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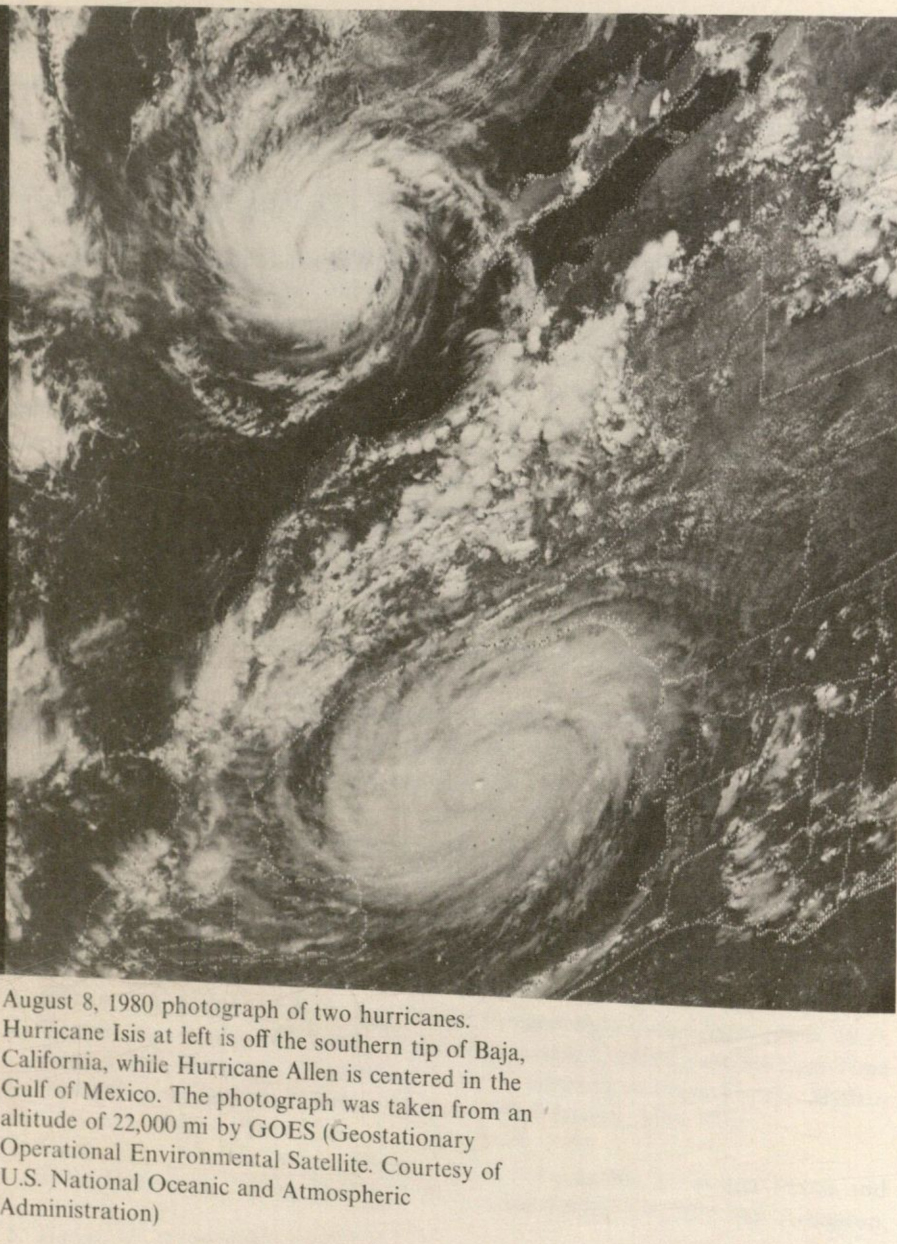


August 8, 1980 photograph of two hurricanes. Hurricane Isis at left is off the southern tip of Baja, California, while Hurricane Allen is centered in the Gulf of Mexico. The photograph was taken from an altitude of 22,000 mi by GOES (Geostationary Operational Environmental Satellite. Courtesy of U.S. National Oceanic and Atmospheric Administration)

Hydrology

2-1 General Considerations

Hydrology can broadly be defined as the technical field encompassing the occurrence of water *on*, *above*, or *within* the earth. However, in this text, as is generally common practice in engineering, hydrology is considered to deal primarily with surface-water hydrology, or water on the surface of the earth. Hydrogeology will be assumed to deal with the flow and occurrence of groundwater, and meteorology with the occurrence of water in the atmosphere. Because engineers have recognized that the design of major projects must consider unusual storms as design events, hydrometeorology has evolved in the last 30 years as a separate science dealing with the generation of rainfall by major storms including local effects of topography.

The science of hydrology has made major advancements during the twentieth century. Its major developments have occurred partly because of increased collection of data for precipitation, streamflow, and groundwater. Since 1960, developments in electronic instrumentation, satellite communication, and computerized analysis have made data not only easier to collect but also easier to analyze, tabulate, store, and publish. Improved analytical capabilities alone have greatly increased the mathematical sophistication of hydrologic analysis.

2-2 Project Needs

Designers of every project must consider the hazards and benefits of precipitation and streamflow. Since the earliest times, the human race has dwelled near streams or rivers because they provided a source of food, an avenue of transportation, and a water supply for domestic needs. Through trial and error, human beings learned of the hazards due to flooding and drought. As long as people were transient, little was lost by moving everything out of the path of a flood. Drought caused migrations in search of more dependable water supplies and may have contributed to the loss of entire civilizations, such as the one that flourished along the Tigris and Euphrates rivers well before the birth of Christ (3).

Today's comprehensive projects, such as major bridges, airports, factories, power plants, buildings, roads, hydroelectric plants, and irrigation projects, cannot readily be moved to avoid flood hazards, and water supplies that prove inadequate may bankrupt large-investment projects. Thus, it is important to be able to predict the stage of flooding in a river; the depth of rainfall to be expected at a site; or the volume of water a river can provide in a day, a month, or a year. To establish the floor elevation of an industrial facility to be built near a river, choosing a design flood and being able to calculate a maximum water-surface elevation associated with that flood are necessary. If the plant is constructed above that elevation, it will be safe against the design flood and lesser events.

Larger floods will produce higher flood elevations and, as a result, damages. Projecting those damages and assessing their economic impact on a project should be a vital part of selecting the design flood.

A simple irrigation project may involve pumping water from a river directly to a sprinkler system, which in turn provides water for crops. Since developing land for irrigation is expensive, it is critical to know how much water can be pumped from the river during a dry year. How dry is a dry year? Designing for the lowest river flow rate recorded implies that the project cannot function at the design condition for years in which the flow rate is lower. It is important in economic planning for a project to assess the losses that will occur during periods that are drier than the design condition. Associating those losses with their probability of occurrence provides the information needed to determine the project's probable income.

Thus, in hydrology, it is necessary to quantitatively assess streamflow and rainfall and, through statistical analysis, to estimate the probability of occurrence of larger and smaller events. The following sections will deal with each of these requirements.

2-3 Hydrologic Cycle

Hydrologic engineering differs from hydrology primarily in that an engineering application is implied. Thus, engineering considerations deal mostly with estimating, predicting, or forecasting precipitation or streamflow. By contrast, scientific hydrology deals primarily with the basic physical laws governing the elements of hydrology. Certainly, engineering hydrology must be fully aware of the scientific advancements in hydrology to properly apply those advancements in engineering practice. This chapter emphasizes the engineering applications of hydrology while attempting to use a scientific basis.

Hydrology deals intimately with the complex natural phenomena that govern our weather and climate. It is equally difficult to predict rainstorms and the resulting flood flows or extreme droughts and the resulting low flows. All the energy that drives the physical processes of hydrology comes from the sun. Differential warming and cooling of the atmosphere produces both large- and small-scale atmospheric motions. Since earth and water absorb energy at different rates and cool at different rates, the presence of land further complicates atmospheric movement and the occurrence of precipitation. Therefore, weather at any point on the earth exhibits characteristics that appear to be random but actually contain some cyclical components. Figure 2-1 shows the average monthly temperature for Glenwood Springs, Colorado, for the period 1959 through 1960. Figure 2-2 shows the average monthly precipitation for the same period. Note that temperature shows a regular seasonal variation with a 12-month period, but for precipitation, short-term fluctuations tend to mask any evidence of a regular period. Although the qualitative aspects of hydrology are

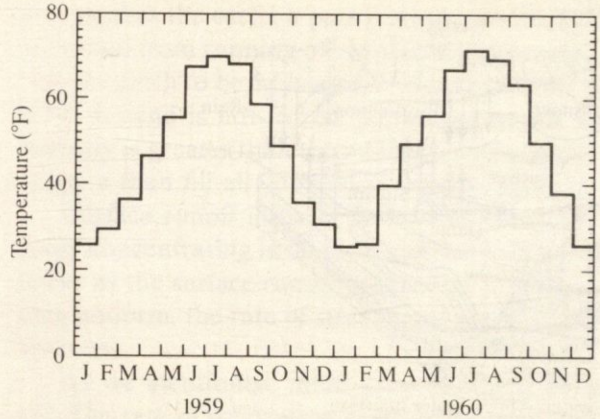


Figure 2-1 Average monthly temperature at Glenwood Springs, Colorado, 1959–1960

well understood, limitations of the current state of knowledge result in considerable uncertainty about quantitative analysis. Just as weatherforecasters cannot predict the occurrence of rain with 100% accuracy, so are engineering hydrologists' predictions uncertain. Because of this inherent uncertainty, hydrologists must learn to assess the quality of data and to use it reasonably.

