{2.241}$	
Northern Basin & Range	$Q_2 = 5.320A^{0.415}\left[\frac{H}{1000}\right]^{0.928}$ $Q_5 = 29.71A^{0.360}\left[\frac{H}{1000}\right]^{0.296}$ $Q_{10} = 85.76A^{0.314}\left[\frac{H}{1000}\right]^{-0.109}$ $Q_{25} = 275.5A^{0.253}\left[\frac{H}{1000}\right]^{-0.555}$ $Q_{50} = 616.9A^{0.281}\left[\frac{H}{1000}\right]^{-0.867}$ $Q_{100} = 1293A^{0.166}\left[\frac{H}{1000}\right]^{-1.154}$	

(c) Rational Method

The recommended upper limit for California's desert regions is 160 acres (0.25 mi^2).

Table 819.7B lists common runoff coefficients for Desert Areas. These coefficients are applicable for storms with 2-year to 10-year return intervals, and should be adjusted for larger, less frequent storms by multiplying the coefficient by an appropriate frequency factor, C(f), as stated in Index 819.2(1) of this manual. The frequency factors, C(f), for 25-year, 50-year and 100-year storms are 1.1, 1.2 and 1.25, respectively. Under no circumstances should the product of C(f) times the runoff coefficient exceed 1.0. It is recommended not to use a value that exceeds 0.95.

(d) Rainfall-Runoff Simulation

A rainfall-runoff simulation approach uses a numerical model to simulate the rainfall-runoff process and generate discharge hydrographs. It has four main components: rainfall; rainfall losses; transformation of effective rainfall; and channel routing.

(1) Rainfall

(a) Design Rainfall Criteria

The selection of an appropriate storm duration depends on a number of factors, including the size of the watershed, the type of rainfall-runoff approach and hydrologic characteristics of the study watershed. Watershed sizes are analyzed below and are applied to California's Desert regions in Table 819.7C.

Drainage Areas $\leq$ 20 mi^2 – Drainage areas less than 20 mi^2 are primarily representative of summer convective storms, and usually occur in the southern desert regions (Colorado Desert, Sonoran Desert, Antelope Valley and Mojave Desert regions). Since these storms usually result in intense rainfall, over a small drainage area and are generally less than 6 hours, it is recommended that a 6-hour local design storm be utilized.

Drainage Areas > 20 mi^2 & ≤ 100 mi^2 – For drainage areas between 20 mi^2 and 100 mi^2, the critical storm can be a summer convective storm or a general thunderstorm. For these drainage areas, it is recommended that both 6-hour and 24-hour design storm be analyzed, and the storm that produces the largest peak discharge be chosen as the design basis.

Drainage Areas > 100 mi^2 – Since general storms usually cover a larger area and have a longer duration, for drainage areas greater than 100 mi^2, a 24-hour design storm is recommended.

(b) Depth-Duration-Frequency Characteristics

In 2011, NOAA published updated precipitation-frequency estimates for all of California including the desert regions, often cited as NOAA Atlas 14. This information is available online, via the Precipitation Frequency Data Server at http://hdsc.nws.noaa.gov/hdsc/pfds/ NOAA Atlas 14 supersedes NOAA's previous effort, NOAA Atlas 2, the 2004 Atlas 14 which covered the Southwestern U.S., and California's Department of Water Resources (DWR) Bulletin No. 195, where their coverages overlap.

Table 819.7B

Runoff Coefficients for Desert Areas

Type of Drainage Area	Runoff Coefficient
Undisturbed Natural Desert or Desert Landscaping (without impervious weed barrier)	0.30 – 0.40
Desert Landscaping (with impervious weed barrier)	0.55 – 0.85
Desert Hillslopes	0.40 – 0.55
Mountain Terrain (slopes greater than 10%)	0.60 – 0.80

Table 819.7C

Watershed Size for California Desert Regions

Desert Region	Duration (based on Watershed size)
Southern Regions (Colorado Desert, Sonoran Desert, Antelope Valley and Mojave Desert)	6-hour local storm ($\leq$ 20 mi^2) 6-hour local storm and 24-hour general storm (between 20 mi^2 & 100 mi^2); use the larger peak discharge 24-hour general storm (> 100 mi^2)
Northern Regions (Owens Valley/Mono Lake and Northern Basin and Range)	24-hour general storm

NOAA Atlas 14 provides a vast amount of information, which includes:

- Point Estimates
- ESRI shapefiles and ArcInfo ASCII grids
- Color cartographic maps: all possible combination of frequencies (2-year to 1,000-year) and durations (5-miunte to 60-day)
- Associated Federal Geographic Data Committee-compliant metadata
- Data series used in the analysis: annual maximum series and partial duration series
- Temporal distributions of heavy precipitation (6-hour, 12-hour, 24-hour and 96-hour)
- Seasonal exceedance graphs: counts of events that exceed the 1 in 2, 5, 10, 25, 50 and 100 annual exceedance probabilities for the 60-minute, 24-hour, 48-hour and 10-day durations

