

Rainfall-Runoff Relationships

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

6.1 RAINFALL LOSSES AND RUNOFF PRODUCTION

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

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

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

6.2 INFILTRATION

Infiltration losses have the greatest effect on surface runoff. The rate of infiltration is a function of the state of the soil and is highly heterogeneous over space and time. Hydraulic conductivity is a measure of the ease with which water can travel through the soil and is a measure of the infiltration

rate when the soil is saturated. Similar soils generally have similar hydraulic conductivities. However, the infiltration rate is also affected by the degree of soil saturation. Dry soil allows more infiltration than wet soil. Factors such as ground cover or recent fires within the watershed affect the soil surface and infiltration rates.

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

6.3 MODIFIED RATIONAL LOSS CALCULATIONS

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

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

$$Q = C \cdot I \cdot A$$

Equation 6.3.1

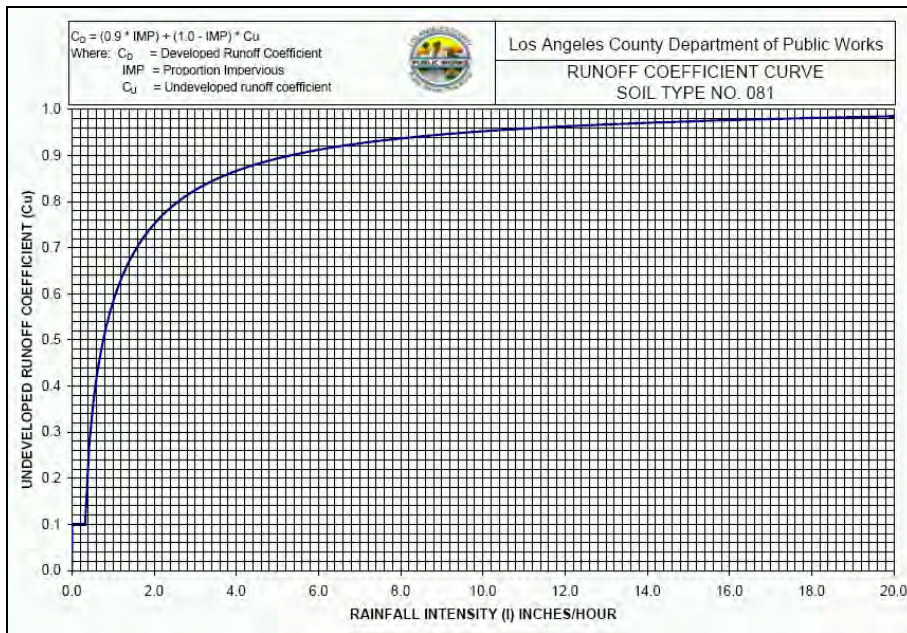
Where:

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

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

Undeveloped Runoff Coefficient (C_u)

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

**Figure 6.3.1**Runoff Coefficient Curve for
Soil 081

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

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

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

Developed Soil Runoff Coefficient Curves (C_D)

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

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

Equation 6.3.2

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

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

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

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

Table 6.3.1

Composite Impervious Values

To determine the composite impervious value for this subarea, calculate the area weighted average of imperviousness. First, multiply each impervious

value by the area it represents. Then sum these products and divide by the total area. The composite area weighted imperviousness for the example subarea is:

$$\text{Composite imperviousness} = \frac{2245}{40} = 56\%$$

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

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

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

Table 6.3.2

Standard Range of Percent Impervious

Burned Soil Runoff Coefficient Curves (C_{ba})

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

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

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



Figure 6.3.2

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

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

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

Table 6.3.3

Design Fire Factors for Use
with Burned Watershed
Hydrology

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

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

Equation 6.3.3

Where:

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

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

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

Equation 6.3.4

Where:

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

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

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

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

6.4 CONSTANT LOSS METHOD

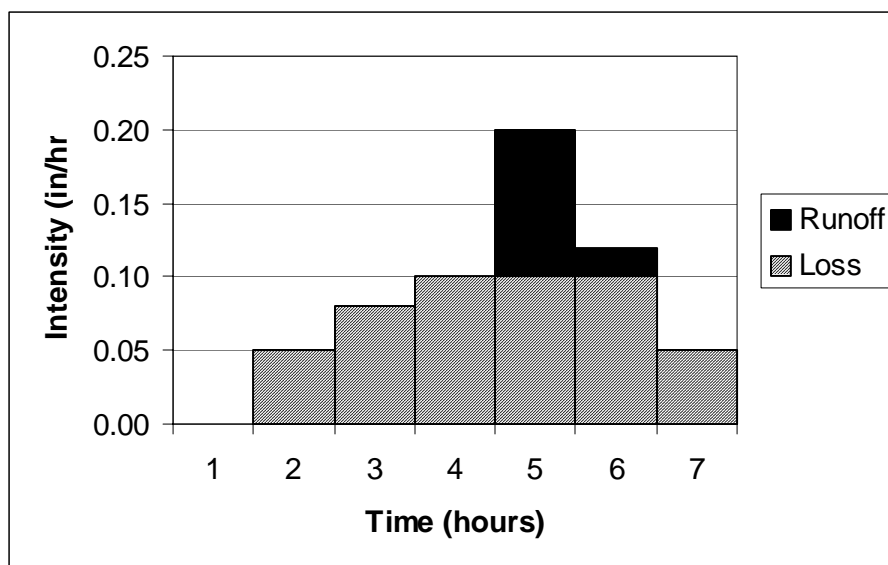
The constant loss method is a frequently used and generally accepted rainfall loss method for flood hydrology. The constant loss method models infiltration by allowing all rainfall to infiltrate when the rainfall intensity is below a certain rate. All rainfall exceeding this infiltration rate will run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

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

Table 6.4.1

Application of Constant Loss Method

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

**Figure 6.4.1**

Rainfall Hyetograph and
resulting Constant Loss
Runoff

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

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

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

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

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

CHAPTER

7

Runoff Calculation Methods

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

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

7.1 SELECTING THE PROPER METHOD

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

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

Table 7.1.1

County of Los Angeles
Hydrologic Methods

7.2 RATIONAL METHOD

Mulvaney first outlined the rational method¹, which assumes that a steady, uniform rainfall rate will produce maximum runoff when all parts of the watershed are contributing to outflow². This occurs when the storm event lasts longer than the time of concentration. The time of concentration is the time it takes for rain in the most hydrologically remote part of the watershed to reach the outlet. The method assumes that the runoff coefficient remains constant during a storm. The rational method formula is $Q = CIA$, previously mentioned in Chapter 6 as Equation 6.3.1. The direct runoff volume is calculated using the following equation:

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

Equation 7.2.1

Where:

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

Use of the rational method for drainage system design in small urban areas is appropriate. Use within the County of Los Angeles requires subarea division when³:

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

The following are disadvantages of the classic rational method:

- Does not produce a hydrograph
- Runoff coefficient, C, is usually the same regardless of rainfall intensity
- Results are unreliable for areas greater than 200 acres⁸

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

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

7.3 MODIFIED RATIONAL METHOD

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

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



Figure 7.3.1

Water storage
occurring in
Bradbury Channel
May 28, 1981

Time of Concentration

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

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

Time of Concentration - Kinematic Wave Theory⁴

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

$$T_C = t_o + t_c$$

Equation 7.3.1

Where:

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

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

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

Equation 7.3.2

$$I_x = C * I$$

Equation 7.3.3

Where:

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

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

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

Table 7.3.1

Roughness Coefficients for Overland Flow Computation

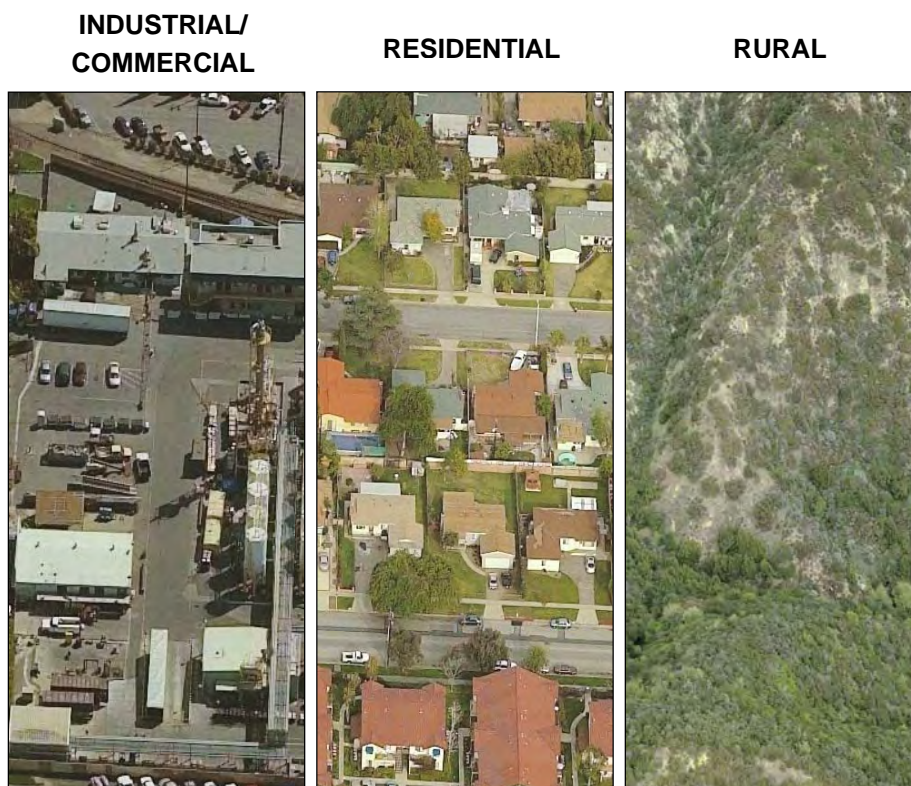
Table 7.3.2 shows standard values for different types of lots. The kinematic wave method requires evaluation of each subarea to determine the overland flow length and slope.