The total volume of water on, below, and above the surface of the earth is a constant. Its movement through various phases, known as the hydrologic cycle, is shown in Fig. 2-3. In general, the total cycle involves evaporation from bodies of water, movement of water vapor through the atmosphere, precipitation, infiltration and runoff, groundwater flow, and streamflow, which com-

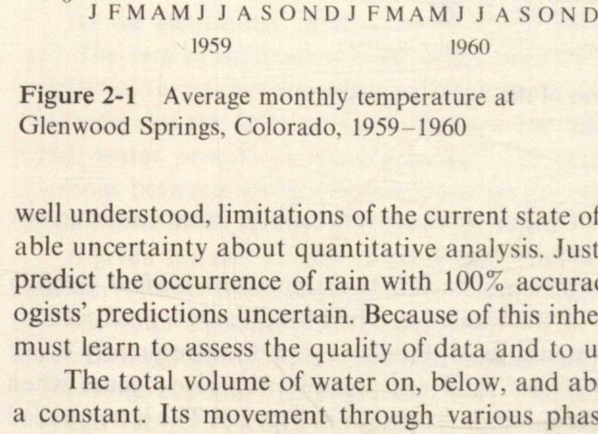


Figure 2-2 Average monthly precipitation at Glenwood Springs, Colorado, 1959–1960

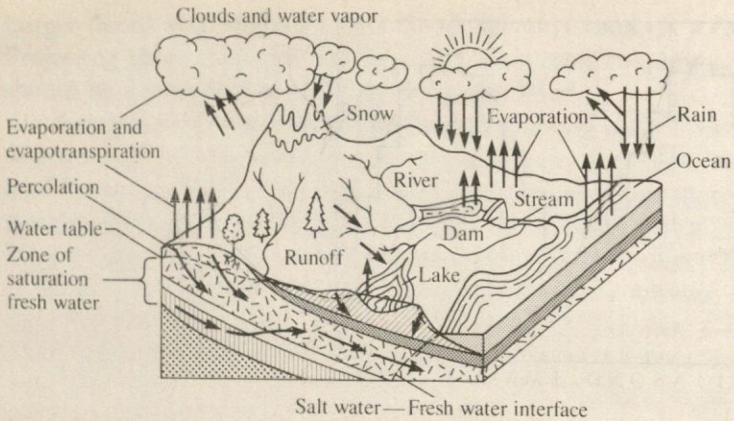


Figure 2-3 Schematic illustration of the hydrologic cycle

pletes the cycle. The volume of water in any given state or zone continuously varies, but the total volume remains constant.

Precipitation is produced when moisture-laden atmosphere is lifted, cooling the air and condensing some of the moisture. The lifting can be produced by local heating, producing the intense local thunderstorms that frequently occur on summer afternoons. Lifting with resultant precipitation also occurs when moving air passes over a mountain range, as shown in Fig. 2-3. The latter, called *orographic* lifting, results in zones of high annual precipitation on the windward side of mountains. For example, rainfall on the ocean side of the Olympic Mountain range along the western coast of the United States increases with elevation, reaching as much as 200 in. per year. On the lee (east) side of the mountains, the air descends and is warmed, increasing its capacity to hold moisture. Thus, central Washington, east of the Cascade Mountains, has a normal annual rainfall that averages less than 10 in.

The individual processes of the hydrologic cycle are actually complex, and today, neither analytical nor physical models are available to precisely describe them. Because of the complexity of the hydrologic cycle, it is often difficult for students to grasp the essence of it. To help you understand the basic principles of hydrology, a simplified conceptual model is shown in Fig. 2-4. The process shown in Fig. 2-3 is detailed and complex, whereas the conceptual model shown in Fig. 2-4 is simplified and, hence, amenable to approximate analytical simulation.

We will trace the hydrologic cycle beginning with precipitation. As precipitation occurs, part of it is intercepted by trees, grass, stones, or other cover, preventing it from reaching the earth. Part of this intercepted volume is retained and eventually evaporates. Other parts, such as snow on trees, will eventually fall to earth. The part that evaporates is referred to as *interception*. Of the rainfall that reaches the earth, a part is stored in depressions (*depression storage*) and is prevented from running off. Molecular forces and gravity cause water in contact with the earth to be *infiltrated*, or drawn into the openings between soil particles. If the ground is not frozen, *infiltration* begins immediately. When the rainfall intensity is greater than the infiltration rate and the volume of rainfall is enough to more than fill all depression storage, surface runoff begins.

Surface runoff initially flows over the ground surface as sheetflow, eventually concentrating in rivulets and finally in streams or rivers. Infiltration continues as the surface runoff progresses. If precipitation over the drainage basin were uniform, the rate of streamflow would always increase in the downstream direction.

As we mentioned, infiltration begins as soon as precipitation reaches the soil. The rate of infiltration is governed both by the amount and rate at which precipitation reaches the soil and by the ease with which water can penetrate the surface of the soil and move through the interstices between soil particles. Thus, water penetrates loosely packed soils easily. Because small interstitial openings between soil grains require large driving heads, water moves through sands much more readily than through clays.

A given soil can absorb and hold, through forces of molecular attraction, a specific volume of water known as the *field capacity*. Once this field capacity is filled, water moves directly through the soil. In that case, the ability for movement through the soil can limit the rate of infiltration. The volume of water infiltrating, beyond that required to fill the field capacity, moves downward through the soil as *deep percolation* and eventually reaches the *groundwater zone* (the *zone of saturation*). Water moves down gradient through the ground-

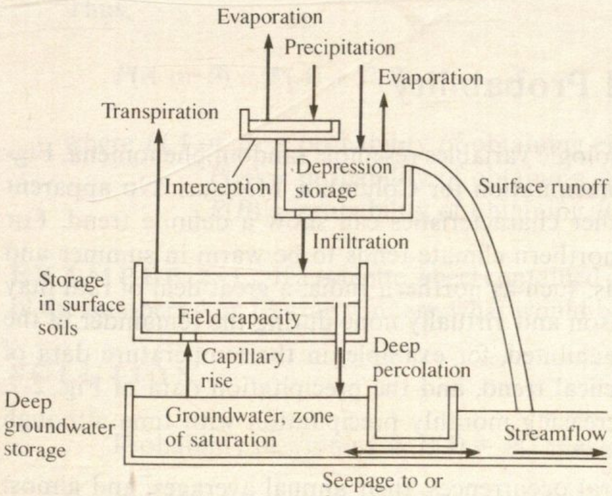


Figure 2-4 Conceptual diagram of hydrologic cycle

water zone eventually flowing into streams, and during periods of zero surface runoff, provides a *baseflow* for the stream. Water may leave the groundwater zone to enter the *root zone* by capillary action. The movement of water through the groundwater zone is much slower than the movement of surface runoff.

That part of precipitation greater in volume than the combined interception, depression-storage, evaporation, and infiltration volumes becomes surface runoff and is known as the *rainfall excess*.

The volume and rate of surface runoff is of great interest in engineering design and water-resources planning. Many different approaches have therefore evolved to estimate runoff rate or volume as a result of a given amount of precipitation. The description of the hydrologic cycle briefly points out the complex considerations affecting surface runoff. A small drainage area, such as a parking lot, may have uniform characteristics (particularly if it is paved) such as depression storage, slope, and texture of the soil surface. However, a large natural basin can be expected to exhibit great areal variations in vegetal cover, soil type, slope and land use. Thus, simple rainfall-runoff relationships such as the rational equation

$$Q_p = CIA \tag{2-1}$$

can be confidently applied only to small drainage areas. In Eq. (2-1), Q_p is peak rate of runoff, I is intensity of rainfall, A is the drainage area, and C is a dimensionless empirical coefficient that is primarily dependent on texture and permeability of the area's surface. The units of Q_p are a volume rate of flow dependent on the units used for A and I. Table 2-9, page 67, lists values of C applicable for various surfaces. We will apply this and other rainfall-runoff methods later.

2-4 Statistics and Probability

In many ways, hydrologic variables resemble random phenomena. Figure 2-5 shows the annual precipitation for Columbia, Missouri. No apparent trend exists in the data. Other characteristics can show a definite trend. For example, temperature in a northern climate tends to be warm in summer and cold in winter. In other areas, such as northern India, a great deal of rain may fall during the monsoon season and virtually none during the remainder of the year. Apparent trends are exhibited, for example, in the temperature data of Fig. 2-1, which shows a cyclical trend, and the precipitation data of Fig. 2-2, which appears to show decreasing monthly precipitation with time although fluctuation is profound.