(c) Depth-Area Reduction

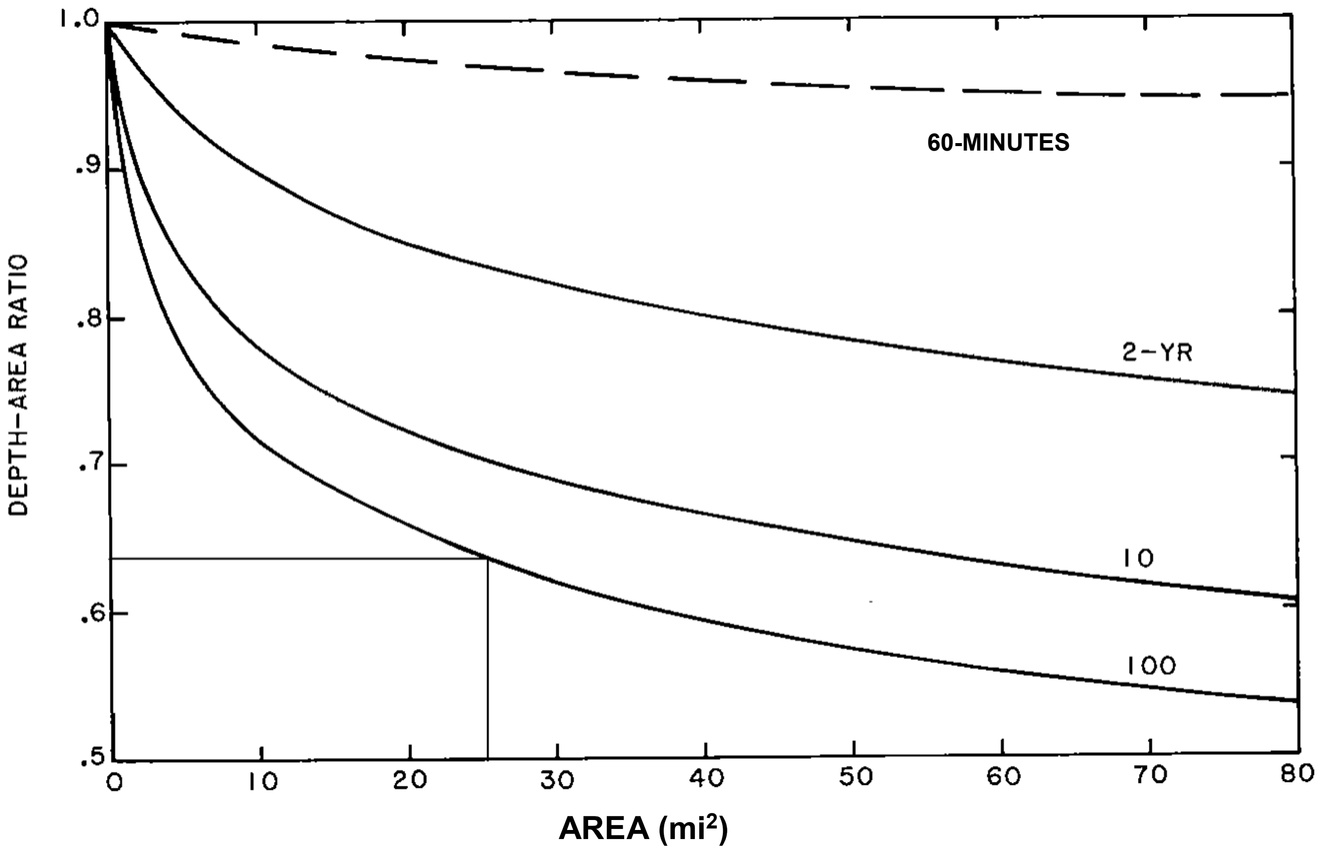
Depth-area reduction is the method of applying point rainfall data from one or several gaged stations within a watershed to that entire watershed. NOAA Atlas 14 provides high resolution depth-duration frequency point data which can then be computed with other depth-duration frequency data in that cell to obtain an average depth-duration frequency over a watershed. However, as this data is available as point data, the average calculated depth-duration frequency may not represent an entire watershed. To convert this point data into watershed area, a conversion factor may be applied, of which, two methods are available: applying a reduction factor; or applying depth-area reduction curves.

NOAA is currently working on updating the reduction factors, thus, until then, the depth-area reduction curves are recommended. Two depth-area reduction curves are available: (1) the depth curves in National Weather Service's HYDRO-40 http://www.nws.noaa.gov/oh/hdsc/PF_related_studies/TechnicalMemorandum_HYDRO40.pdf); and (2) the depth curves in NOAA Atlas 2. The general consensus is that the depth curves from HDRO-40 better represent the desert areas of California, and are recommended for the southern desert regions (Colorado Desert, Sonoran Desert, Antelope Valley and the Mojave Desert). For the upper regions (Owens Valley/Mono Lake and Northern Basin and Range), the curves from NOAA Atlas 2 are recommended.

The variables needed to apply depth area reduction curves to a watershed are a storm frequency (i.e., a 100-year storm), storm duration (i.e., a 30-minutes storm), and the area of a watershed. For example, if a 100-year storm with a duration of 60-minutes were to be analyzed over a desert watershed of 25 mi^2, then using Figure 819.7B, the Depth-Area Ratio would be 0.64.

Figure 819.7B

Example Depth-Area Reduction Curve



This ratio would then be multiplied by the averaged point- rainfall data, which would then result in the rainfall over the entire watershed.

Point rainfall data is available from NOAA Atlas 14, which must then be converted to area rainfall data. Conversions are available in two forms: (1) the National Weather Service's HYDRO-40, and (2) NOAA Atlas 2. The National Weather Service's HYDRO-40 is recommended for the southern desert regions (Colorado Desert, Sonoran Desert, Antelope Valley and Mojave Desert.) NOAA Atlas 2 is recommended for the northern desert regions (Owens Valley/Mono Lake and Northern Basin and Range).

(2) Rainfall Losses

Antecedent Moisture Condition – The Antecedent Moisture Condition (AMC) is the amount of moisture present in the soil before a rainfall event, or conversely, the amount of moisture the soil can absorb before becoming saturated (Note: the AMC is also referred to as the Antecedent Runoff Condition [ARC]). Once the soil is saturated, runoff will occur. Generally, the AMC is classified into three levels:

- AMC I – Lowest runoff potential. The watershed soils are dry enough to allow satisfactory grading or cultivation to take place.

- AMC II – Moderate runoff potential. AMC II represents an average study condition.
- AMC III – Highest runoff potential. The watershed is practically saturated from antecedent rainfall.

Because of the different storm types present in California's desert regions, AMC I is recommended as design criteria for local thunderstorms, and AMC II is recommended as design criteria for general storms.

Curve Number – The curve number was developed by the then Soil Conservation Service (SCS), which is now called the National Resource Conservation Service (NRCS). The curve number is a function of land use, soil type and the soil's AMC, and is used to describe a drainage area's storm water runoff potential. The soil type(s) are typically listed by name and can be obtained in the form of a soil survey from the local NRCS office. The soil surveys classify and present the soil types into 4 different hydrological groups, which are shown in Table 819.7D. From the hydrological groups, curve numbers are assigned for each possible land use-soil group combinations, as shown in Table 819.7E. The curve numbers shown in

Table 819.7D

Hydrologic Soil Groups

Hydrologic Soil Group	Soil Group Characteristics
A	Soils having high infiltration rates, even when thoroughly wetted and consisting chiefly of deep, well to excessively-drained sands or gravels. These soils have a high rate of water transmission.
B	Soils having moderate infiltration rates when thoroughly wetted and consisting of moderately deep to deep, moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.
C	Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.
D	Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

Table 819.7E are representative of AMC II, and need to be converted to represent AMC I, and AMC III, respectively. The following equations to convert an AMC II curve number to an AMC I or AMC III curve number, using a five-day period as the minimum for estimating the AMC's:

$$CN_{AMCI} = \frac{4.2CN_{AMCII}}{10 - 0.058CN_{AMCII}}$$

$$CN_{AMCIII} = \frac{23CN_{AMCII}}{10 + 0.13CN_{AMCII}}$$

Note: The AMC of a storm area may vary during a storm; heavy rain falling on AMC I soil can change the AMC from I to II or III during the storm.