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

Table 7.3.2

Standard Values for Overland Flow Computation

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

**Figure 7.3.2**

Different Types of Lots Where Overland Flow Occurs

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

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

Equation 7.3.4

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

Comparison of results from Equation 7.3.1 with Izzard's overland flow experimental results and the results of Yu and McNown showed good correlation⁶.

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

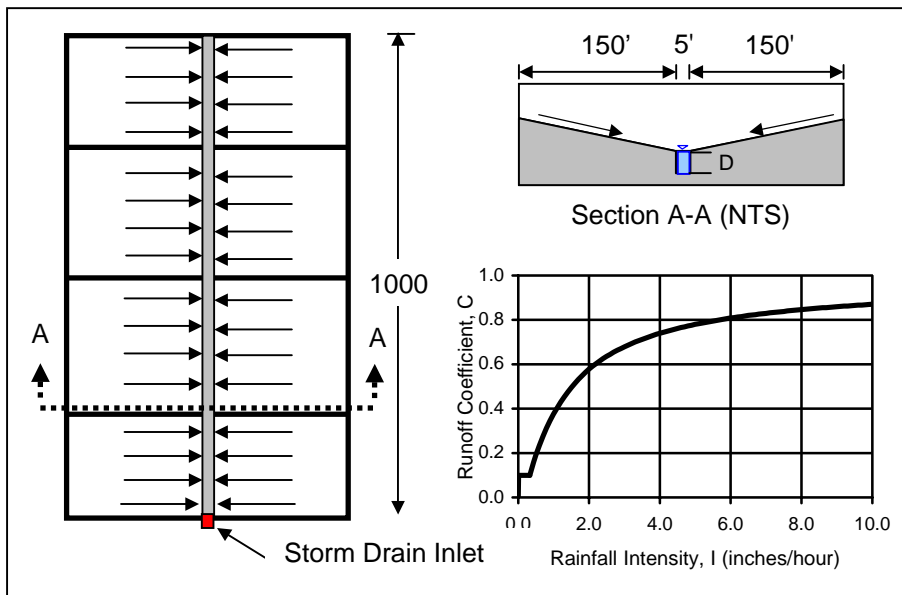


Figure 7.3.3

Example Subarea
 Demonstrating Kinematic
 Wave Method

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

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

Table 7.3.3

Kinematic Wave
Conveyance Data

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

1. Assume an initial time of concentration

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

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

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

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

$$C_d = (0.9 * IMP) + (1.0 - IMP) * C_u \\ = (0.9 * 0.42) + (1.0 - 0.42) * 0.58 = 0.71$$

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

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

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

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

5. **Determine the velocity for the average channel flow**

Solving Manning's Equation for $V = 3.39 \text{ ft/s}$

6. **Calculate the conveyance flow travel time using Equation 7.3.4**

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

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

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

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

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

$$\begin{aligned} I &= 1.90 \text{ in/hr} \\ t_o &= 7.94 \text{ minutes} \\ Q_{ave} &= 4.66 \text{ cfs} \\ V_{ave} &= 3.33 \text{ ft/s} \\ t_c &= 5.00 \text{ minutes} \\ T_C &= 7.94 + 5.00 = 12.94 \text{ minutes} \end{aligned}$$