Extremes in these natural occurrences, their annual averages, and almost any other measure of magnitude vary in ways that appear to be random and can often be considered random. Techniques of probability and statistics are used to analyze these random events. In this section, we discuss these statistical

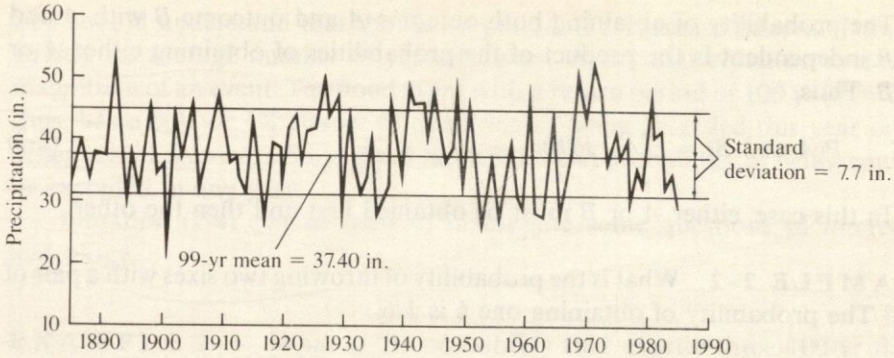


Figure 2-5 Annual precipitation at Columbia, Missouri 1887–1986

tools but neglect most mathematical derivations. You will likely have seen these tools in other contexts.*

The concept of probability is paramount in the field of hydrology. It is convenient to relate probability to a familiar procedure such as tossing a coin, throwing dice, or drawing numbered balls from a hat, all of which are random procedures (if the game is honest). The numerical value of probability ranges from 0 to 1, with 1 being absolute certainty that the event will happen and 0 being absolute certainty that it will not. We will now set forth several relationships regarding probability and then illustrate each with an example.

I. The probability of obtaining either outcome A or B, with A and B independent and mutually exclusive, is the sum of the probability of obtaining each. Thus,

$$P(A \text{ or } B) = P(A) + P(B) \tag{2-2}$$

where $P(A \text{ or } B)$ = probability of obtaining either A or B
$P(A)$ = probability of obtaining A
$P(B)$ = probability of obtaining B

EXAMPLE 2-1 If a roulette wheel contained only 50 numbers, and if either of the numbers 4 or 15 were winners, what would be the probability of winning?

SOLUTION

$$\text{Probability of winning} = P(4) + P(15) = \frac{1}{50} + \frac{1}{50} = \frac{1}{25} = 0.04 = 4\%$$

■

* The serious student can find the theoretical developments in most textbooks on statistics or in textbooks dealing primarily with the statistical methods in hydrologic analysis (8).

II. The probability of obtaining both outcome A and outcome B with A and B independent is the product of the probabilities of obtaining either A or B. Thus,

$$P(A \text{ and } B) = P(A) \cdot P(B) \tag{2-3}$$

In this case, either A or B must be obtained first and then the other.

EXAMPLE 2-2 What is the probability of throwing two sixes with a pair of dice? The probability of obtaining one 6 is 1/6.

SOLUTION The probability of obtaining two sixes is

$$P = \frac{1}{6} \cdot \frac{1}{6} = \frac{1}{36} = 0.028 = 2.8\%$$

■

III. The probability P of having exactly K occurrences in n trials is

$$P(K \text{ in } n) = \frac{n!}{K!(n-K)!} p^K (1-p)^{n-K} \tag{2-4}$$

where $n!$ represents factorial of n, and p is the probability of success in any one attempt.

EXAMPLE 2-3 What is the probability of throwing three sixes in eight throws using only one of a pair of dice?

SOLUTION The probability of throwing a six with only one of the dice is 1/6. Thus,

$$P(3 \text{ in } 8) = \frac{8!}{3!5!} \left(\frac{1}{6}\right)^3 \left(1 - \frac{1}{6}\right)^{8-3}$$

or
$$P(3 \text{ in } 8) = \frac{6 \cdot 7 \cdot 8}{1 \cdot 2 \cdot 3} \left(\frac{1}{6}\right)^3 \left(\frac{5}{6}\right)^5 = 0.104 = 10.4\%$$

■

Probability in hydrology is conveniently referred to in terms of the average *return period* of a particular event. Thus, a rainstorm with an average return period of 100 yr has a probability of occurring or being exceeded once in 100 yr or $1/100 = 0.01$. By average return period, we mean the average number of years between occurrences of this magnitude. Return period T is thus related to probability by the equation

$$P = \frac{1}{T} \tag{2-5}$$

You should understand that the return period or recurrence interval T implies strictly an average number of years between events that equal or exceed the magnitude of an event. The flood event with a return period of 100 yr ($P = 0.01$), thus, has a 0.01 or 1% chance of occurring or being exceeded this year or any other year. Likewise, a 50-yr flood has a 0.02 (2%) probability of being equalled or exceeded in any year.

Equation (2-4) can be used to investigate some questions of interest in hydrology.

EXAMPLE 2-4 What is the probability that exactly three 100-yr floods will occur during the 50-yr expected life of a particular highway bridge?

SOLUTION For the 100-yr flood: $p = 1/100 = 0.01$. In Eq. (2-4), n is now the 50-yr design life. Thus,

$$P(3 \text{ in } 50) = \frac{50!}{3!(50-3)!}(0.01)^3(1-0.01)^{47}$$

or $$P(3 \text{ in } 50) = 0.012 = 1.2\%$$

Thus, if the bridge is designed to be safe during floods up to the 100-yr event (the *design flood*), there is a 1.2% chance that the 100-yr flood will be equalled or exceeded exactly three times during *the design life of the bridge.* ■

A project is normally designed to be safe for floods of less than or equal to some magnitude, known as the *design flood.* The question posed in Example 2-4 is not the one of real interest in hydrologic design. The important question is: What is the probability that the design flood will be equalled or exceeded at least once during the design life of the project? We can compute the answer with the following equation, which follows directly from Eq. (2-4):

$$P(\text{failure in } n \text{ years}) = 1 - (1 - p)^n \tag{2-6}$$

EXAMPLE 2-5 What is the probability that a 100-yr design flood will be exceeded at least once in a 50-yr project life?

SOLUTION Using Eq. (2-5):

$$P = \frac{1}{100} = 0.01$$

Using Eq. (2-6):

$$P = 1 - (1 - 0.01)^{50} = 0.39 = 39\%$$

Thus, if an engineer designs a dam to be safe during floods up to a 100-yr event, there is a 39% chance that this design flood will be exceeded during a 50-yr design life. If the dam fails during a flood greater than the 100-yr event, there is a 39% chance that the *dam will fail during its design life* as a result of flooding. ■

Example 2-5 briefly presents the concept of probability of failure. It is interesting to compute a set of design events that must be designed for in order to realize a given risk (probability of failure) during the planned project life. Table 2-1 contains values computed using Eq. (2-6) for the required return period of a design event if that design event is not to be exceeded by more than a chosen probability (risk) in a given project life.

Table 2-1 shows that if the permissible risk of failure is 1%, a project with a 50-yr design life must be designed for an event that has an average recurrence interval of 4975 yr. To find this value we substitute the risk 0.01 into Eq. (2-6) to obtain.

$$0.01 = 1 - (1 - p)^{50}$$

and applying the reciprocal yields

$$p = 4975$$

For reasons of comparison, it is interesting to note that large dams and nuclear power plants are designed to be safe during a probable maximum flood, which is thought to have an exceedance probability in the order of 10^{-6} (see Tables 2-14 and 2-15, pages 88 and 93).

The design life of a project is primarily an economic consideration. At the end of its design life, a project would presumably have zero worth. The actual life of most structures is often much longer than their design life. However, the cost and benefits accrued during the design life balance, and in theory, the project could be abandoned at the end of its design life without loss. This practice

Table 2-1 Required Design Return Interval as a Function of Acceptable Risk

Acceptable Risk of Failure (P)	Planned Useful Project Life (n) (yr)			
	1	25	50	100
0.01	100	2487	4975	9950
0.25	4	87	174	348
0.39	2.6	51	100	203
0.50	2	37	73	145
0.75	1.3	19	37	73
0.99	1.01	6	11	22

is much more common for buildings than for major structures such as hydroelectric plants, bridges, or dams.

Flood Frequency Analysis

The tools of statistics and probability are particularly useful in hydrology. The data on precipitation kept by the U.S. National Weather Service and those on streamflow kept by the U.S. Geological Survey are examples of enormous databases that are of little use unless the data are refined, transformed, and summarized by statistical analysis.

Mean daily, mean monthly, and mean annual values are only three parameters that are continually computed as data are collected. Moreover, information is frequently desired on the return period of particular flows and on the magnitude of a flow that is equalled or exceeded with a given probability (a design flood).

In making a frequency analysis of floods, the annual peak flow is selected for each year of the recorded flows of the stream of interest. The series of flows as shown in Table 2-2 was measured on the Ok Ma River in Papua, New Guinea.