(3) Transformation

Total runoff can be characterized by two types of runoff flow: direct runoff and base flow. Direct runoff is classified as storm runoff occurring during or shortly after a storm event. Base flow is classified as subsurface runoff from prior precipitation events and delayed subsurface runoff from the current storm. The transformation of precipitation runoff to excess can be accomplished using a unit hydrograph approach. The unit hydrograph method is based on the assumption that a watershed, in converting precipitation excess to runoff, acts as a linear, time-invariant system.

Unit Hydrograph Approach:

A unit hydrograph for a drainage area is a curve showing the time distribution of runoff that would result at the concentration point from one inch of effective rainfall over the drainage area above that point.

The unit hydrograph method assumes that watershed discharge is related to the total volume of runoff, that the time factors that affect the unit hydrograph shape are invariant, and that watershed rainfall-runoff relationships are characterized by watershed area, slope and shape factors.

(a) SCS Unit Hydrogoraph

The SCS dimensionless unit hydrograph is based on averages of unit hydrographs derived from gaged rainfall and runoff for a large number of small rural basins throughout the U.S. The definition of the SCS unit hydrograph normally only requires one parameter, which is lag, defined as the time from the centroid of precipitation excess to the time of the peak of the unit hydrograph. For ungaged watersheds, the SCS suggests that the unit hydrograph lag time, t_{lag}, may be related to time of concentration t_c, through the following relation:

$$t_{lag} = 0.6t_c$$

The time of concentration is the sum of travel time through sheet flow, shallow concentrated flow, and channel flow segments. A typical SCS Unit Hydrograph is similar to Figure 816.5.

A unit hydrograph can be derived from observed rainfall and runoff, however either may be unavailable. In such cases, a synthetic unit hydrograph can be developed using the S-graph method.

Table 819.7E

Curve Numbers for Land Use-Soil Combinations

Description	Average % Impervious	Curve Number by Hydrological Soil Group				Typical Land Uses
		A	B	C	D	
Residential (High Density)	65	77	85	90	92	Multi-Family, Apartments, Condos, Trailer Parks
Residential (Medium Density)	30	57	72	81	86	Single-Family, Lot Size ¼ to 1 acre
Residential (Low Density)	15	48	66	78	83	Single-Family, Lot Size 1 acre or greater
Commercial	85	89	92	94	95	Strip Commercial, Shopping Centers, Convenience Stores
Industrial	72	81	88	91	93	Light Industrial, Schools, Prisons, Treatment Plants
Disturbed / Transitional	5	76	85	89	91	Gravel Parking, Quarries, Land Under Development
Agricultural	5	67	77	83	87	Cultivated Land, Row Crops, Broadcast Legumes
Open Land – Good	5	39	61	74	80	Parks, Golf Courses, Greenways, Grazed Pasture
Meadow	5	30	58	71	78	Hay Fields, Tall Grass, Ungrazed Pasture
Woods (Thick Cover)	5	30	55	70	77	Forest Litter and Brush adequately cover soil
Woods (Thin Cover)	5	43	65	76	82	Light Woods, Woods-Grass Combination, Tree Farms
Impervious	95	98	98	98	98	Paved Parking, Shopping Malls, Major Roadways
Water	100	100	100	100	100	Water Bodies, Lakes, Ponds, Wetlands

(b) S-graph

An S-graph is a summation hydrograph of runoff that would result from the continuous generation of unit storm effective rainfall over the area (1-inch per hour continuously). The S-graph method uses a basic time-runoff relationship for a watershed type in a form suitable for application to ungaged basins, and is based upon percent of ultimate discharge and percent of lag time. Several entities, including local and Federal agencies, have developed location-specific S-Graphs that are applicable to California's desert regions.

The ordinate is expressed in percent of ultimate discharge, and the abscissa is expressed in percent of lag time. Ultimate discharge, which is the maximum discharge attainable for a given intensity, occurs when the rate of runoff on the summation hydrograph reaches the rate of effective rainfall.

Lag for a watershed is an empirical expression of the hydrologic characteristics of a watershed in terms of time. It is defined as the elapsed time (in hours) from the beginning of unit effective rainfall to the instant that the summation hydrograph for the point of concentration reaches 50 percent of ultimate discharge. When the lags determined from summation hydrographs for several gaged watersheds are correlated to the hydrologic characteristics of the watersheds, an empirical relationship is usually apparent. This relationship can then be used to determine the lags for comparable ungaged drainage areas for which the hydrologic characteristics can be determined, and a unit hydrograph applicable to the ungaged watersheds can be easily derived.

Figure 819.7C is a sample illustration of a San Bernardino County S-Graph, while Figure 819.7D shows an example S-Graph from USBR.

Recommendations:

For watersheds with mountainous terrain/high elevations in the upper portions, the SanBernardino County Mountain S-Graph (http://www.sbcounty.gov/dpw/floodcontrol/pdf/HydrologyManual.pdf)is recommended. For watersheds in the southern desert regions with limited or no mountainous terrain/high elevations, the San Bernardino County Desert S-Graph (http://www.sbcounty.gov/dpw/floodcontrol/pdf/HydrologyManual.pdf) is recommended. The U.S. Bureau of Reclamation (USBR) S-Graph (http://www.usbr.gov/pmts/hydraulics_lab/pubs/manuals/SmallDams.pdf) is recommended for watersheds in the Northern Basin and Range.

As an alternative to the above mentioned S-Graphs, the SCS Unit Hydrograph may also be used.

(4) Channel Routing

Channel routing is a process used to predict the temporal and spatial variation of a flood hydrograph as it moves through a river reach. The effects of storage and flow resistance within a river reach are reflected by changes in hydrograph shape and timing as the flood wave moves from upstream to downstream. The four commonly used methods are the kinematic wave routing, Modified Puls routing, Muskingum routing, and Muskingum-Cunge routing. The advantages and disadvantages for each method are described in Table 819.7F. Table 819.7G provides guidance for selecting an appropriate routing method. The Muskingum-Cunge routing method can handle a wide range of flow conditions with the exception of significant backwater. The Modified Puls routing can model backwater effects. The kinematic wave routing method is often applied in urban areas with well defined channels.