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

Time of Concentration - Regression Equation⁷

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

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

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

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

$$C_d = (0.9 * IMP) + (1.0 - IMP) * C_u \quad \text{(Equation 6.3.2)}$$

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

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

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

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

Hydrograph Generation

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

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

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

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

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

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

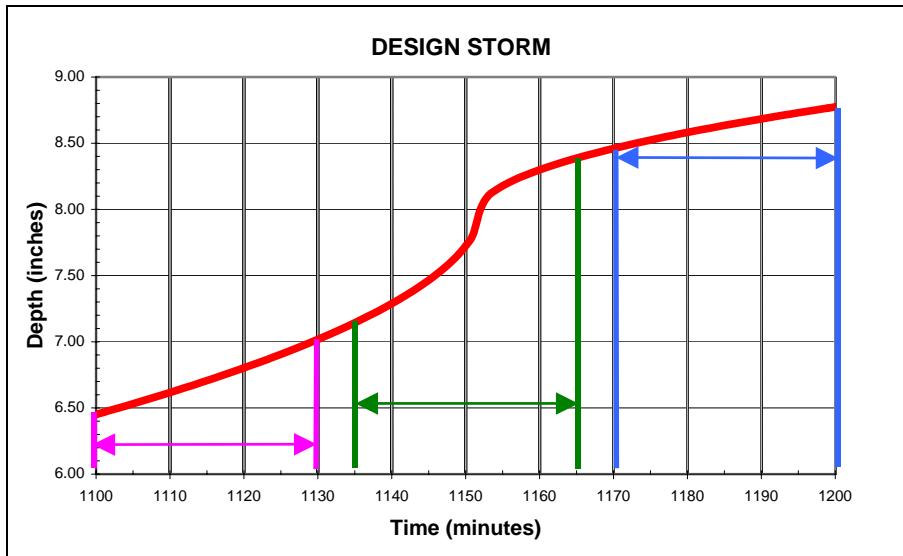


Figure 7.3.4

Three Time Steps for Modified Rational Runoff Calculations

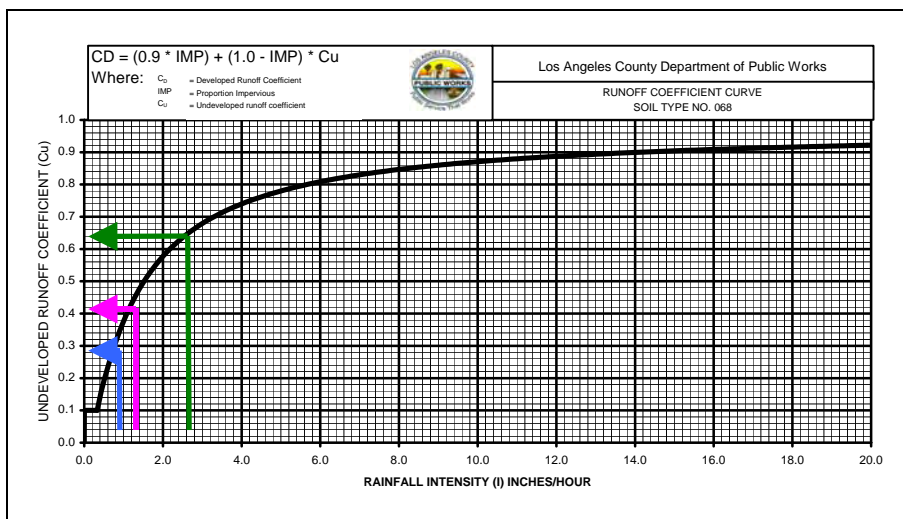


Figure 7.3.5

Undeveloped Runoff Coefficients for 3 Time Steps

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

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

Table 7.3.4
Table of Runoff
Calculations

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

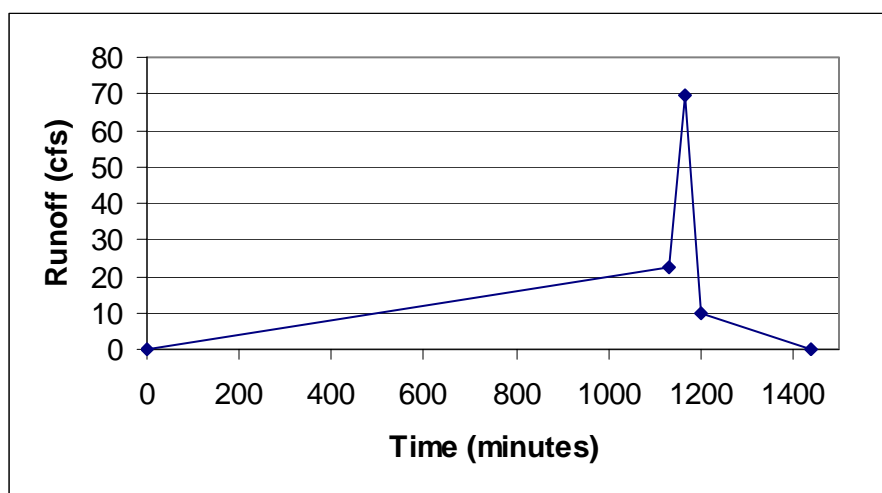


Figure 7.3.6
Hydrograph Generate Using
MODRAT Method

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

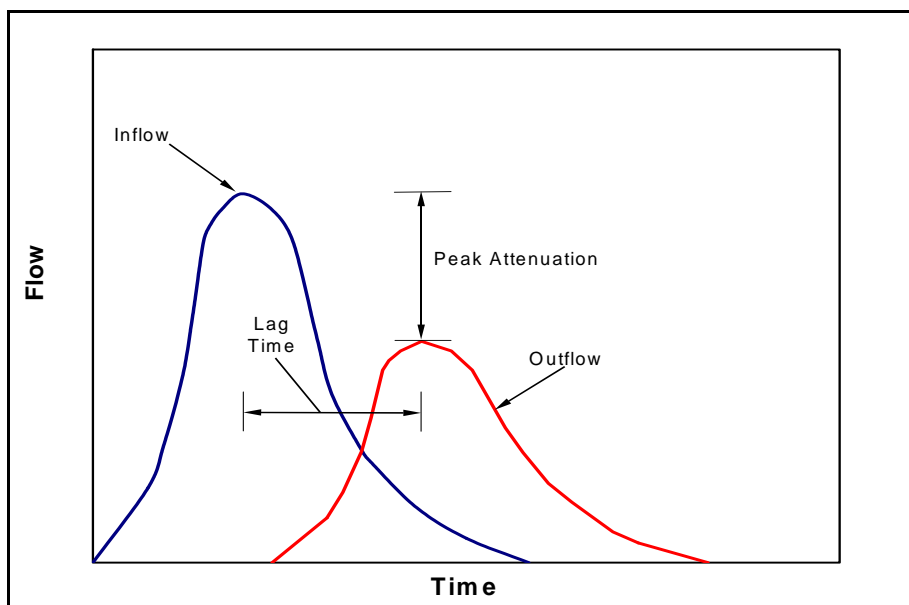
$$\text{Vol} = \frac{1}{2} * b * h = \frac{1}{2} * 1130 \text{ minutes} * 22.3 \frac{\text{ft}^3}{\text{sec}} * 60 \frac{\text{sec}}{\text{min}} = 755,970 \text{ ft}^3 = 17.35 \text{ ac} \cdot \text{ft}$$

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

Channel Routing of Flows

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

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

**Figure 7.3.7**

Peak Attenuation Related to
Channel Storage

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

There are many methods available for hydrologic routing⁸. The MODRAT method uses the Modified Puls, or level pool, routing method to determine channel storage effects. The method relies on a finite difference approximation of the continuity equation and an empirical representation of the momentum equation. Equation 7.3.8 is the basic equation for the Modified Puls method. The equation allows calculation of the outflow for each time step except the first. Chapter 8 shows another way to write the equation for the Modified Puls method that removes the need to calculate the storage for each time step.

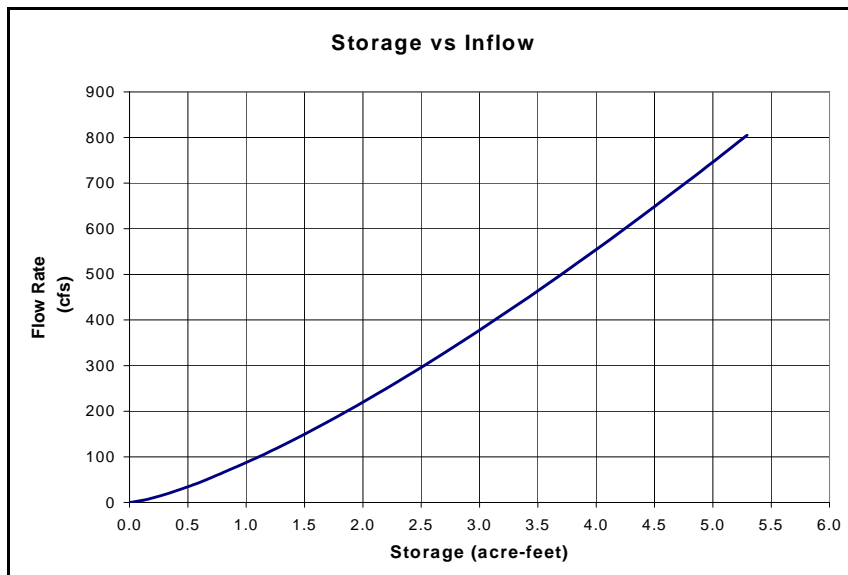
$$\frac{1}{2}(I_i + I_{i-1}) - \frac{(S_i - S_{i-1})}{t_i - t_{i-1}} = \frac{1}{2}(O_{i-1} + O_i)$$

Equation 7.3.8

Where:

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

The method ignores wedge storage within the channel reach and assumes that lateral inflow effects are insignificant. A storage-discharge relationship is also required between the inflow rate and storage in the system⁹. The method requires a defined channel storage versus inflow relationship. The relationship is established using the Manning equation to determine depth of flow. Multiplying channel length, water depth, and cross sectional area provides the channel storage for a specific flow value. Using different flow values produces a storage curve. Figure 7.3.8 presents the channel storage relationship for a triangular channel with the following characteristics: slope = 0.001 ft/ft, length = 1000 ft, Manning $n = 0.03$, side slope = 1:1 ft:ft, and max depth = 6.8 ft.

**Figure 7.3.8**

Storage-Inflow Relationship
for a Triangular Channel

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

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

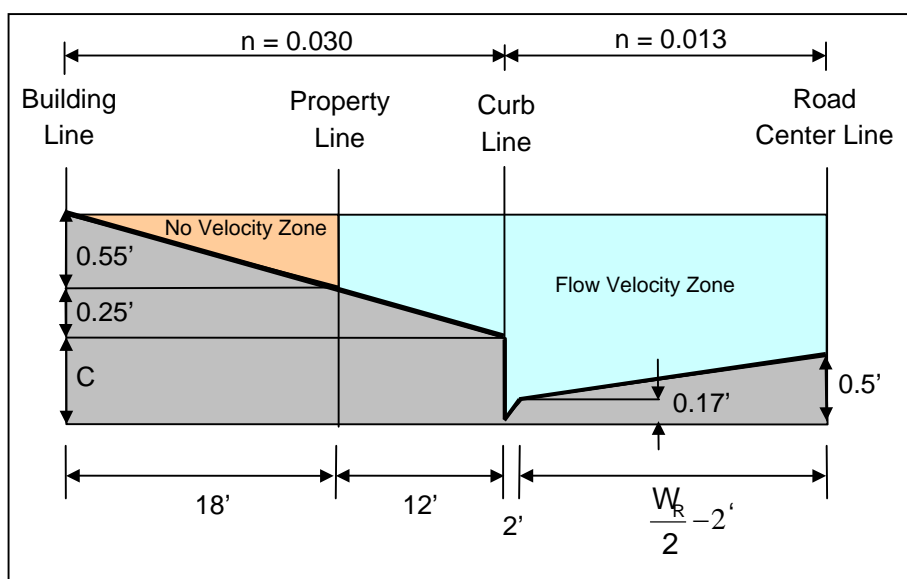
Computer programs implement this approach to reduce the amount of work required to define these relationships and route flows through the channels. Chapter 8 contains a detailed example of the Modified Puls routing method.