Table 2-2 Peak Flows Above 250 m^3/s for the Ok Ma River in Papua, New Guinea

Date	Peak Flow Rate (m^3/s)	Date	Peak Flow Rate (m^3/s)
11/11/76	253	7/10	353
11/25	296	7/23	560
12/3	353	7/29	393
1/14/77	250	8/1	436
1/30	366	8/5	287
2/4	436	10/21/80	408
2/28	290	1/21/81	303
4/8	505	2/15	290
4/15	273	4/19	387
4/25	560	6/2	285
5/4	273	9/1	260
6/4	766	9/11	509
6/8	380	11/18	327
6/18	301	12/24	300
6/20	340	1/28/82	254
6/27	400	8/13	276
6/30	736	5/2/83	350
7/6	347		

The series of annual maximum flows, so selected, is called, reasonably enough, an *annual series.* In this particular case, all flows greater than 250 m^3/s, and not just the maximum for each year, were listed, which is called a *partial duration series.* To analyze these values statistically, we must compute the following parameters where Q_i is the annual peak flow for year i, and N is the number of years of recorded flows.

$$\text{Mean} = \overline{M} = \frac{\sum_{i=1}^{N} Q_i}{N} \tag{2-7}$$

$$\text{Standard deviation} = S = \sqrt{\frac{\sum_{i=1}^{N} (Q_i - \overline{M})^2}{N-1}} \tag{2-8}$$

$$\text{Skew} = G = \frac{\sum_{i=1}^{N} (Q_i - \overline{M})^3}{N} \tag{2-9}$$

These three parameters are, respectively, measures of the first moment of the flows about the origin, and the second and third moments of the flows about the mean. Figure 2-6 illustrates their properties. The mean $\overline{M}$ is, of course, a measure of the location of the centroid of the distribution of flows.

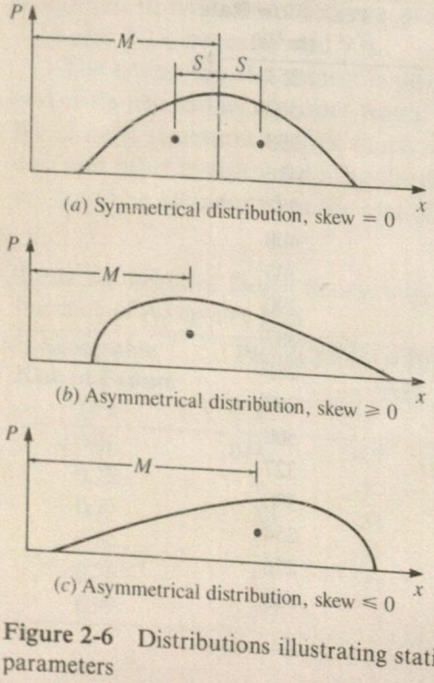


Figure 2-6 Distributions illustrating statistical parameters

The *standard deviation* S is a measure of the way in which the flows are distributed about the mean, and the *skew* G is a measure of the degree of symmetry of the distribution. Figure 2-6*b* illustrates a distribution where most values are located to the right side of the mean (a positive skew), whereas Fig. 2-6*c* illustrates the opposite (a negative skew).

EXAMPLE 2-6 Determine the mean, standard deviation, and skew of the annual peak flows shown in Table 2-2.

SOLUTION In the accompanying table the largest flow for each year has been selected. There are only six years of record, since 1978 and 1979 are missing.

Year	Largest Flow (ft^3/s)	Order (m)	Apparent Probability ($m/N+1$)
1976	353	4	0.57
1977	766	1	0.14
1980	408	3	0.43
1981	509	2	0.28
1982	276	6	0.86
1983	350	5	0.71
sum =	2662		

Thus, $\bar{M} = 2662/6 = 444$ m^3/s. In computing the standard deviation, Eq. (2-8) can be rearranged to give the following equation, which we can compute with a numerical accuracy greater than is inherent in Eq. (2-8).

$$S = \sqrt{\frac{\sum_{i=1}^{N} (Q_i^2) - \left(\sum_{i=1}^{N} Q_i\right)^2 \Big/ N}{N-1}} \tag{2-10}$$

$$= \sqrt{\frac{1335586 - 7086244/6}{5}} = 176 \text{ m}^3\text{/s}$$

Instead of calculating the skew directly, a coefficient of skewness is used more frequently and is equal to the skew divided by the cube of the standard deviation. Equation (2-9) can be rearranged to give the following:

$$g = \left[\frac{N^2\left(\sum_{i=1}^{N} Q_i^3\right) - 3N\sum_{i=1}^{N} Q_i \sum_{i=1}^{N} Q_i^2 + 2\left(\sum_{i=1}^{N} Q_i\right)^3}{N(N-1)(N-2)S^3}\right] \tag{2-11}$$

$$= \left[\frac{36(757131190) - 3(6)(2662)(1335586) + 2(1.8863582 \times 10^{10})}{6(5)(4)(176)^3}\right]$$

$$= 1.51$$

■

Example 2-6 is a case where the record of flows is very short. A short record of data like this normally does not define a distribution well, and skew particularly is not defined well for a short record. In general, the skew should not be estimated for records much less than 100 yr in length (2). In practice, short records are encountered often, and care must always be used in attempting to extrapolate information calculated from them.

Several theoretical distributions have been used in attempts to fit curves to distributions of peak flows. One such curve is the normal distribution, the symmetrical bell-shaped curve used in many familiar fields. One method used to describe any distribution uses the following equation:

$$Q = \overline{M} + KS \tag{2-12}$$

where $\overline{M}$ and S are the mean and standard deviation as defined in Eqs. (2-7) and (2-8), and K is a factor for which values are given in Table 2-3 for a normal distribution.

A second distribution frequently used for extreme values is the Pearson Type III. Values of K for use in the Pearson Type III distribution are shown in Table 2-4.

The normal distribution is symmetrical about the mean, and thus, the skew and the coefficient of skew are zero for any normal distribution. However, the

Table 2-3 Values of K to be Used in Eq. 2-12 With the Normal Distribution (2)

Exceedance* Probability	K	Exceedance* Probability	K
0.0001	3.719	0.500	0.000
0.0005	3.291	0.550	−0.126
0.001	3.090	0.600	−0.253
0.005	2.576	0.650	−0.385
0.010	2.326	0.700	−0.524
0.025	1.960	0.750	−0.674
0.050	1.645	0.800	−0.842
0.100	1.282	0.850	−1.036
0.150	1.036	0.900	−1.282
0.200	0.842	0.950	−1.645
0.250	0.674	0.975	−1.960
0.300	0.524	0.990	−2.326
0.350	0.385	0.995	−2.576
0.400	0.253	0.999	−3.090
0.450	0.126	0.9995	−3.291
0.500	0.000	0.9999	−3.719

* Exceedance probability is the probability that an event will be equaled or exceeded, and is equal to $1/T$ where T is the return period.

Table 2-4 Values of K to be Used With the Pearson Type III Distribution (2)

Skew Coefficient (g)	Exceedance Probability 0.99	0.90	0.50	0.10	0.02	0.01
3.0	−0.667	−0.660	−0.396	1.180	3.152	4.051
2.5	−0.799	−0.771	−0.360	1.250	3.048	3.845
2.0	−0.990	−0.895	−0.307	1.302	2.912	3.605
1.5	−1.256	−1.018	−0.240	1.333	2.743	3.330
1.2	−1.449	−1.086	−0.195	1.340	2.626	3.149
1.0	−1.588	−1.128	−0.164	1.340	2.542	3.022
0.9	−1.660	−1.147	−0.148	1.339	2.498	2.957
0.8	−1.733	−1.166	−0.132	1.336	2.453	2.891
0.7	−1.806	−1.183	−0.116	1.333	2.407	2.824
0.6	−1.880	−1.200	−0.099	1.328	2.359	2.755
0.5	−1.955	−1.216	−0.083	1.323	2.311	2.686
0.4	−2.029	−1.231	−0.066	1.317	2.261	2.615
0.3	−2.104	−1.245	−0.050	1.309	2.211	2.544
0.2	−2.178	−1.258	−0.033	1.301	2.159	2.472
0.1	−2.252	−1.270	−0.017	1.292	2.107	2.400
0.0	−2.326	−1.282	0.000	1.282	2.054	2.326
−0.1	−2.400	−1.292	0.017	1.270	2.000	2.252
−0.2	−2.472	−1.301	0.033	1.258	1.945	2.178
−0.3	−2.544	−1.309	0.050	1.245	1.890	2.104
−0.4	−2.615	−1.317	0.066	1.231	1.834	2.029
−0.5	−2.686	−1.323	0.083	1.216	1.777	1.955
−0.6	−2.755	−1.328	0.099	1.200	1.720	1.880
−0.7	−2.824	−1.333	0.116	1.183	1.663	1.806
−0.8	−2.891	−1.336	0.132	1.166	1.606	1.733
−0.9	−2.957	−1.339	0.148	1.147	1.549	1.660
−1.0	−3.022	−1.340	0.164	1.128	1.492	1.588
−1.2	−3.149	−1.340	0.195	1.086	1.379	1.449
−1.5	−3.330	−1.333	0.240	1.018	1.217	1.256
−2.0	−3.605	−1.302	0.307	0.895	0.980	0.990
−2.5	−3.845	−1.250	0.360	0.771	0.798	0.799
−3.0	−4.051	−1.180	0.396	0.660	0.666	0.667

Pearson Type III distribution can be used to fit a skewed distribution, and the values of K in Table 2-4 are seen to vary with the value of the skew coefficient. In using the Pearson Type III distribution, logarithms of the discharge are used as the argument while the discharges are used with the normal distribution.

These distributions can be readily demonstrated by example.

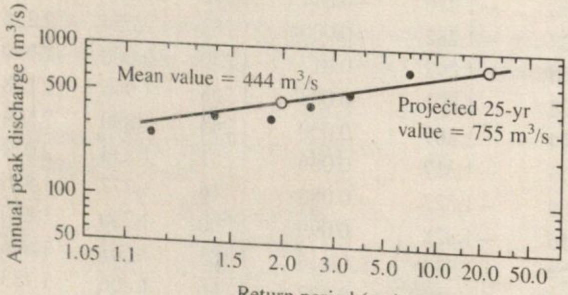
EXAMPLE 2-7 Perform a frequency analysis for the annual maximum flows as given in the table for Example 2-6.