Figure 819.7C

San Bernardino County Hydrograph for Desert Areas

Discharge in Percent of Ultimate Discharge (K)

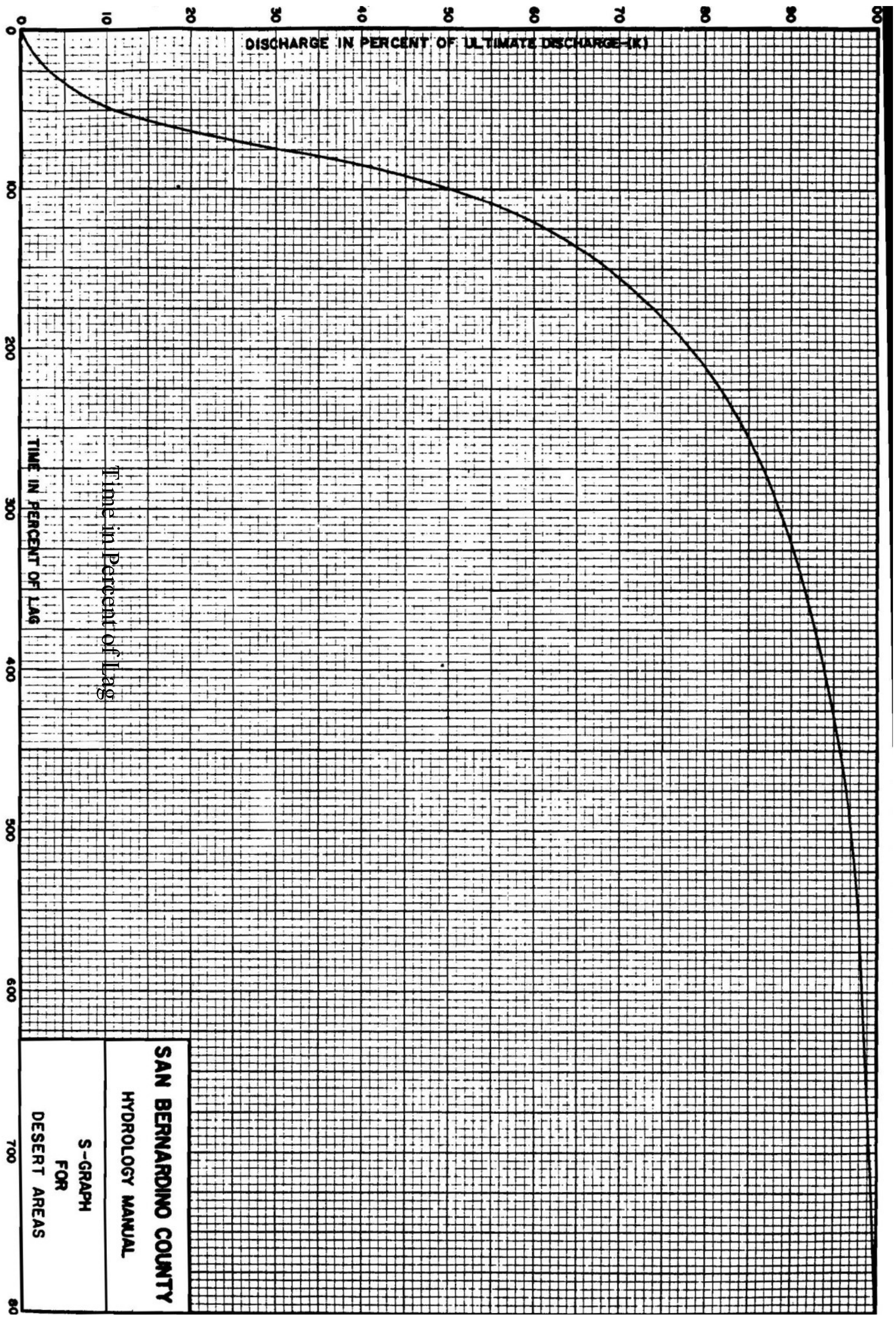
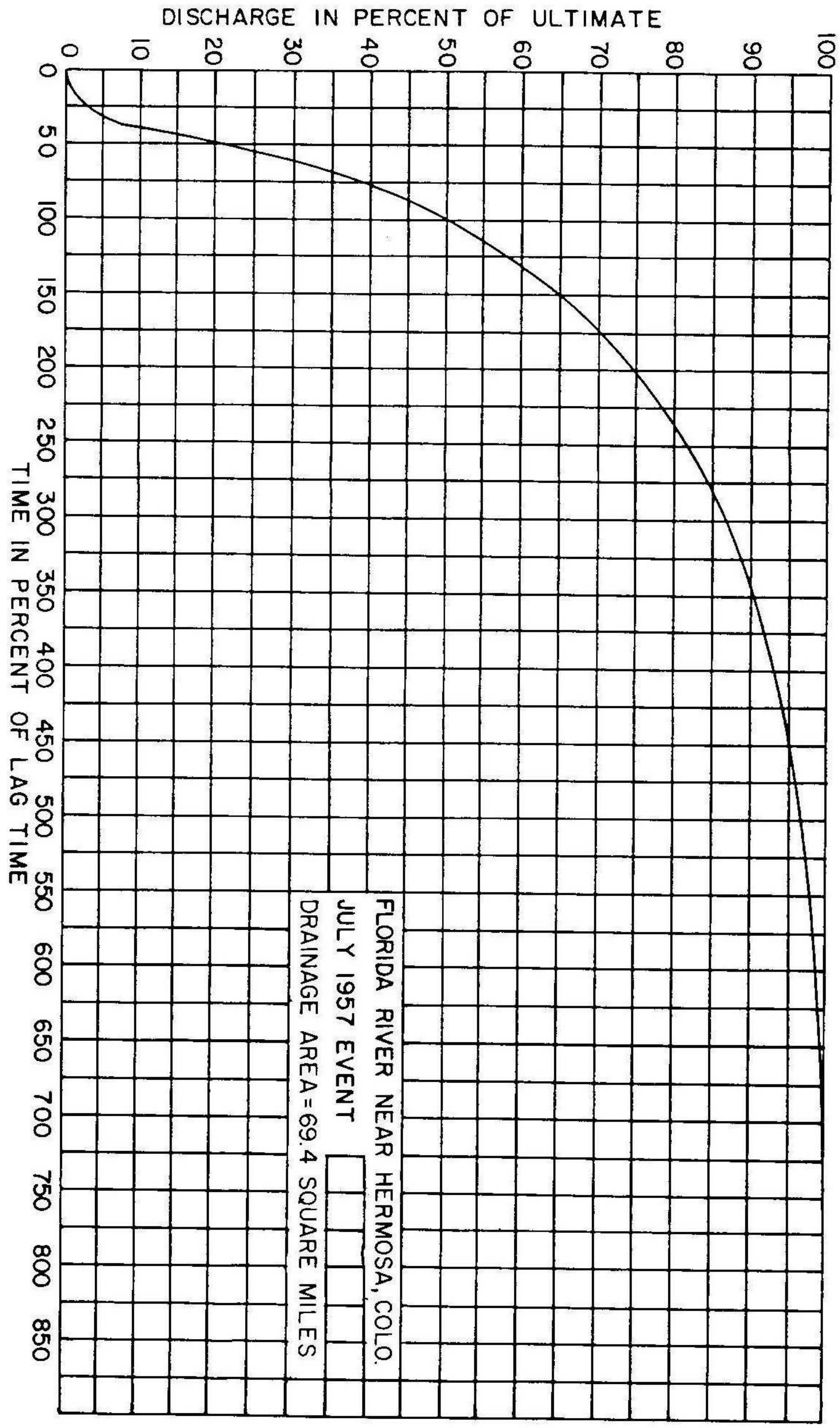