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

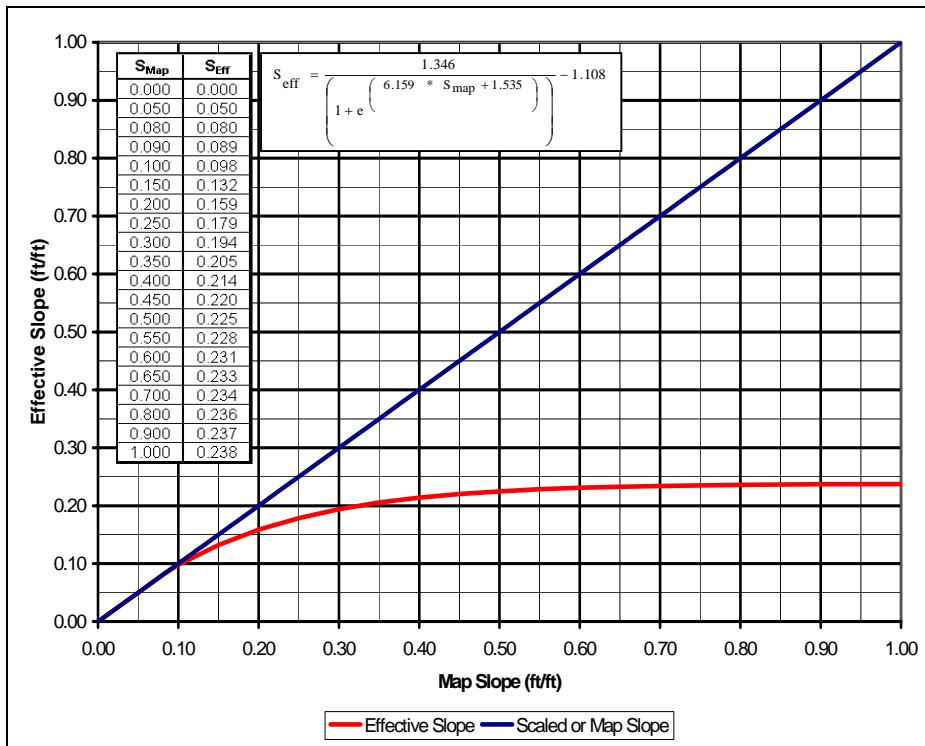
Table 7.3.5

Hydrograph Translation Equations

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

**Figure 7.3.9**

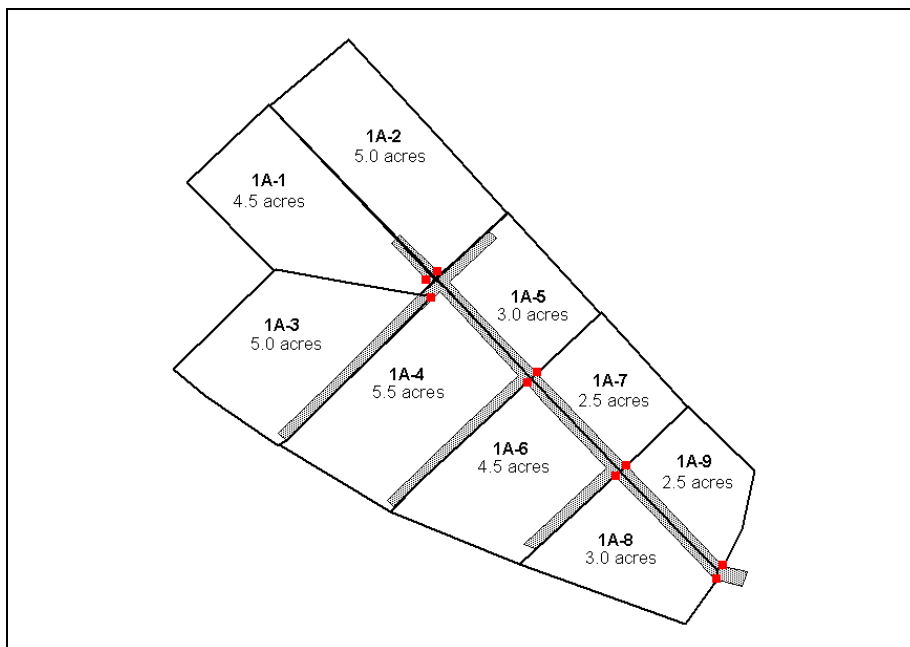
Typical Street Cross Section

**Figure 7.3.10**

Effective Slope to Map Slope Relationship

7.4 CATCH BASIN FLOW CALCULATIONS

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

**Figure 7.4.1**

Catch Basin Flow Allotment

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

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

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

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

Table 7.4.1

Peak Flow Allotment for Catch Basins within Subarea 1A

7.5 REPORTING RUNOFF VALUES

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

Peak Flow Reporting - USGS Method

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

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

Table 7.5.1

USGS Flow Reporting Rounding Rules

Peak Flow Reporting - Reach Grouping

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

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

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

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

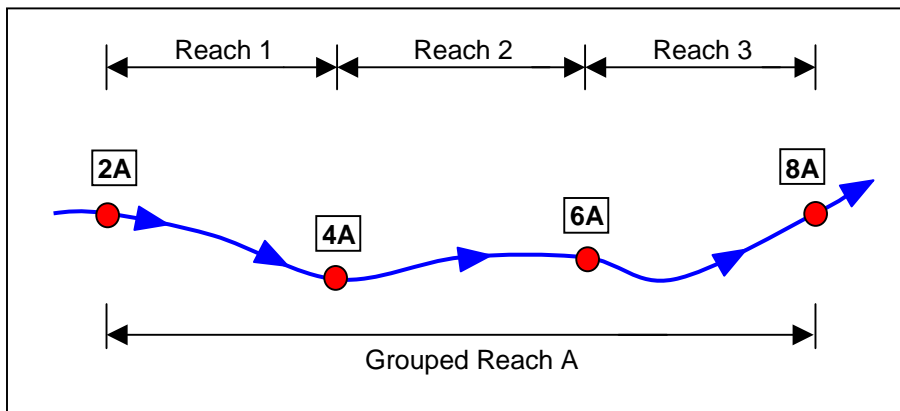
Flow Rate (cfs)	Round Flow To Nearest
$0 \leq Q_{\text{burn}} < 20$	0.1 cfs
$20 \leq Q_{\text{burn}} < 100$	5 cfs
$100 \leq Q_{\text{burn}} < 1,000$	10 cfs
$1,000 \leq Q_{\text{burn}} < 100,000$	100 cfs
$Q_{\text{burn}} \geq 100,000$	1,000 cfs

Table 7.5.2

Rounding Rules for
Reach Grouping

EXAMPLE – Reach Grouping for Reporting Bulked Flow Rates

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

**Figure 7.5.1**

Grouped Channel Reach
Based on Reach Flows

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

Table 7.5.3

Grouped Reach Flow Rates

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

Figure 7.5.2

Downstream of Hook Canyon
in Glendora
January 26, 1969



- ¹ Mulvaney, T.J. "On the Use of Self-Registering Rain and Flood Gauges. Inst. Civ. Eng. (Ireland) Trans. Vol. 4. pages 1-8. 1851.
- ² Bedient, P.B. and W.C. Huber. Hydrology and Floodplain Analysis, 3rd Ed. Prentice-Hall, Inc. NJ. page 84. 2002.
- ³ US Army Corps of Engineers. Hydraulic Design of Stream Restoration (ERDC/CHL TR-01-28). page 24, Washington, D.C. 2001.
- ⁴ Nasser, I. Use of Kinematic Wave Theory With the Rational Method. ASCE Engineering Workshop on Peak Reduction for Drainage and Flood Control Projects. Proceedings May 9, 1987.
- ⁵ Bedient, P.B. and W.C. Huber. Hydrology and Floodplain Analysis, 3rd Ed. Prentice-Hall, Inc. NJ. page 246. 2002.
- ⁶ Nasser, I. Use of Kinematic Wave Theory With the Rational Method. ASCE Engineering Workshop on Peak Reduction for Drainage and Flood Control Projects. Proceedings May 9, 1987. page 132.
- ⁷ Los Angeles County Hydrologic Method Approval Memorandum. Los Angeles County Department of Public Works. March 4, 2002.
- ⁸ US Army Corps of Engineers. Hydrologic Modeling System HEC-HMS Technical Reference Manual. CPD-74B. March 2000.
- ⁹ Bedient, P.B. and W.C. Huber. Hydrology and Floodplain Analysis, 3rd Ed. Prentice-Hall, Inc. NJ. page 246. 2002.

CHAPTER

8

Reservoir and Basin Routing

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



Figure 8.1

San Gabriel Reservoir
April 28, 1975

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

Modified Puls method in more detail. Equation 8.1 is the finite difference form of the continuity equation used for reservoir routing¹. Equation 8.2 provides a relationship that is used to calculate outflow without actually calculating storage for a given time step. The example problem illustrates use of the equations.

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

Equation 8.1

Form of the Continuity
Equation Used for Reservoir
Routing

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

Equation 8.2

Relationship Used to
Calculate Outflow Without
Calculating Storage

Where:

I_n	= Inflow at time _n
I_{n+1}	= Inflow at time _{n+1}
Δt	= Difference in time, time _{n+1} - time _n
S_n	= Storage at time _n
S_{n+1}	= Storage at time _{n+1}
O_n	= Outflow at time _n
O_{n+1}	= Outflow at time _{n+1}

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

EXAMPLE – Modified Puls Routing Through a Reservoir

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

The detention basin has the storage capacity shown in Table 8.1. Outflow from the basin occurs through an 24-inch drain when the water surface elevation is below 6 feet. When the water surface elevation is above 6 feet, outflow occurs through the drainpipe and over a weir. The weir is 20 feet long and has a weir coefficient of 3.5. Equations 8.3 and 8.4 provide the outflow relationships for the weir and drainpipe based on elevation as shown in Table 8.1.