SOLUTION In order to perform a frequency analysis it is necessary to select the maximum flow for each year. The table for Example 2-6, page 33, shows the maximum flow for each year as selected from Table 2-2. The third column of the table shows the order number (m) of the flow. The largest flow has order $m = 1$, whereas the smallest has order number $m = 6$. The order numbers are used to calculate the apparent probabilities of occurrence of each annual peak flow. The largest flow appears to have been equalled or exceeded once six years. However, because of the short record, that probability may be erroneous. A relationship used to calculate apparent probability is

$$p = \frac{m}{N+1} \tag{2-13}$$

where m is the order number, and N is the number of years of record. On the average, this relationship tends to be correct (2).

Applying Eq. (2-13) yields the values shown in column 4. The apparent probabilities computed with Eq. (2-13) are the probabilities that each flow in column 2 will be equalled or exceeded in any year.



The values shown in column 4 are plotted in the accompanying figure. Probability paper (available commercially) has been used for plotting. ■

EXAMPLE 2-8 Using the annual flows for the Ok Ma River in New Guinea as given in Table 2-2, and assuming that the flows follow a normal distribution, compute the peak flow that can be equalled or exceeded only once in 25 yr.

SOLUTION In Example 2-6 the mean was calculated as $\overline{M} = 444$ m^3/s and the standard deviation S was 176. The mean's return period is approximately 2 years. The value has been plotted in the figure for Example 2-7. For a 25-yr event, the probability of exceedance is

$$P = \frac{1}{25} = 0.04$$

Using Table 2-3, $K = 1.771$ is obtained by interpolation. Thus from Eq. (2-12),

$$Q_{25} = 444 + 1.771(176)$$
$$= 756 \text{ m}^3/\text{s}$$

Note: The skew was assumed to be zero. The mean and the 25-yr value have been plotted in the figure for Example 2-7. The straight line drawn between the mean and the 25-yr values could be used to read apparent peak flows for any given return period. ■

Risk

As we mentioned, civil engineering design, flood plain delineation, and planning all require the determination of a design flood of a given return interval. The adoption of any particular recurrence interval automatically assumes a particular degree of risk. This degree of risk is not always apparent to engineering planners or designers because many design floods have been fixed far enough in the past that the reasons for those choices are no longer well known. In addition, conditions that reflect hazard, such as population and development, may have changed in the interim. Often, conditions downstream from a dam change drastically with time. Because of reduced frequency and magnitudes of downstream flooding, realized after construction of a dam, a downstream valley that once had few residents and little economic development may experience substantial development and investment after construction of the dam. Thus, at a later date, the area downstream from the dam may need to be reclassified as "high hazard and substantial economic loss," whereas the earlier risk would have been low in both categories.

Extrapolation of Data

Extrapolation of peak floods to return intervals well beyond the period of record is a common requirement because of the shortness of streamflow records on many streams. Extrapolation of these records to a 100-yr flood or larger should be done with much caution and a full appreciation for the uncertainties involved. Figure 2-7 illustrates a historical case. Values in Fig. 2-7 are plotted on special probability paper. The record of annual peak flows from 1901 to 1953 appeared to be readily acceptable for extrapolation to higher return intervals. However, the flood of 1954 is an obvious "outlier" in the statistical series since it plots as a point so far above any smooth curve that might be drawn through the earlier values. Although the apparent recurrence interval of the 1954 flood is only 55 yr, its appearance dramatically changes the series. If either the normal distribution or the Pearson Type III distribution is fit to the data,

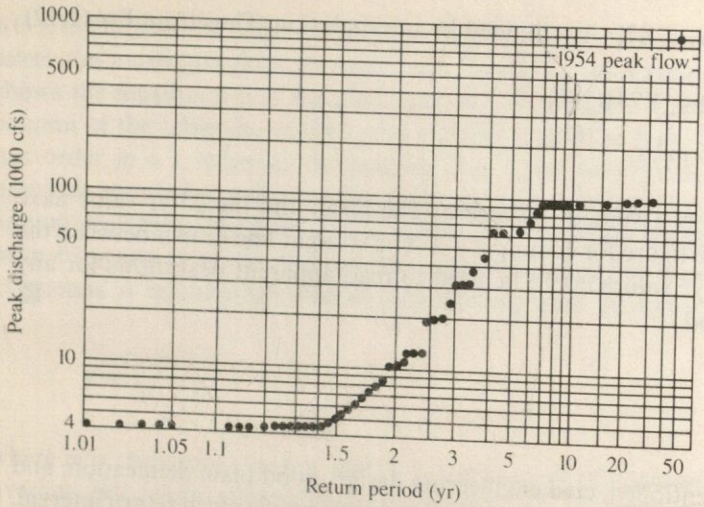


Figure 2-7 Annual maximum peak discharges for Pecos River near Comstock, Texas, 1901–1954 (16)

the prediction of the 100-yr flood peak, including the 1954 event, will be drastically different from what it will be if the 1954 event were excluded. This particular example should be remembered when fitting a distribution to a short record. Statistical distributions have no recognition of the physical actions that produce the particular events. They are "simple" curves that fit the available historical data reasonably well. One new year of data could easily change the theoretical distribution and drastically change predicted results, particularly if the record is short.

2-5 Precipitation

Precipitation is usually the independent variable in engineering hydrology. Most often we are interested in calculating a rate or volume of runoff that can be expected to occur because of a given amount of rainfall. It now seems terribly obvious that streamflow is the direct result of precipitation, but in 1674 when Pierre Perrault published his book *De L'Origine des Fontaines*, that concept was quite controversial (17). Perrault showed, with the results of his own rainfall measurements, that precipitation was indeed enough to explain the total streamflow of rivers. Measurement of precipitation and streamflow rates provides the basic data on which modern hydrology is based.

Occurrence of Precipitation

Precipitation is quantitatively described in terms of duration (minutes or hours) and depth (inches or millimeters). Intensity is the rate of rainfall (inches

Figure 2-8 Average annual precipitation in the United States (31) (Enlarged map on page 649)

per hour or millimeters per hour), and can be quite variable throughout the duration of a storm.

Precipitation occurs as rainfall, sleet, snow, hail, fog, and dew depending primarily on temperature. Snow falls in the northern climates every winter almost without fail but only infrequently in southern climates. Precipitation varies significantly from place to place not only in the amount that falls but also the seasons during which it falls. Figure 2-8 shows the annual distribution of precipitation throughout the United States in inches per year. Considerable variation exists. Generally, more precipitation occurs on the coasts and less in the higher inland areas. Figure 2-8 has been drawn from the historical measurements of precipitation at approximately 14,000 standard rain gauges maintained by the U.S. National Weather Service. Further variability exists from year to year, as can be seen in the data plotted in Fig. 2-2. This timewise and spacewise variation in precipitation adds considerably to the uncertainty in hydrologic predictions.

Weather and Meteorology

Temperature and precipitation are the two characteristics of weather most familiar to all of us. Quantitatively, each is governed by energy given off by the sun and distribution and absorption of that energy on the earth. All weather, and hence all precipitation, is governed by movement of the air mass surrounding the earth. Motion of that air mass is unsteady and turbulent. The scale of the turbulence is extremely large and, in general, is governed on a grand scale by the size of continents and on a lesser scale by the size of mountains or other topographic features.

In understanding the motion of the earth's atmosphere, two factors are important: atmospheric pressure and the Coriolis effect. Near the earth's surface, air flows into a low-pressure region and out of a high-pressure region (see Fig. 2-9). The Coriolis effect causes the flow of air to bend toward the right as it is initially drawn into a low-pressure area in the northern hemisphere. Thus, a counterclockwise rotation tends to develop around a low-pressure zone and a clockwise rotation around a high-pressure zone in the northern hemisphere, whereas the reverse occurs in the southern hemisphere. Continuity requires an upward flow of air in the middle of a low-pressure region and a downward flow in a high-pressure region. Rising air expands with increasing altitude and decreasing pressure. The expansion causes the air to cool, which in turn lessens its capacity to hold moisture. Moisture condenses forming clouds, and if sufficient moisture is originally contained in the air mass, precipitation occurs. Local low-pressure zones are formed by local heating of the atmosphere and often produce thunderstorms, which generally cover 40 sq mi or less and may cause intense rainfall for short durations, usually less than a few hours.

Large low-pressure regions occur as the result of instabilities in a weather front. Such a large low-pressure zone is called an extratropical cyclone. These intense low-pressure zones may cover areas as small as a few hundred square miles or as large as several thousand square miles. Rainfall produced is generally less intense than that from a local thunderstorm but may last from 24 to 72 hr or even longer.