Figure 819.7D

USBR Example S-Graph



(5) Storm Duration and Temporal Distribution

Temporal distribution is the time-related distribution of the precipitation depth within the duration of the design storm. Temporal distribution patterns of design storms are based on the storm duration. The temporal distribution pattern for short-duration storms represents a single cloudburst and is based on rainfall statistics. The temporal distribution for long-duration storms resembles multiple events and is patterned after historic events. Since the storm events in California's desert regions are made up of two distinct separate storm types, the summer convective storm and the general winter storm, the design storm durations should be adjusted accordingly. For California's desert regions, the 100-year 6-hour storm is recommended for the convective storms, and the 100-year 24-hour storm is recommended for the winter storms. Table 819.7H summarizes the design storm durations for the different desert regions throughout California.

(2) Sediment/Debris Bulking

The process of increasing the water volume flow rate to account for high concentrations of sediment and debris is defined as bulking. Debris carried in the flow can be significant and greatly increase flow volume conveyed from a watershed. This condition occurs frequently in mountainous areas subject to wildfires with soil erosion, as well as arid regions around alluvial fans and other geologic activity. By bulking the flow through the use of an appropriate bulking factor, bridge openings and culverts can be properly sized for areas that experience high sediment and debris concentration.

(a) Bulking Factor

Bulking factors are applied to a peak (clear-water) flow to obtain a total or bulked peak flow, which provides a safety factor in the sizing of hydraulic structures. For a given watershed, a bulking factor is typically a function of the historical concentration of sediment in the flow.

(b) Types of Sediment/Water Flow

The behavior of flood flows will vary depending on the concentration of sediment in the mixed flow, where the common flow types are normal stream flow, hyperconcentrated flow, and debris flow.

(1) Normal Stream Flow

During normal stream flow, the sediment load minimally influences flow behavior or characteristics. Because sediment has little impact, this type of flow can be analyzed as a Newtonian fluid and standard hydraulic methods can be used. The upper limit of sediment concentration by volume for normal stream flow is 20 percent and bulking factors are applied cautiously because of the low concentration. (See Table 819.7I) The small amount of sediment is conveyed by conventional suspended load and bed-load.

(2) Hyperconcentrated Flow

Hyperconcentrated flow is more commonly known as mud flow. Because of potential for large volumes of sand in the water column, fluid properties and transport characteristics change and the mixture does not behave as a Newtonian fluid. However, basic hydraulic methods and models are still generally accepted and used for upto 40 percent sediment concentration by volume. For hyperconcentrated flow, bulking factors vary between 1.43 and 1.67 as shown in Table 819.7I.

Table 819.7F

Channel Routing Methods

Routing Method	Pros	Cons
Kinematic Wave	▪ A conceptual model assuming a uniform flow condition. ▪ In general, works best for steep (10 ft/mile or greater), well defined channels. ▪ It is often applied in urban areas because the routing reaches are generally short and well-defined.	▪ Cannot handle hydrograph attenuation, significant overbank storage, and backwater effects.
Modified Puls	▪ Known as storage routing or level-pool routing. ▪ Can handle backwater effects through the storage-discharge relationship.	▪ Need to use hydraulic model to define the required storage-outflow relationship.
Muskingum	▪ Directly accommodates the looped relationship between storage and outflow. ▪ A linear routing technique that uses coefficients to account for hydrograph timing and diffusion.	▪ The coefficients cannot be used to model a range of floods that may remain in bank or go out of bank. Therefore, not applicable to significant overbank flows.
Muskingum-Cunge	▪ A nonlinear coefficient method that accounts for hydrograph diffusion based on physical channel properties and the inflowing hydrograph. ▪ The parameters are physically based. ▪ Has been shown to compare well against the full unsteady flow equations over a wide range of flow conditions.	▪ It cannot account for backwater effects. ▪ Not very applicable for routing a very rapidly rising hydrograph through a flat channel.

Table 819.7G

Channel Method Routing Guidance

IF THIS IS TRUE…	… THEN THIS ROUTING MODEL MAY BE CONSIDERED.
No observed hydrograph data available for calibration	Kinematic wave; Muskingum-Cunge
Significant backwater will influence discharge hydrograph	Modified Puls
Flood wave will go out of bank, into floodplain.	Modified Puls; Muskingum-Cunge with 8-point cross section
Channel slope > 0.002 and $\frac{TS_o u_o}{d_o} \geq 171$	Any
Channel slopes from 0.002 to 0.0004 and $\frac{TS_o u_o}{d_o} \geq 171$	Muskingum-Cunge; Modified Puls; Muskingum
Channel slope < 0.0004 and $TS_o\left(\frac{g}{d_o}\right)^{1/2} \geq 30$	Muskingum-Cunge
Channel slope < 0.0004 and $TS_o\left(\frac{g}{d_o}\right)^{1/2} < 30$	None

Notes:
T = hydrograph duration
u_o = reference mean velocity
d_o = reference flow depth
S_o = channel slope

Table 819.7H

Design Storm Durations

Drainage Area	Desert Region	100-year, 6-hour Convective Storm (AMC I)	100-year, 24-hour General Storm (AMC II)	Regional Regression Equations
< 20 mi^2	Colorado Desert	X		
	Sonoran Desert	X		
	Mojave Desert	X		
	Antelope Valley Desert	X		
> 20 mi^2	Colorado Desert	X*	X*	
	Sonoran Desert	X*	X*	
	Mojave Desert	X*	X*	
	Antelope Valley Desert	X*	X*	
	Owens Valley/Mono Lake			X**
	Northern Basin & Range		X	

*For watersheds greater than 20 mi^2 in the southern desert regions, both the 6-hour Convective Storm (AMC I) and the 24-hour General Storm (AMC II) should be analyzed and the larger of the two peak discharges selected.