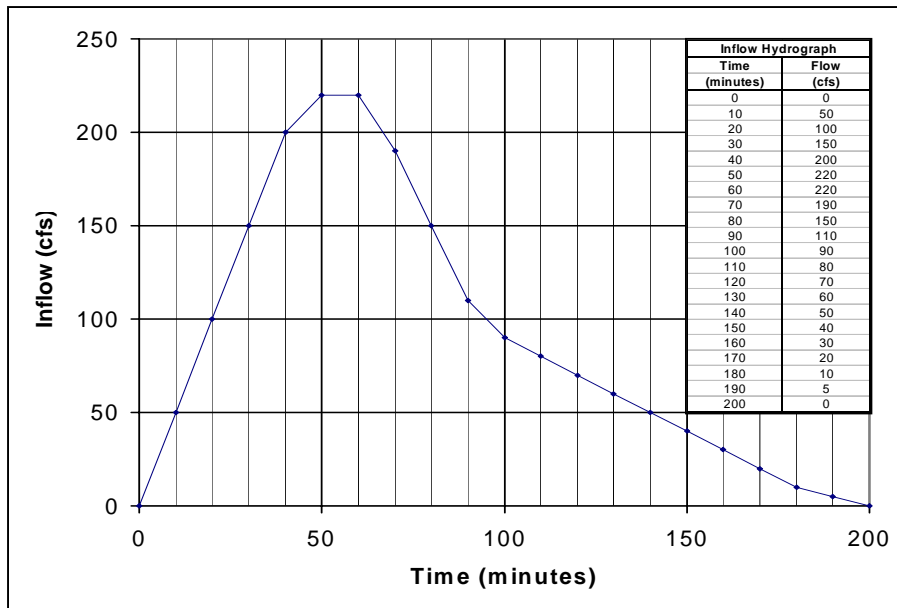


Figure 8.2
Inflow Hydrograph

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

$$Q = CLH^{1.5}$$

Equation 8.3
Weir Flow Equation

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

Equation 8.4
Orifice Flow Equation

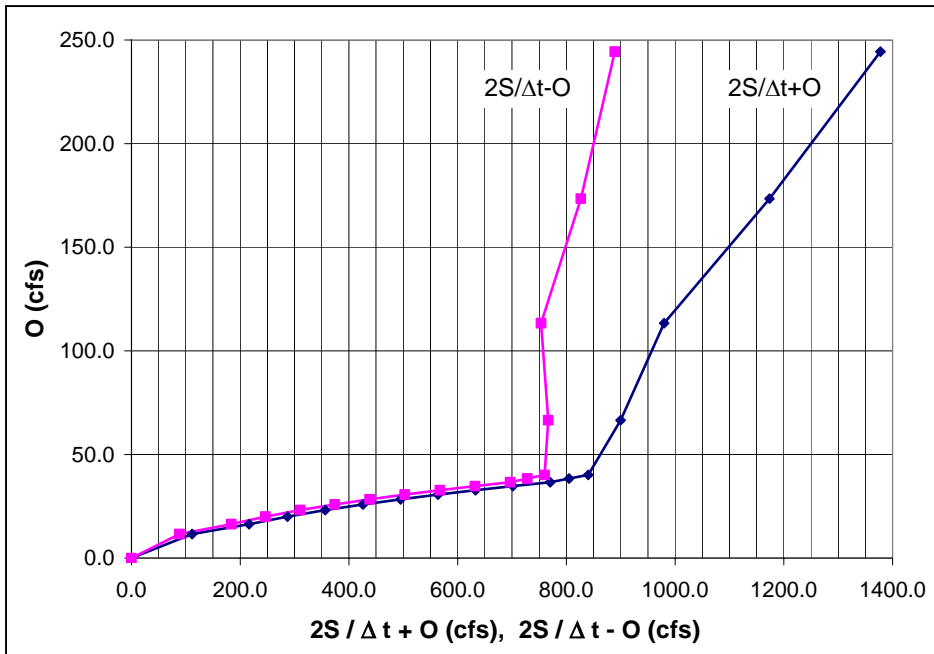
Where:

- Q = Outflow in cfs
- C = Weir Coefficient, 3.5
- L = Length of weir crest in feet
- H = Water surface elevation above weir in feet
- K = Orifice flow coefficient, 0.65
- A = Cross sectional area of orifice in ft²
- g = Gravitational acceleration in ft/sec²

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

Table 8.1Storage-Elevation and
Outflow-Elevation
Relationships

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

**Figure 8.3**

Storage-Indication Curve
Based on 10-minute
Time Interval

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

1. Determine the initial storage, inflow, and outflow conditions and the inflow at the first time step (S_n , I_n , O_n , and I_{n+1}). The inflow cannot be greater than the outflow for the first time step.
2. Use the storage-indication curve to determine the storage and outflow for the second time step (S_{n+1} and O_{n+1}).
3. Repeat the steps 1 and 2 until the outflow hydrograph is completed.

The initial values for this example are:

$$\begin{aligned}
 S_1 &= 0 \text{ ft}^3 \\
 I_1 &= 0 \text{ cfs} \\
 O_1 &= 0 \text{ cfs} \\
 I_2 &= 50 \text{ cfs} \\
 \Delta t &= (10 \text{ minutes}) \cdot (60 \text{ sec/minute}) = 600 \text{ sec}
 \end{aligned}$$

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

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

$$(0 + 50) + (0) = \left(\frac{2S_2}{\Delta t} + O_2 \right) = 50$$

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

$$O_2 = 5.2 \text{ cfs (from storage indication curve)}$$

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

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

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

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

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

The values for the second iteration are:

$$\begin{aligned} I_2 &= 50 \text{ cfs} \\ O_2 &= 5.2 \text{ cfs} \\ I_3 &= 100 \text{ cfs} \\ \Delta t &= (10 \text{ minutes}) * (60 \text{ sec/minute}) = 600 \text{ sec} \end{aligned}$$

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

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

$$O_3 = 15.2 \text{ cfs (from storage indication curve)}$$

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

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

Table 8.2

Outflow Hydrograph
Calculation Using
Modified Puls Method

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

Water Quality Hydrology

Water quality has been an important aspect of water resources planning and use for many years in Southern California¹. Regulations protect water quality and seek to limit pollution in part by requiring that new developments meet certain criteria for pollution prevention. Other regulations sometimes result in the retrofitting of existing storm water conveyances to reduce pollution of impaired receiving water bodies. Since problems with the quality of runoff can be associated with common rainfall events, smaller, more frequent storms must be addressed. This section discusses several of the issues that relate hydrology to water quality issues.

9.1 STANDARD URBAN STORMWATER MITIGATION PLANS (SUSMP)²

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

Development Type and Activities
<ul style="list-style-type: none"> • Single-family hillside homes • Residential development of ten or more units • Industrial/commercial developments with 1 acre or more of impervious surface area • Automotive service facilities • Retail gasoline outlets • Restaurants • Parking lots 5,000 ft² or more of surface area or with 25 or more parking spaces • Redevelopment projects in these categories that meet redevelopment thresholds • Locations within or directly adjacent to or discharging directly to an environmentally sensitive area • Fueling Areas • Equipment maintenance, washing and repair areas • Commercial/Industrial waste handling or storage • Outdoor hazardous material handling or storage • Outdoor manufacturing areas • Outdoor food handling or processing • Outdoor animal care, confinement, or slaughter • Outdoor horticultural activities

Table 9.1.1

Development or
Redevelopment Activities
Regulated by SUSMP

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

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

SUSMP regulations allow four methods of runoff volume calculation for BMPs that treat stormwater on a volumetric basis. The four methods allowed to calculate flow volume are:

1. The 85th percentile 24-hour runoff event determined as the maximized capture storm water volume for the area, from the formula recommended in Urban Runoff Quality Management, WEF Manual of Practice No. 23/ ASCE Manual of Practice No. 87, (1998).
2. The volume of annual runoff based on unit basin storage water quality volume, to achieve 80 percent or more volume treatment by the method recommended in California Stormwater Best Management Practices Handbook – Industrial/Commercial, (1993).
3. The volume of runoff produced from a 0.75-inch storm event, prior to its discharge to a storm water conveyance system.
4. The volume of runoff produced from a historical-record based reference 24-hour rainfall criterion for “treatment” (0.75 inch average for the county area) that achieves approximately the same reduction in pollutant loads as the 85th percentile 24-hour runoff event.

SUSMP regulations also allow three methods to calculate flow rates for BMPs that treat stormwater on a flow through basis. The three methods allowed to calculate flow rates are:

1. The flow of runoff produced from a rain event equal to at least 0.2 in/hr intensity.
2. The flow of runoff produced from a rain event equal to at least two times the 85th percentile hourly rainfall intensity for the County of Los Angeles.
3. The flow of runoff produced from a rain event that will result in treatment of the same portion of runoff as treated using volumetric standards above.

SUSMP also requires controlling peak flow discharges to provide stream channel and overbank flood protection. This requirement relies on hydrology based on flow design criteria selected by the local regulatory agency. Chapter 4 specifies the peak flow discharge criteria.

Many of the references for the SUSMP manual are available online. The following web addresses are links to the SUSMP Manual and a few of the references.