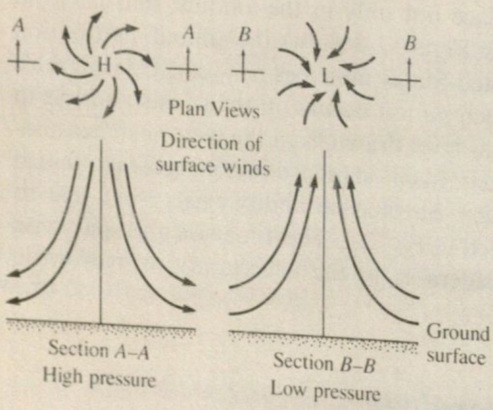


Figure 2-9 Flow of air associated with high- and low-pressure zones in the northern hemisphere

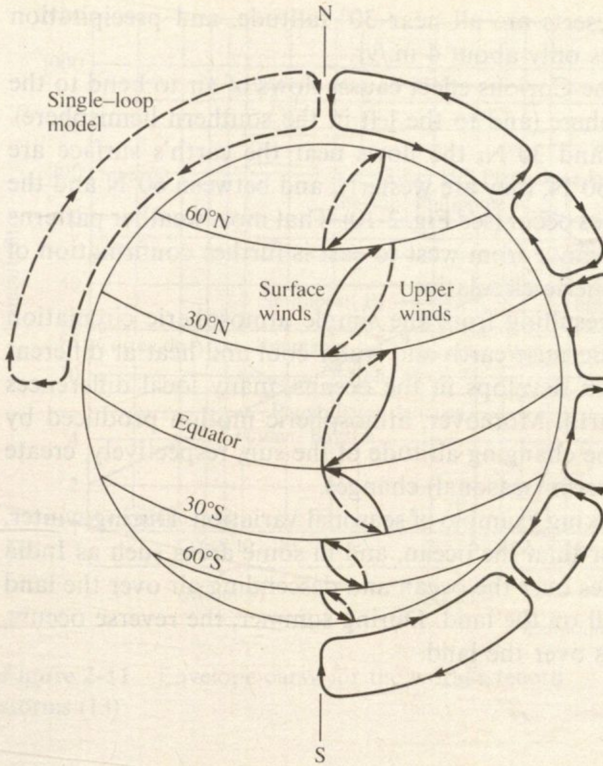


Figure 2-10 Flow patterns of circulation of the earth's atmosphere

Generalities of climate can be discerned from a simplified view of the motion of the atmosphere. Figure 2-10 shows such a simplified picture. Air warmed at the equator rises causing a southerly flow toward the equator in the northern hemisphere. At the North Pole, very cool air descends. Coupling these two simple observations and the principle of continuity, one might expect a southerly flow of air along the surface of the Earth and a northerly flow at higher altitudes (as shown in the single-loop model at the left of Fig. 2-10). However, the rising, cooling air at the equator develops enough momentum to carry it to an elevation above its equilibrium position. Being too high, and thus heavier than its equilibrium weight, it descends again at approximately 30° latitude. Continuity then requires that two other loops form, one between 30°N and 60°N and another between 60°N and 90°N. The position of these loops varies with the seasons in accordance with the elevation of the sun.

High pressure is characteristic of the descending cool air near the poles and at 30°N and 30°S latitude. Low pressure develops where warm air ascends at the equator and at 60°N and 60°S. Thus, one would expect relatively little precipitation to occur near the poles and at 30° latitude. Supporting this conclusion, the world's great deserts are all near 30° latitude, and precipitation near the North Pole averages only about 4 in./yr.

As pointed our earlier, the Coriolis effect causes flows of air to bend to the right in the northern hemisphere (and to the left in the southern hemisphere). Thus, between the equator and 30°N, the flows near the earth's surface are easterly. Between 30°N and 60°N, they are westerly, and between 60°N and the North Pole, the polar easterlies occur (see Fig. 2-10). That most weather patterns in the United States tend to move from west to east is further confirmation of this simple model of atmospheric circulation.

The picture of climate resulting from the simple atmospheric circulation model fails in many places. Because earth and water cool and heat at different rates, and because circulation develops in the oceans, many local differences can be found around the earth. Moreover, atmospheric motion produced by large-scale turbulence and the changing altitude of the sun, respectively, create short-term (daily) and long-term (seasonal) changes.

Monsoon rains are a striking example of seasonal variation. During winter, the land area becomes colder than the ocean, and in some areas such as India this results in rising air masses over the ocean and descending air over the land producing little or no rainfall on the land. During summer, the reverse occurs, and monsoon rainfall occurs over the land.

World's Record Storms

From the standpoint of predicting floods or flood levels, extreme precipitation events are of predominant interest. Extreme precipitation events usually occur because of several simultaneous meteorological occurrences (37). The atmosphere near the earth always contains moisture to a greater or lesser degree. Warm air moving inland from the ocean will generally have a large moisture content because of evaporation from the ocean, whereas cold air descending from aloft in a high-pressure zone will be relatively dry. For a major storm to develop, the atmosphere must contain a large amount of moisture, have a source of energy, and have a liberal inflow of moisture from outside the storm area. Great depths of precipitation occur as a result of a low-pressure zone becoming stalled for long periods with inflow of moisture-laden air. As we mentioned, short-duration storms frequently have high intensities of precipitation, whereas long-duration storms may have lower intensities but great total depths.

Figure 2-11 shows several of the world's greatest rainfall events in terms of depth and duration (13). The short-duration storms on the left of the graph are primarily thunderstorms and can apparently be expected almost anywhere in the tropical, arid, or temperate world. The world's record long-duration storms on the right of the graph, however, have occurred predominantly in areas subject to monsoon rainfall.

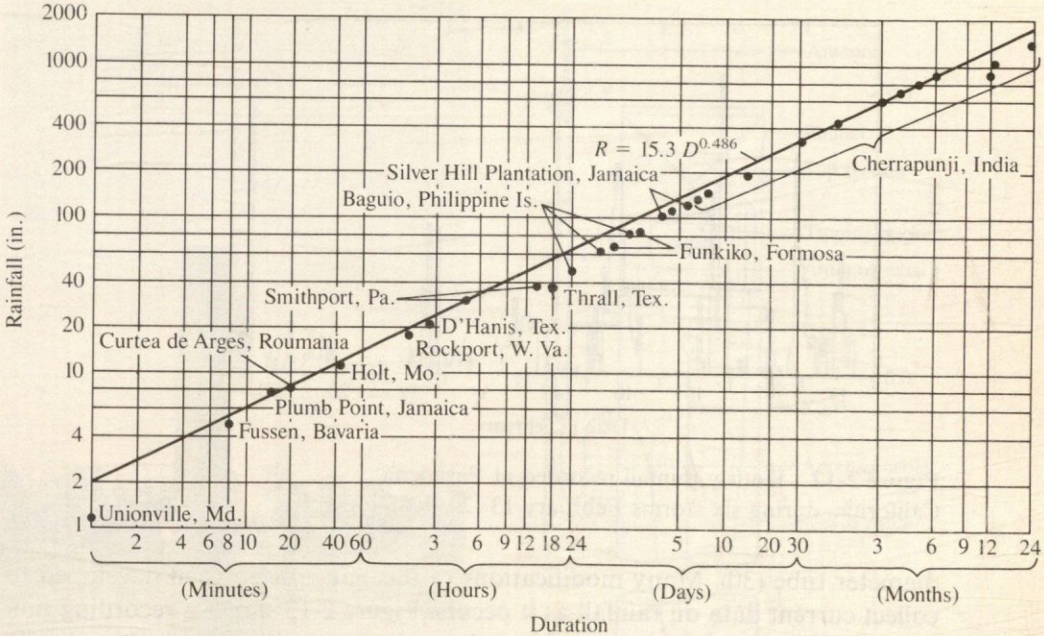


Figure 2-11 Envelope curve for the world's record storms (13)

Rainfall Distribution

To fully describe precipitation quantitatively requires at least two parameters: depth and duration. Alternatively, we could use intensity and duration, since intensity is depth of rainfall divided by duration. Depth is the total amount of precipitation (millimeters or inches) to fall in a given amount of time (*duration*). Precipitation is actually quite variable. Figure 2-12 is a plot of precipitation versus time recorded for actual rainstorms at Pasadena, California, and shows that incremental depth of rainfall (or intensity) varies greatly during a storm. The variability of the precipitation is due to the storm changing both direction and velocity as it moves, the storm cell enlarging and reducing in size with time, and the amount of energy driving the storm changing with time. Precipitation information is normally required to determine runoff and/or streamflow. Thus, quantitative information on precipitation is often vital to hydrologic analysis.

Quantitative information on precipitation is collected by means of a rain gauge. Some rain gauges have recorders or telemeters and provide information on the time rate of rainfall. Other gauges are read only once each day and, thus, provide only data on 24-hr precipitation depths. The amount of rainfall recorded during a period is defined as the "catch" or "depth" of rainfall. The standard rain gauge used in the United States consists of a straight-sided, 8-in.