**The use of regional regression equations is recommended where streamgage data are not available; otherwise, hydrologic modeling could be performed with snowmelt simulation.

(3) Debris Flow

In debris flow state, behavior is primarily controlled by the composition of the sediment and debris mixture, where the volume of clay can have a strong influence in the yield strength of the mixture.

During debris flow, which has an upper limit of 50 percent sediment concentration by volume, the sediment/debris/water mixture no longer acts as a Newtonian fluid and basic hydraulic equations do not apply. If detailed hydraulic analysis or modeling of a stream operating under debris flow is needed, FLO2DH is the recommended software choice given its specific debris flow capabilities. HEC-RAS is appropriate for normal stream flow and hyperconcentrated flow, but cannot be applied to debris flow.

For a typical debris flow event, clear-water flow occurs first, followed by a frontal wave of mud and debris. Low frequency events, such as the 100-year flood, most likely contain too much water to produce a debris flow event. Normally, smaller higher frequency events such as 10-year or 25-year floods actually have a greater probability of yielding a debris flow event requiring a higher bulking factor.

As outlined in Table 819.7I, bulking factors for debris flow vary between 1.67 and 2.00.

(c) Sediment/Debris Flow Potential

(1) Debris Hazard Areas

Mass movement of rock, debris, and soil is the main source of bulked flows. This can occur in the form of falls, slides, or flows. The volume of sediment and debris from mass movement can enter streams depending upon hydrologic and geologic conditions.

The location of these debris-flow hazards include:

(a) At or near the toe of slope 2:1 or steeper

(b) At or near the intersection of ravines and canyons

(c) Near or within alluvial fans

(d) Soil Slips

Soil slips commonly occur at toes of slope between 2:1 and 3:1. Flowing mud and rocks will accelerate down a slope until the flow path flattens. Once energy loss occurs, rock, mud, and vegetation will be deposited. Debris flow triggered by soil slips can become channelized and travel distances of a mile or more. Figure 819.7E shows the potential of soil slip versus slope angle. As seen in this Figure, the flatter the slope angle, the less effect on flow speed and acceleration.

(2) Geologic Conditions

In the Transverse Ranges that include the San Gabriel and San Bernardino Mountains along the southern and southwestern borders of the Antelope Valley (Region 3) and Mojave Desert (Region 4), their substrate contains sedimentary rocks, fractured basement rocks, and granitic rocks. This type of geology has a high potential of debris flow from the hillsides of these regions.

While debris flow potential is less prevalent, it is possible to have this condition in the Peninsula Ranges that include the San Jacinto, Santa Rosa, and Laguna Mountains along the western border of the Colorado Desert (Region 1).

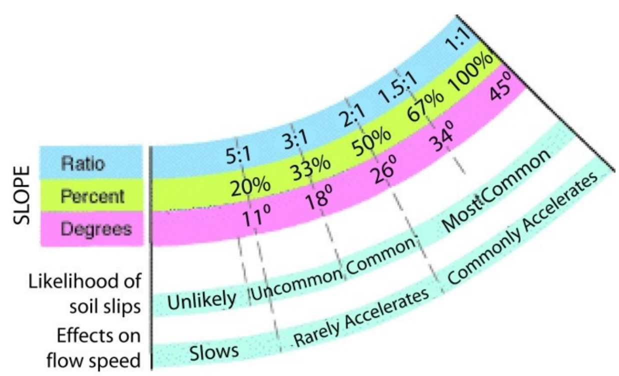
Table 819.7I

Bulking Factors & Types of Sediment Flow

Sediment Flow Type	Bulking Factor	Sediment Concentration by Weight (100% by WT = 1 x 10^6 ppm)	Sediment Concentration by Volume (specific gravity = 2.65)
Normal Streamflow	0	0	0
	1.11	23	10
	1.25	40	20
Hyperconcentrated Flow	1.43	52	30
	1.67	53	40
Debris Flow	2.00	72	50
Landslide	2.50	80	60
	3.33	87	70

Figure 819.7E

Soil Slips vs. Slope Angle



(d) Alluvial Fans

An alluvial fan is a landform located at the mouth of a canyon, formed in the shape of a fan, and created over time by deposition of alluvium. With the apex of the fan at the mouth of a canyon, the base of the fan is spread across lower lying plains below the apex. Over time, alluvial fans change and evolve when sediment conveyed by flood flows or debris flows is deposited in active channels, which creates a new channel within the fan. Potentially, alluvial fan flood and debris flows travel at high velocity, where large volumes of sediment can be eroded from mountain canyons down to the lower fan surface. Given this situation, the alignments of the active channels and the overall footprint of an alluvial fan are dynamic. Also, the concentration of sediment /debris volume is dynamic, ranging from negligible to 50 percent.

Alluvial fans can be found on soil maps, geologic maps, topographic maps, and aerial photographs, in addition to the best source which is a site visit. An example of an alluvial fan, shown in plan view, is in Figure 819.7F and Figure 872.3.

(e) Wildfire and Debris Flow

After fires have impacted a watershed, sediment/debris flows are caused by surface erosion from rainfall runoff and landsliding due to rainfall infiltration into the soil. The most dominant cause is the runoff process because fire generally reduces the infiltration and storage capacity of soils, which increases runoff and erosion.

Figure 819.7F

Alluvial Fan

(1) Fire Impacts

Arid regions do not have the same density of trees and vegetation as a forested area, but the arid environment still falls victim to fires in a similar manner. Prior to a fire, the arid region floor can contain a litter layer (leaves, needles, fine twigs, etc.), as well as a duff layer (partially decomposed components of the litter layer). These layers absorb water, provide storage of rainfall, and protect hillsides. Once these layers are burned, they become ash and charcoal particles that seal soil pores and decrease infiltration potential of the soil, which ultimately increases runoff and erosion.