SUSMP Manual:

http://ladpw.org/WMD/npdes/SUSMP_MANUAL.pdf

CalTrans Storm Water Quality Manual:

<http://www.dot.ca.gov/hq/oppd/stormwtr/PPDG-stormwater-2002.pdf>

California Storm Water Best Management Practices Handbooks (2003) for Construction Activity, Municipal, Industrial/Commercial, and new development:

<http://www.cabmphandbooks.com/>

Start at the Source (1999) by Bay Area Stormwater Management Agencies Association:

www.mcstoppp.org/acrobat/StartattheSourceManual.pdf

9.2 TOTAL MAXIMUM DAILY LOADS (TMDL)

Total Maximum Daily Loads fall under Section 303 of the Federal Clean Water Act, which is a different section than the NPDES permit section. Impaired water bodies require reducing the pollutant discharge to a level that the water body can assimilate. The reduction could decrease the pollutant discharges to levels lower than required by an NPDES permit in order to meet the TMDL. TMDLs apply to both wastewater and stormwater discharges. Control of stormwater pollutant concentrations and loads requires implementing Best Management Practices (BMPs). TMDL requirements can relate to storms greater than storms required by SUSMP³.

Understanding and implementing the TMDL program mandated by the Clean Water Act (Section 303(d)) presents significant challenges for the responsible State Environmental Agencies. States develop TMDLs to determine how to reduce pollution from point sources and non-point sources so that the pollutant loads stay below the maximum specified in the TMDL. Point sources include industrial and municipal facilities that discharge to water bodies. Non-point sources of pollution include urban runoff, agriculture, forestry, septic systems, and air deposition⁴.

States are required to prioritize waters/watersheds for TMDL development. States compile this information in a list and submit the list to the United States Environmental Protection Agency for review and approval. The list is known as the 303(d) list of impaired waters. TMDLs are documents that describe a specific water quality attainment strategy for a water body and the related impairment identified on the 303(d) list. TMDLs may include more than one water body and more than one pollutant.

The TMDL defines specific measurable features that describe attainment of the relevant water quality standards. TMDLs include a description of the total allowable level of the pollutant(s) in question and allocation of allowable loads to individual sources or groups of sources of the pollutant(s) of concern⁵.

Each TMDL is for a specific water body and runoff mitigation can be represented by various hydrologic methods. For example, current trash TMDL regulations require that no man-made trash enter the water body at any time. However, hydrology studies for the trash TMDL use the 1-year, 1-hour storm to determine the flow rate that certain treatment systems must accommodate. The Santa Monica Bay Bacteria TMDL does not specify a design storm, but requires that bacteria levels remain below a certain concentration within the wave-wash of the bay. Figure 9.2.1 shows an example of low flow in a channel.



Figure 9.2.1
San Gabriel River
Low Flow Channel

Establishing TMDL hydrology requires data for rainfall, runoff, and water quality. Several agencies recognize the need to collect more water quality data, standardize collection methods, and create reporting methods that make this data more available^{6,7}. Defining hydrology methods used to design systems to meet TMDL standards requires understanding of the TMDL and water quality issues. As more data is collected and more TMDLs are established, standard TMDL hydrology procedures must be established.

9.3 BEST MANAGEMENT PRACTICES (BMPS)

Best Management Practices (BMPs) are actions and devices that improve or prevent the pollution of urban runoff and stormwater. The 2001 Los Angeles Municipal Stormwater Permit defines BMPs as "...methods, or practices, designed and selected to reduce or eliminate the discharge of pollutants to surface waters from point and non-point source discharges including storm water. BMPs include, but are not limited to, structural and nonstructural controls, and operation and maintenance procedures. BMPs can be applied before, during, and after pollution-producing activities."

BMPs can be proprietary or nonproprietary. Proprietary BMPs include patented and/or manufactured devices. Nonproprietary BMP designs are public domain and include detention basins, grassy drainage swales, catch basin stenciling, and public education.

Under the stormwater requirements of the federal Clean Water Act, stormwater quality must be improved to the "Maximum Extent Practical." The installation of BMPs is considered to meet that requirement.

In Phase II of the Federal Stormwater Permit process, the EPA breaks BMPs into six categories that deal with prevention and treatment of stormwater. The list is:

1. [Public education and outreach on stormwater impacts](#)
2. [Public involvement/participation](#)
3. [Illicit discharge detection and elimination](#)
4. [Construction site stormwater runoff control](#)
5. [Post-construction stormwater management in new development and redevelopment](#)
6. [Pollution prevention/good housekeeping for municipal operations](#)

Each of the six categories contains specific BMPs targeted to improve water quality. More information on the categories and BMPs is available through the EPA⁸. Figure 9.3.1 shows an example of a coastal wetland.



Figure 9.3.1
Coastal Wetland

¹ California Environmental Protection Agency. State Water Resources Board History. www.calepa.ca.gov/About/History01/

² Los Angeles County Department of Public Works. Development Planning for Stormwater Management:

A Manual for the Standard Urban Stormwater Mitigation Plan. September 2002.

³ Los Angeles County Department of Public Works. TMDL Information on Webpage. www.ladpw.org/general/faq/index.cfm?Action=searchResults

⁴ America's Clean Water Foundation and the Association of State and Interstate Water Pollution Control Administrators. www.tmdls.net

⁵ California Environmental Protection Agency, State Water Resources Control Board. <http://www.swrcb.ca.gov/tmdl/>

⁶ Committee on Assessment of Water Resources Research, National Research Council. Confronting the Nation's Water Problems: The Role of Research. The National Academies Press. Washington, D.C. 2001. <http://books.nap.edu/catalog/11031.html>

⁷ United States Government Accountability Office. Watershed Management: Better Coordination of Data

Collection Efforts Needed to Support Key Decisions. GAO-04-382.

www.gao.gov/cgi-bin/getrpt?GAO-04-382

- ⁸ National Menu of Best Management Practices for Stormwater Phase II. United States Environmental Protection Agency.

<http://cfpub.epa.gov/npdes/stormwater/menuofbmps/menu.cfm>

Hydrologic Data Requirements and Sources

Hydrologic studies require the use of mathematical models. A model is a representation of physical systems using equations. The parameters in these equations change to represent different hydrologic conditions. Hydrologic models have many forms and attempt to represent many different physical processes. The models used by the County of Los Angeles are lumped parameter models. This means that they consider the spatial variation of parameters only down to a certain level. Below this level, parameters are aggregated using an average.

Whether using hand or computer automated calculations, an important task of model preparation is gathering the input data. Section 10 provides information on obtaining various types of data required for hydrologic modeling.

10.1 REQUIRED DATA

Creating watershed models commonly requires the data types shown in Table 10.1.1. The following sections and chapters present the procedures for obtaining and using data for hydrologic modeling.

Required data	Description
Subarea Size	The surface area inside the subarea boundaries
Flow Path Length	Length of the conveyance between subarea collection points
Flow Path Slope	Slope of the flow path used for calculating the T_c
Conveyance Data	A description of the flow conveyance between subarea collection points (length, slope, width, roughness, etc.)
Soil Types	A soil classification identifying the hydrologic characteristics of the area's surface soils
Land Use / Imperviousness	A classification of impervious surface area based on development types within the subarea
Design Storm Definition	Each subarea has a unique design storm based on the location and the rainfall recurrence interval being modeled
Time of Concentration	The time required for runoff from the most hydrologically remote point in a subarea to reach the subarea collection point

Table 10.1.1

Required Watershed Data

10.2 DATA SOURCES

The Hydrology Manual is the official reference for developing design hydrology. There are several other resources available to provide data for hydrologic studies within the County of Los Angeles.

Hydrology Manual Appendices

The Hydrology Manual and Appendices contain the maps and charts necessary to create the hydrologic models.

Appendix A includes a chart and a table representing the unit hyetograph used to develop design storms for the County of Los Angeles. Section 5.2 discusses the development and application of this temporal rainfall distribution.

Appendix B contains USGS Quadrangle maps overlaid with spatial data for the entire county. These include overlays of the 50-year, 24-hour rainfall isohyets, soil type, and debris production area (DPA) zones. Soil type boundaries assist in determining the predominate soil type within a subarea and the appropriate runoff coefficient curve. DPA zones are provided for use

in the bulking process and to determine sediment production rates (see the Sedimentation Manual).

Appendix C contains soil names and characteristic information for the 179 soils defined for use with the Modified Rational Method. A soil identification table relates the soil numbers used by Public Works to the Natural Resources Conservation Service (NRCS) or Public Works assigned soil names. Graphs of the soil runoff coefficient curves represent the relationship between undeveloped runoff coefficients and rainfall intensity.

Appendix D contains a table of proportion impervious values for each of the SCAG land use types. While not shown in Appendix B, the land use patterns for the entire county are available as Geographic Information System (GIS) shapefiles.

Geographic Information System (GIS) and Electronic Data

Geographic Information Systems have an important role in current Public Works hydrologic studies. Most watershed characteristics vary by location. These spatial distributions lend themselves to GIS uses. The use of GIS allows the modeler to collect data quickly and accurately. Some computer programs integrate GIS and hydrologic modeling. These programs import and extract GIS data and provide this data to the hydrologic model for use in calculations.

Table 10.2.1 contains information on the principle GIS data available for hydrologic studies within the County of Los Angeles.

Georeferenced USGS Quadrangle map images are used as topographic maps for developing county design hydrology. These images can be opened in the GIS. These maps serve as the basis for delineation of watershed subareas and flow paths. These maps also serve as the basis for delineating the location of hydrologically important structures. Since the image is georeferenced, the resulting lines and subarea polygons have an associated length and area.