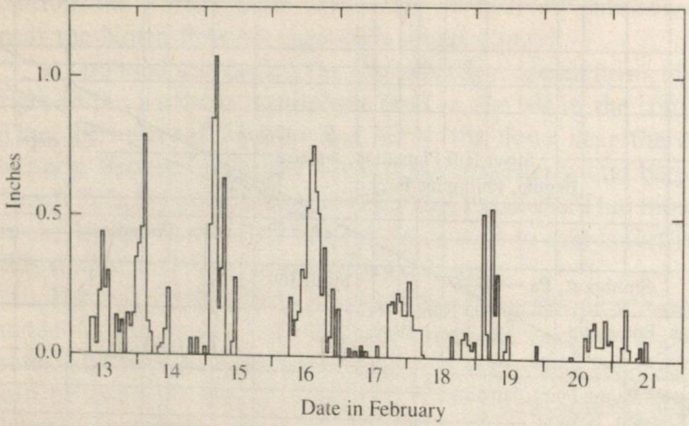


Figure 2-12 Hourly rainfall recorded at Pasadena, California, during six storms February 13–20, 1980 (4).

diameter tube (30). Many modifications of this gauge have been developed to collect current data on rainfall as it occurs. Figure 2-13 shows a recording rain gauge that incorporates a recorder and a radio transmitter that automatically sends data on rainfall to a central location for analysis and processing. Rain gauges like this one are also used to provide real-time warnings of possible flooding.

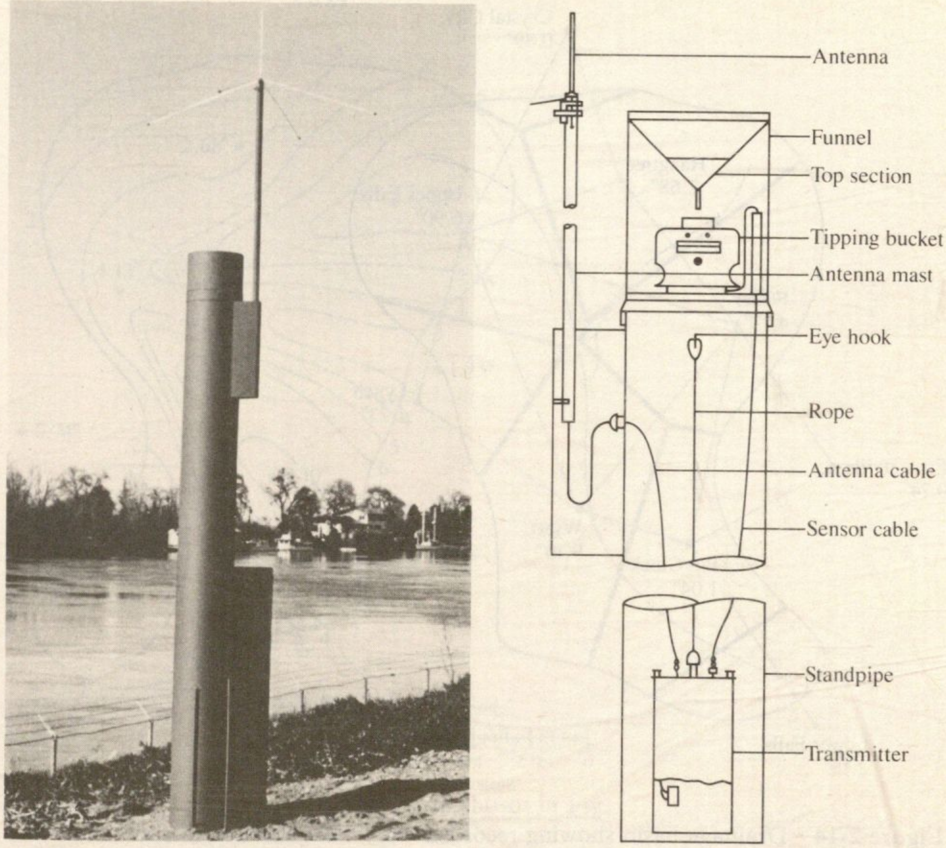
Because rainfall varies spatially during a storm, the catch of a single rain gauge may be significantly different from that of another gauge within the same general area. When several gauges have recorded rainfall on a given drainage area, the catch of all gauges should be considered in determining the average depth of rainfall over the drainage area. Three different methods can be used to compute average depth.

STATION AVERAGE METHOD Using the station average method, the catch of all gauges is simply averaged:

$$P_{\text{avg}} = \frac{\sum_{i=1}^{N} P_i}{N} \tag{2-14}$$

where P_{avg} = average precipitation depth over the basin
P_i = precipitation measured at gauge i
N = number of gauges

This method is applicable where the drainage area contains a relatively large number of uniformly distributed gauges. Figure 2-14 shows a typical drainage area. Some gauges outside the drainage area may be used if they appear to represent part of the area. The next section describes how, in some



(*a*) Photo of installed gauge (*b*) Schematic diagram of gauge

Figure 2-13 Self-reporting rain gauge at Shadun rain station on the Lu River, Hubei, Republic of China. (Courtesy of Sierra-Misco, Inc., Berkeley, California) (a) Photo of installed gauge. (b) Schematic diagram of gauge.

cases, a gauge outside the basin may be representative of some of the area in the basin.

Thiessen Polygon Method In this method, the precipitation measured by each gauge is assumed to be representative only of the area closest to it. No consideration is given to topography or storm characteristics. To construct the Thiessen polygons, lines are drawn connecting all adjoining gauge locations (the dashed lines in Fig. 2-14). Perpendicular bisectors to each of these (the solid lines in Fig. 2-14) are constructed. The perpendicular bisectors (and the boundaries of the drainage area) then form the portion of the drainage area represented by each gauge. The average precipitation is then computed:

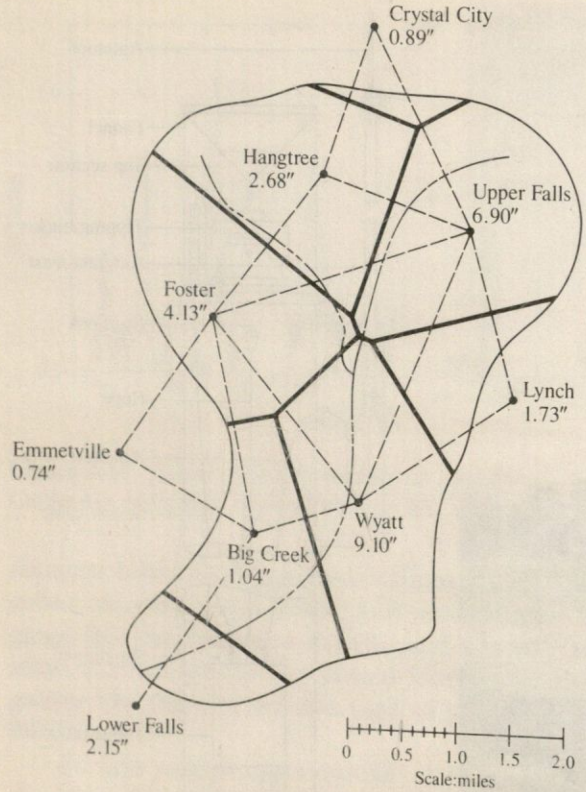


Figure 2-14 Drainage basin showing recorded rainfall for a 24-hour storm and constructed Thiessen polygons

$$P_{avg} = \frac{\sum_{i=1}^{N} P_i \cdot A_i}{\sum_{i=1}^{N} A_i} \tag{2-15}$$

Where A_i is the area represented by gauge i.

The Thiessen polygon method accounts for nonuniform distribution of stations within the drainage area and also provides guidance on which stations outside the drainage area should be included in calculating the average precipitation. Note that in Fig. 2-14, four stations which are actually outside the drainage area are used to represent rainfall on portions of the drainage area because they are closer to those areas than are any of the other stations within the area.

Isohyetal Method The isohyetal method consists of plotting isohyets (contours of equal precipitation) on a map of the drainage area. Figure 2-15

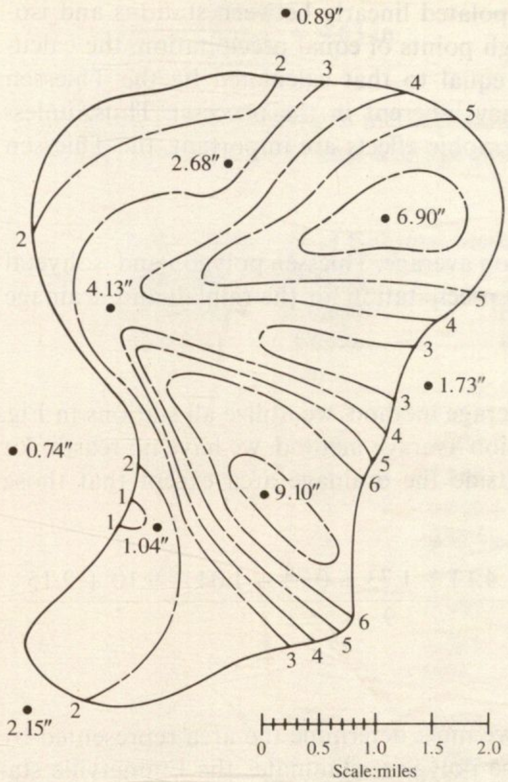


Figure 2-15 Drainage basin and storm shown in Fig. 2-14 with constructed isohyetal lines

illustrates an isohyetal map for the same storm illustrated in Fig. 2-14. Construction of the isohyetal lines is based on the recorded precipitation at the stations.

In actuality, few drainage areas have enough rain gauges to enable the isohyets to be constructed using only rainfall data. However, isohyetal lines can be drawn parallel to ground contour lines where orographic effects appear to be important. The average precipitation is calculated:

$$P_{\text{avg}} = \frac{\sum_{j=1}^{m} A_j \cdot (P_j + P_{j+1})/2}{\sum_{j=1}^{m} A_j} \tag{2-16}$$

where P_j = the precipitation on isohyet j

A_j = the area between isohyet j and $j + 1$

m = the number of intervals between isohyets.