In order to measure the burn severity of watersheds with respect to hydrologic function, classes of burn severity have been created. These classes are simply stated as high, moderate, low, and unburned. From moderate and high burn severity slopes, the generated sediment can reach channels and streams causing bulked water flows during storm events. Generally speaking, the denser the vegetation in a watershed prior to a fire and the longer a fire burns within this watershed, the greater the effects on soil hydrologic function. This occurs due to the fire creating a water repellent layer at or near the soil surface, the loss of soil structural stability, which all results in more runoff and erosion. After a one or two-year period, the water repellent layer is usually washed away.

(f) Local Agency Methods For Predicting Bulking Factors

(1) San Bernardino County

Instead of conducting a detailed analysis, San Bernardino Flood Control District uses a set value for bulking of 2 (i.e., 100 percent bulking) for any project where bulking flows may be anticipated. This bulking factor of 2 can also be expressed as a 50 percent sediment concentration by volume, which is about the upper limit of debris flow. A higher percentage of sediment concentration would be considered a landslide instead of debris flow. Basically, the San Bernardino County method assumes debris flow conditions for all types of potential bulking.

(2) Los Angeles County

The Los Angeles (LA) County method uses a watershed-specific bulking factor. The LA County Sedimentation Manual, which is located at http://ladpw.org/wrd/publication, divides the county into three basins: LA Basin, Santa Clara River Basin, and Antelope Valley, where only the latter is located in the Caltrans desert hydrology regions. The production of sediment from these basins is dependent upon many factors, including rainfall intensity, vegetative cover, and watershed slope. For each of the LA County basins, Debris Potential Area (DPA) zones have been identified.

The Design Debris Event (DDE) is associated with the 50-year, 24-hour duration storm, and produces the quantity of sediment from a saturated watershed that is recovered from a burn. For example, a DPA 1 zone sediment rate of 120,000 cubic yards per square mile has been established as the DDE for a 1-square mile drainage area. This sediment rate is recommended for areas of high relief and granitic formation found in the San Gabriel Mountains. In other mountainous areas in LA County, lower sediment rates have been assigned based on differences in topography, geology, and precipitation. For the Antelope Valley basin, eight debris production curves have been generated, and can be found in Appendix B of the LA County Sedimentation Manual along with curves for the other basins.

In addition to sediment production rates, a series of peak bulking factor curves are presented for each LA County basin in Appendix B of the LA manual. The peak bulking factor can be estimated using these curves based on the watershed area and the DPA. Within the Antelope Valley basin, maximum peak bulking factors range from 1.2 in DPA Zone 11 to 2.00 in DPA Zone 1.

(3) Riverside County

For Riverside County, a bulking factor is calculated by estimating a sediment/debris yield rate for a specific storm event, and relating it to the largest expected sediment yield of 120,000 cubic yards per square mile for a 1-square mile watershed from the LA County procedure. This sediment rate from LA County is based on the DPA Zone 1 corresponding to the highest expected bulking factor of 2.00.

The bulking factor equation from the Riverside County Hydrology Manual (http://www.floodcontrol.co.riverside.ca.us/downloads/planning/) is as follows:

$$BF = 1 + \frac{D}{120,000}$$

BF = Bulking Factor

D = Design Storm Sediment/Debris Production Rate For Study Watershed (cubic yards/square mile)

(4) U.S. Army Corps of Engineers- LA District

This method, located at http://www.spl.usace.army.mil/resreg/htdocs/Publications.html, was originally developed to calculate unit sediment/debris yield values for an “n-year” flood event, and applied to the design and analysis of debris catching structures in coastal Southern California watersheds. The LA District method considers frequency of wildfires and flood magnitude in its calculation of unit debris yield. Even though its original application was intended for coastal-draining watersheds, this method can also be used for desert-draining watersheds for the same local mountain ranges.

The LA District method can be applied to watershed areas between 0.1 and 200 mi^2 that have a high proportion of their total area in steep, mountainous topography. This method is best used for watersheds that have received significant antecedent rainfall of at least 2 inches in 48 hours. Given this criteria, the LA District method is more suited for general storms rather than thunderstorms.

As shown below, this method specifies a few equations to estimate unit debris yield dependent upon the areal size of the watershed. These equations were developed by multiple regression analysis using known sediment/debris data.

For watersheds between 3 and 10 mi^2, the following equations can be used:

$$\log Dy = 0.85 \log Q + 0.53 \log RR + 0.04 \log A + 0.22 FF$$

D_y = Unit Debris Yield (cubic yards/square mile)

RR= Relief Ratio (foot/mile), which is the difference in elevation between the highest and lowest points on the longest watercourse divided by the length of the longest watercourse

A = Drainage Area (acres)

FF = Fire Factor

Q = Unit Peak Runoff (cfs/square mile)

In order to account for increase in debris yield due to fire, a non-dimensional fire factor (FF) is a component in the equation above. The FF varies from 3.0 to 6.5, with a higher factor indicating a more recent fire and more debris yield. This factor is 3.0 for desert watersheds because the threat and effects from fire are minimal.

Because the data used to develop the regression equation was taken from the San Gabriel Mountains, an Adjustment and Transposition (A-T) factor needs to be applied to debris yields from the study watersheds. The A-T factor can be determined using Table 819.7J by finding the appropriate subfactor for each of the four groups (Parent Material, Soils, Channel Morphology, and Hillside Morphology) and summing the subfactors. This sum is the total A-T factor, and it must be multiplied by the sediment/debris yield. Once the sediment/debris yield value has been determined based on the unit yield, a bulking factor can be calculated using a series of equations. The first equation provides a translation of the clear-water discharge to a sediment discharge. This clear-water discharge should be developed using a hydrograph method and a hydrologic modeling program, such as HEC-HMS.

$$Q_s = aQ_w{}^n$$

Q_S = Sediment Discharge (cfs)

Q_W = 100-Year Clear-Water Discharge (cfs)

a = Bulking Constant

For a majority of sand-bed streams, the value of “n” is between 2 and 3. When n=2, the bulking factor is linearly proportional to the clear-water discharge. As for the coefficient “a”, it is determined with the following equation:

$$a = \frac{V_s}{\Delta t \sum Q_w{}^2}$$

V_S = Total Sediment Volume (cubic feet)

Δt = Computation Time Interval Used In Developing Hydrograph From Hydrologic Model (e.g. HEC-HMS)

Finally, the bulking factor equation is expressed as follows:

$$BF = \frac{Q_w - Q_s}{Q_s} = 1 + aQ_w{}^{n-1}$$

(g) Recommended Approach For Developing Bulking Factors

A flow chart outlining the recommended bulking factor process is provided in Figure 819.7H, which considers all bulking methods presented in Topic 819.