Aerial photographs can serve a similar function to map images. Photographs are useful because they can be used to identify various features such as roads, structures, land use, vegetative cover, and bodies of water. Aerial photos are also georeferenced images.

Digital Elevation Models (DEMs) and Triangular Irregular Networks (TINs) are used to find slopes for each subarea. Some programs automatically delineate watershed boundaries and stream channels using these data sources. DEMs are grids with an elevation assigned to each grid block. USGS DEMs are available in 10 meter and 30 meter resolutions for most of the county. The resolution refers to the size of each block in the grid. TINs replicate the ground surface using triangles formed by irregularly spaced points with known X, Y, and Z coordinates. DEMs and TINs are created from topographic survey data.

GIS Data Types	File Type
USGS Topographic "Quad" Maps	Image, typically "quad name".tif
Aerial Photographs	Image, typically *.jpg, *.tif
Digital Elevation Models (DEMs)	*.asc,
Triangulated Irregular Networks (TINs)	*.tin
LA County soil shapefile	soils_2004.shp
LA County land use shapefile	ladpw_landuse_2005.shp
LA County rainfall grid	lac50year24hr.asc, for the 50-year frequency

Table 10.2.1
GIS Data Types

The soil type is another attribute represented spatially as GIS data. A soil shapefile indicates the areas covered by each soil type. GIS models then assist in determining which soil type is predominate in a given subarea.

Land use data is available only as a GIS file. Each of the land use polygons represent a different development type and have an imperviousness value assigned. GIS based models can calculate and assign an area weighted composite imperviousness value to each subarea based on the land use data in the GIS files.

10.3 FIELD RECONNAISSANCE

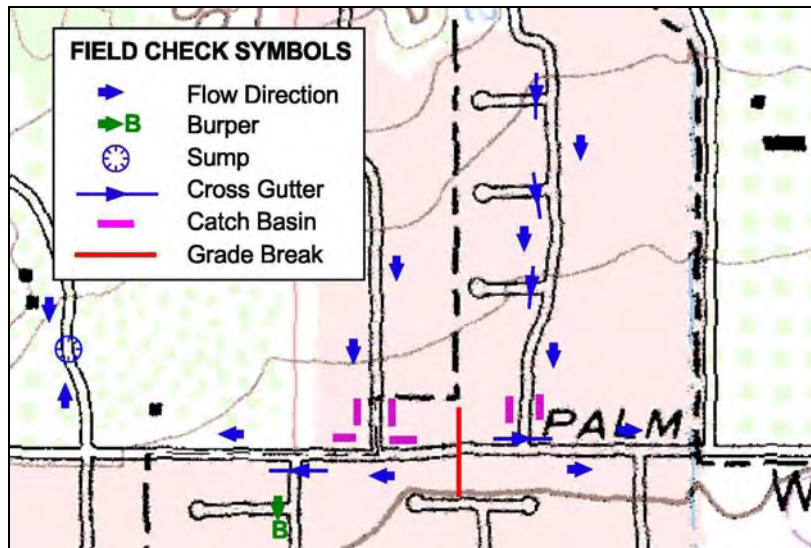
A field investigation is necessary for all design hydrology projects. The purpose is to gather information that might not be apparent from the data sources discussed in Section 10.2, and to confirm that the data gathered is

correct. The field investigation is also an opportunity to resolve any discrepancies present in other data collected. Assumptions such as land use and roughness of conveyances should also be verified.

In urban areas, a field investigation is required. Watershed boundaries in these areas are influenced greatly by man-made drainage features. Many of these features are not significant enough to be represented in elevation data or topographic maps and are not visible on aerial photos. The only way to determine the flow pattern in these cases is by field investigation.

While no standard procedure suits all projects, listed below are some basic field check guidelines as a starting point for urban studies.

1. Take a base map overlaid with the existing and proposed flow paths and conveyances identified in the initial research. Take an enlarged street map to use as your field check map.
2. Start your field check at the outlet of the drainage area. Crisscross the watershed heading upstream while preparing the map.
3. Note the following on the field check map:
 - Surface flow directions at every street intersection for both sides of the street; note the flow direction with an arrow pointing downhill. Show gutters, cross gutters, catch basins, burpers, sumps and grade breaks. Also, note any streets without curbs. Use the field check symbols in Figure 10.3.1.

**Figure 10.3.1**

Field Check Symbols Map

- Check the types of development, such as single family or industrial, in order to verify the percent impervious.
 - Check surface flow directions off property so that “frontage” along streets can be accounted for.
4. Get out of the car to investigate when there is uncertainty about flow directions.
 5. Take a carpenter’s level and place it in the gutter to determine the direction of flow on streets that are flat. Slopes are sometimes deceiving; use the level when in doubt.
 6. Before leaving the area, check the map and note any flow contradictions. Now is the time to go back and resolve them. After the field check, research any new issues that may have come up such as unexpected drain locations or flow patterns.

10.4 WATERSHED DELINEATION

A watershed is an area of land that drains to a given location. The process of delineating the watershed for a given point is an important part of creating a hydrologic model.

Watershed delineation requires a source of elevation data such as a topographic map. For the purposes of delineation, there are several important things to remember about topographic maps.

The contour lines are of equal elevation.

Water will follow a path perpendicular to the contour lines. All streams are perpendicular to the contour lines. Contour lines will generally form a “V” or an arrow pointing upstream where they cross streams.

Ridgelines are lines of high ground separating one watershed from another. Ridges may also appear as “V”s or arrows pointing down hill. A watershed boundary follows ridgelines. A drainage boundary will not intersect a stream or flow path except at the drainage area outlet.

See Figure 10.4.1 for examples of typical topographic forms.

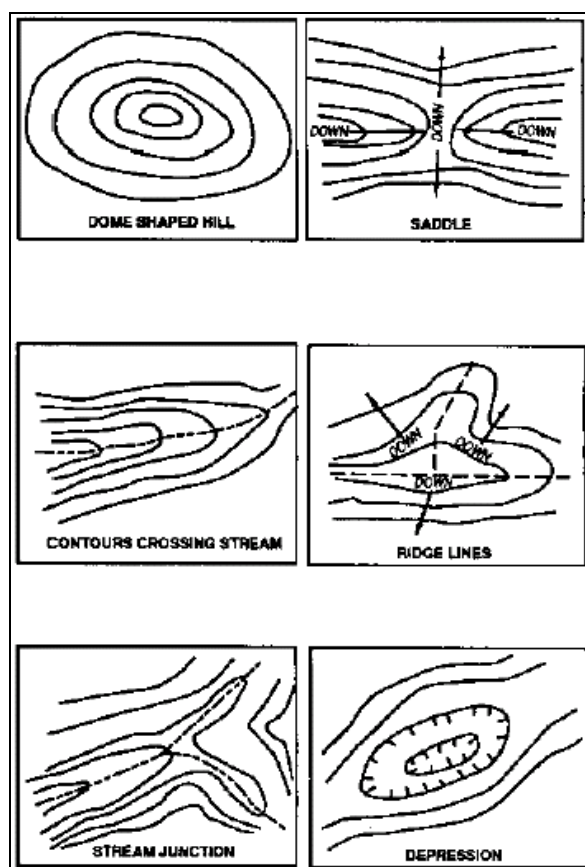


Figure 10.4.1

Typical Topographic Forms
Courtesy of Army Corps¹

As an example, consider the watershed delineation of Webber Canyon. Figure 10.4.2 shows the topographic map in the area surrounding Webber Canyon.

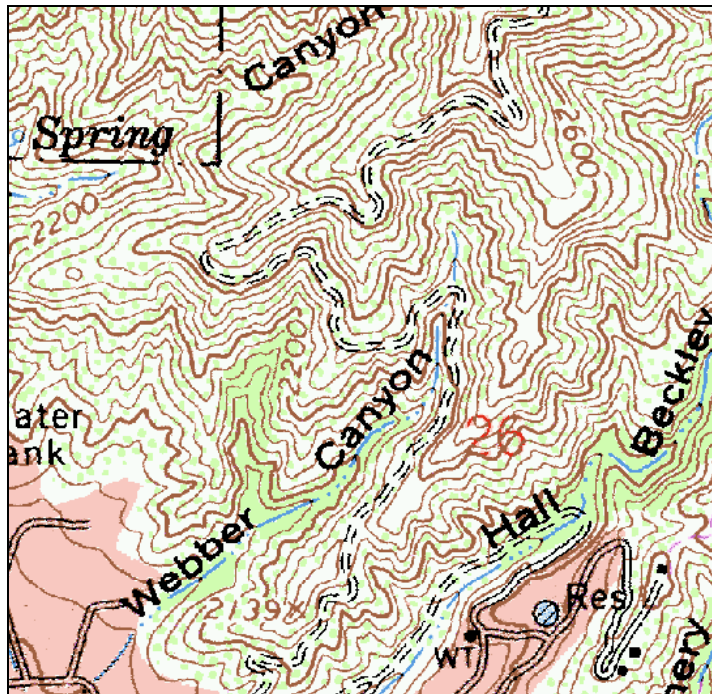


Figure 10.4.2

Topographic Map of
Webber Canyon

Consider a point at the mouth of Webber Canyon, just below the “W” in the word “Webber”. Webber Canyon and its tributaries upstream of this point comprise the watershed for the hydrology study.

Start by determining the outlet location where a flow rate value is needed, which for this example is location 1 in Figure 10.4.3. From this location, draw a line separating areas that contribute water to this location from areas that do not. Draw a line from the outlet point to the point on the adjacent contour. The line must be perpendicular to the contour line at the point where it crosses location 2.

Continue following and crossing the contours lines. Note that where the watershed boundary coincides with a sharp ridge, the line will be following

the “V”s. Where ridgelines meet, it is important to make sure that the areas enclosed within the boundary are part of the same stream network.

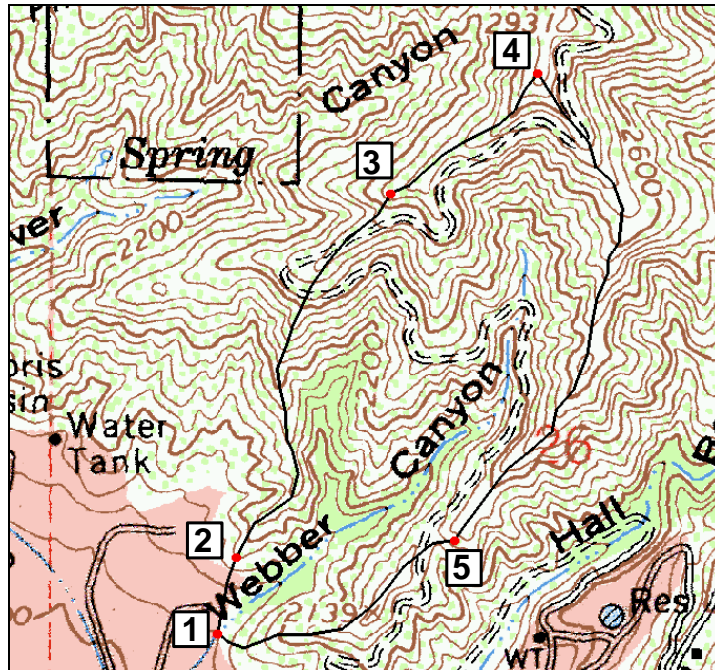


Figure 10.4.3

Webber Canyon Watershed
Delineation

At location 3, continue connecting “V”s past the intersection of ridgelines because the intersecting ridge separates Webber Canyon from a tributary. However, location 4 shows the ridgeline intersection that separates Webber Canyon from other watersheds. Stop at a point on the nose of the ridge and then continue back downhill following the intersecting ridge along the arc between locations 4 and 5. Continue down this ridge until you reach the watershed outlet once again at location 1. The area inside the boundary you have drawn is the watershed tributary to the chosen outlet point.

For modeling purposes, it is sometimes necessary to break a watershed up into smaller pieces called subareas. This is done by adding additional outlets upstream of the final outlet and following the same procedure. Outlets should be added at break points on the flow path. These points might include changes in slope, changes in conveyance, entry of laterals or external flows, or points where catch basins are needed. Subarea definition often depends on the purpose of the hydrologic study.

This simple example shows watershed delineation using only a topographic map. As discussed in Section 10.2, topographic maps are insufficient to define the drainage pattern in flat areas and urban areas where man-made drainage features must be considered. In these cases, watershed delineation must account for the actual drainage patterns and collection systems.

Determine the drainage area boundaries for the entire project watershed first. Then draw in the flow paths. The flow paths should include existing and proposed drains. Divide the drainage area into subareas by locating significant collection points in the watershed and delineating the subareas. Subarea delineation follows the same steps as watershed delineation.

10.5 COLLECTING SUBAREA DATA

A primary task in any hydrology study is gathering site specific data that will dictate the way runoff is produced. After delineating the watershed and subareas as described in Section 10.4, it is now possible to collect subarea data. Studies commonly require the lengths and slopes of flow paths and time of concentration paths, characteristic soil types, and percent imperviousness. Data collection is described in the following list.

1. Determine the subarea size using a planimeter or GIS.
2. Determine the length and average slope of conveyances. Draw a path that follows the main watercourse between the outlet of the upstream subarea and the next downstream subarea outlet. Measure the conveyance length using a scale or GIS. Determine the top and bottom elevation and calculate the slope of each conveyance length.
3. Determine the length and average slope of time of concentration paths. The procedure for T_C paths is the same as for conveyances. However, T_C paths are drawn from the furthest or most hydrologically remote point in a watershed subarea to the outlet. This is not necessarily the longest path distance but the one that would take the longest time for water to travel to the outlet.
4. Locate the soil type boundaries on the maps in Appendix B or using GIS, and determine the predominate soil type in the subarea. For the Modified Rational Method, the selected soil's runoff coefficient curve will be used to carry out all the necessary runoff calculations in the subarea.

5. Determine the type and extent of development in each subarea. Land use helps determine the amount of directly connected impervious area and hence the amount of rain that will runoff directly. The land use types have been assigned a percent imperviousness as shown in Appendix D. Each subarea requires an area-weighted average of percent imperviousness.

10.6 COLLECTING RAINFALL DATA

For simulation of a single event, rather than using the rainfall data from a real storm, a design storm is used. The design storm is described in Chapter 5. In order to account for the spatial variability of rainfall, the design storm assumes different magnitudes based on its location. Each subarea has a distinct, 50-year, 24-hour rainfall depth based on its position within the rainfall grid. The procedure for determining the average design rainfall is called the Isohyetal Method.

1. Locate the isohyetal lines on the quad maps from Appendix B and use the methods from Section 5.4 to assign each subarea an isohyetal depth for the 50-year, 24-hour event.
2. If the modeled event will be other than the 50-year, use the Rainfall Frequency Factors in Table 5.3.1 to convert this isohyetal depth for the desired frequency.
3. Produce the design hyetograph by multiplying each point on the unit hyetograph by the isohyetal depth.

For some dams it is necessary to evaluate runoff from standard design storms and the Probable Maximum Flood (PMF). Development of the design storm for the PMF must follow the procedures of Hydrometeorological Report (HMR) No. 59. In this case, other specific data about the watershed may need to be collected.

Chapter 5 describes the derivation of the design storm and the isohyetal maps from rain gage data collected in the county. Public Works' operates and maintains over 250 rain gages. These rain gages record rainfall amounts for durations from 5 minutes to 24 hours. Many of these rain gages have records that are greater than 50 years in length. Daily and annual rainfall amounts are available in the annual Public Works' Hydrologic Report

and at <http://www.ladpw.org/wrd/report/>. Intensities for other durations are available by contacting the Hydrologic Records Section.

Public Works collects rainfall data using non-recording and automatic recording rain gages. Non-recording gages collect rain and hold it in a container until it can be measured using a dipstick or graduated marking on the side of the collector. Volunteer observers typically read these gages daily at a specified time.

The automatic recording gages record the rainfall amounts for shorter time intervals. All of the Public Works' recording gages use tipping buckets to measure rainfall. The gages have a set of buckets that are alternately filled. When one of the buckets fills to a predefined amount, it tips. The other bucket then moves into the filling position. The frequency of the tipping allows the corresponding rainfall intensity to be calculated. This type of recording gage allows for very precise definition of a hyetograph. Most of the recording rain gages are connected to a central computer system using radio and satellite links so that rainfall amounts can be monitored in real-time. These gages are part of the Automatic Local Evaluation in Real-Time (ALERT) network. This network provides information for decision making during storm events.

10.7 CONVEYANCES

Conveyances are the links within a hydrologic model that simulate the flow of water through channel reaches. A hydrograph is specified at the top of a reach and a resulting outflow hydrograph is calculated at the bottom. Conveyance modeling is necessary due to the reduction of peak flow rates by attenuation and travel time. These processes affect the hydrograph at the downstream end of the conveyances.

The Modified Rational method uses six conveyance types: mountain, valley, street, circular pipe, rectangular channel, and trapezoidal channel. The types of conveyances between subarea collection points must be determined. The type of conveyance is important because water will flow much faster in a pipe than through a valley. Select the type that best characterizes the existing or planned conveyance. Several of the types require additional information about the dimensions and characteristics of the conveyance. The various conveyance types are described in detail in Section 7.3.

The length and slope of the conveyance between collection points are also important in determining the effects of hydrologic routing. The conveyance lengths are determined by measuring the flow path length using a scale or GIS. This length information is combined with the elevation data from a DEM or topographic map to determine the slope. For natural mountain and valley conveyances, the slope must be corrected using the slope correction curve, Figure 7.3.10.

Figure 10.7.1 shows water being conveyed on the streets of Lakewood after the 1950 storm season.



Figure 10.7.1

Streets of Lakewood Flooding
After 1950 Storm

¹ US Army Corps of Engineers, "Topographic Surveying", Manual 1111-1-1005. Washington D.C. August 31, 1994.