As shown in Steps 4 and 5 on Figure 819.7H, a bulking factor can be found by:

(1) Identifying the type of flow within a watershed and selecting the corresponding bulking factor, or

(2) Using one of the agency methods to calculated the bulking factor.

If the type of flow cannot be identified or the project site does not fall within the recommended boundaries from Figure 819.7H, use the LA District Method because it is the most universal given its use of the Adjustment-Transposition factor based on study watershed properties.

Table 819.7J

Adjustment-Transportation Factor Table

	A-T SUBFACTOR				
	0.25	0.20	0.15	0.10	0.05
PARENT MATERIAL			SUBFACTOR GROUP 1		
Folding	Severe		Moderate		Minor
Faulting	Severe		Moderate		Minor
Fracturing	Severe		Moderate		Minor
Weathering	Severe		Moderate		Minor
SOILS			SUBFACTOR GROUP 2		
Soils	Non-cohesive		Partly Cohesive		Highly Cohesive
Soil Profile	Minimal Soil Profile		Some Soil Profile		Well-developed Soil Profile
Soil Cover	Much Bare Soil in Evidence		Some Bare Soil in Evidence		Little Bare Soil in Evidence
Clay Colloids	Few Clay Colloids		Some Clay Colloids		Many Clay Colloids
CHANNEL MORPHOLOGY			SUBFACTOR GROUP 3		
Bedrock Exposures	Few Segments in Bedrock		Some Segments in Bedrock		Many Segments in Bedrock
Bank Erosion	> 30% of Banks Eroding		10 – 30% of Banks Eroding		< 10% of Banks Eroding
Bed and Bank Materials	Non-cohesive Bed and Banks		Partly Cohesive Bed and Banks		Mildly Cohesive Bed and Banks
Vegetation	Poorly Vegetated		Some Vegetation		Much Vegetation
Headcutting	Many Headcuts		Few Headcuts		No Headcutting
HILLSLOPE MORPHOLOGY			SUBFACTOR GROUP 4		
Rills and Gullies	Many and Active		Some Signs		Few Signs
Mass Movement	Many Scars Evident		Few Signs Evident		No Signs Evident
Debris Deposits	Many Eroding Deposits		Some Eroding Deposits		Few Eroding Deposits
The A-T Factor is the sum of the A-T Subfactors from all 4 Subfactor Groups.					

Figure 819.7H

Recommended Bulking Factor Selection Process

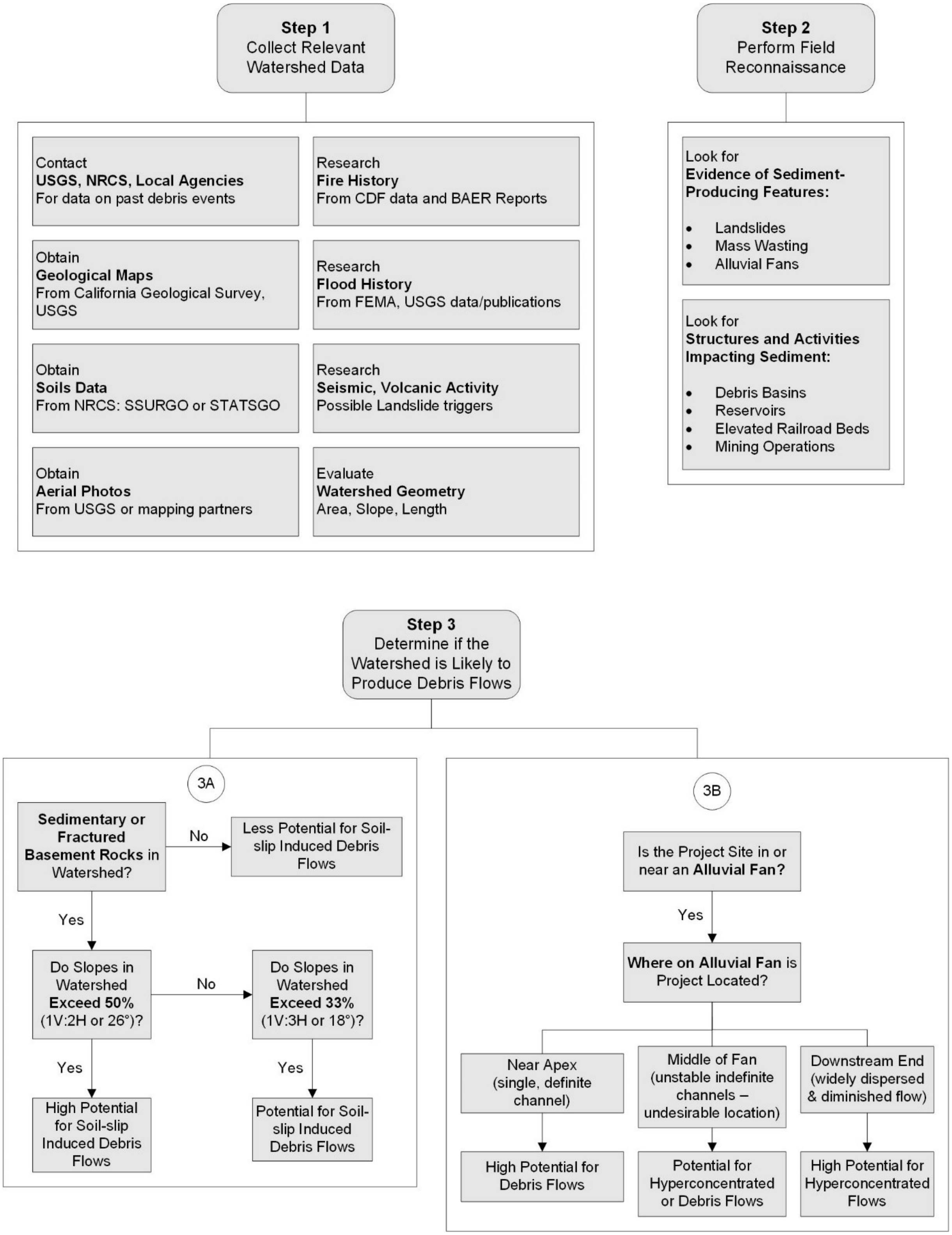


Figure 819.7H

Recommended Bulking Factor Selection Process (Cont'd)