If precipitation depths are interpolated linearly between stations and isohyetal lines are then sketched through points of equal precipitation, the calculated average precipitation will be equal to that calculated by the Thiessen polygon method (within the accuracy inherent in the process). Thus, unless there is good reason to believe orographic effects are important, the Thiessen polygon method is adequate.

EXAMPLE 2-9 Using the station average, Thiessen polygon and isohyetal methods, compute the average basin precipitation for the rainfall and drainage basin shown in Figs. 2-14 and 2-15.

SOLUTION For the station average method, we utilize all stations in Fig. 2-14 and Eq. (2-14). In using the station average method, we have no reason for using data for the four stations outside the drainage area except that those stations are nearby.

$$P_{\text{avg}} = \frac{0.89 + 2.68 + 6.90 + 4.13 + 1.73 + 0.74 + 1.04 + 9.10 + 2.15}{9}$$

$$= 3.26 \text{ in.}$$

For the Thiessen polygon method, we must determine the area represented by each station. The construction of the polygons eliminates the Emmetville station from consideration. The accompanying table gives the areas determined for each station. Calculation of the average precipitation for the basin uses Eq. (2-15).

Illustration of Use of Thiessen Polygon Method

(1) Station	(2) Recorded Rainfall Depth P (in.)	(3) Area A Represented by Station (mi²)	(4) Rainfall Volume (mi²-in.)
Crystal City	0.89	0.21	0.187
Hangtree	2.68	2.82	7.558
Upper Falls	6.90	3.00	20.700
Foster	4.13	2.64	10.903
Lynch	1.73	1.00	1.730
Emmetville	0.74	0	0
Wyatt	9.10	2.94	26.754
Big Creek	1.04	2.07	2.153
Lower Falls	2.15	0.82	1.763
	Totals	15.50	71.748

$$P_{\text{avg}} = \frac{71.748}{15.50} = 4.63 \text{ in.}$$

Note: The Thiessen polygon method assigns a very large area to station Wyatt and as a result yields a large average precipitation.

Summary for Use of Isohyetal Method

Rainfall Depth on Isohyet (in.)	Average Rainfall Depth (in.)	Area Between Isohyets (mi^2)	Rainfall Volume (mi^2-in.)
9.1			
	8.55	0.407	3.480
8.0			
	7.0	1.412	9.884
6.0			
	5.5	0.841 + 1.375 = 2.216	1.219
5.0			
	4.5	0.592 + 1.697 = 2.289	10.300
4.0			
	3.5	3.122	10.927
3.0			
	2.5	2.599 + 0.431 = 3.030	7.575
2.0			
	1.5	2.281	3.422
1.0	1.0	0.05	0.050
6.9			
	6.45	0.693	4.470
6.0	Totals	15.500	51.327

For the Isohyetal method, the areas between adjoining isohyets shown in Fig. 2-15 have been determined using a planimeter and are listed in the accompanying table. Using Eq. (2-16), the average precipitation for the basin is

$$P_{\text{avg}} = \frac{51.327}{15.50} = 3.31 \text{ in.} \qquad \blacksquare$$

Depth-Duration-Frequency Relations

To develop a design storm for analysis of flooding, it is necessary to have information on the depth of precipitation that can be expected to occur within a given period. It will also be necessary to choose a recurrence interval of storm for use in the design. Information on depth, duration, and frequency of rainfall has been extensively analyzed and published for the United States by the U.S. National Weather Service (35, 36).

Depths of rainfall at a particular point appear to be nearly random events, as is shown in Fig. 2-2, page 23. The variation from year to year effectively hides any long-term variation that may be due to climate. Because of its variation,

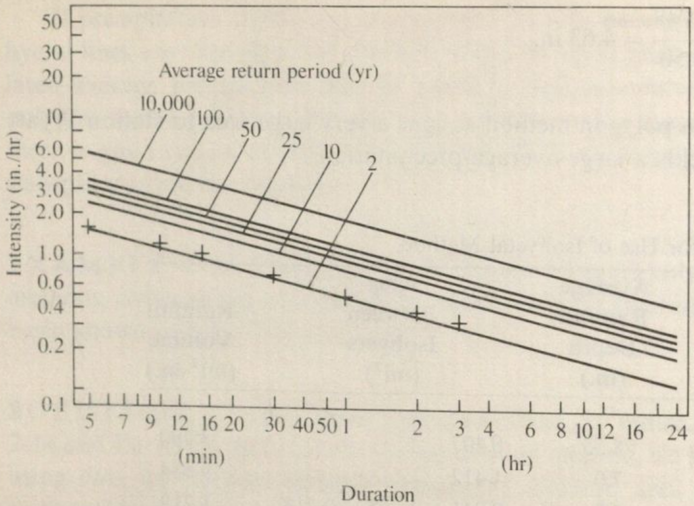


Figure 2-16 Intensity-duration-frequency curves for Grizzley Flat, California (7)

rainfall at a point must be described statistically using a mean for the period (such as mean annual or mean monthly) and its variation about that mean. Precipitation also varies with time during an individual storm, and the duration of rainfall (total time during which rain falls) varies from one storm to another. Thus, to describe rainfall quantitatively it is necessary to include both the amount of rainfall (depth) and the corresponding elapsed time (duration). This is called a depth-duration analysis if depths of rainfall are considered, or alternatively, an intensity-duration analysis of intensities are considered.

To evaluate average frequency of occurrence (or recurrence interval), it is necessary to perform further analyses of rainfall data for each gauge. To make a frequency analysis of intensity-duration data, the data must have been continuously recorded so that various durations of rainfall can be studied. Records for each year are first scanned for intervals of time. For short-duration storms, the data are first scanned using 5-min increments or smaller. The largest single 5-min depth would be selected and assigned order $m = 1$; the second largest, order $m = 2$; the third largest, $m = 3$; and so on. Eventually, a set of 5-min rainfall depths will have been selected equal in number to the number of years of record N. The apparent return frequency of each point is calculated using Eq. (2-13). A curve of frequency versus rainfall depth is then plotted on probability paper. The same process is repeated for each duration of interest until curves of frequency versus depth have been developed for all durations of interest. That family of curves can then be used to prepare a set of depth-duration-frequency curves, as shown in Fig. 2-16.

The foregoing type of analysis can cover a point, a region, or an entire nation. The National Weather Service (NWS) has prepared intensity-duration-

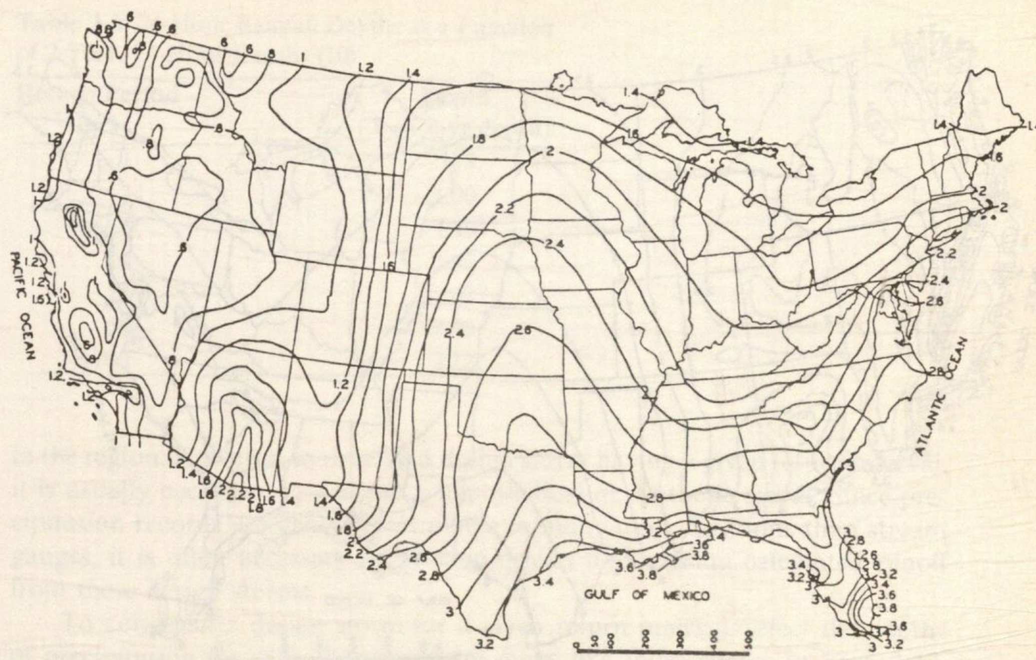


Figure 2-17 10-year 1-hour rainfall for the United States (36)

frequency values for most recording stations in the United States (35). Those results and more recent analyses have been used to construct depth-duration-frequency values of rainfall for the United States (37). Figures 2-17, 2-18, and 2-19, show depth-duration values for a 10-yr return interval and three durations. Further analysis has led to separate NWS publications for each of the states on a scale more detailed than is shown in Figs. 2-17 through 2-19.

Synthetic Storm Design

For drainage design of a particular project, selecting a design storm is necessary. For small drainage projects such as a parking lot or an industrial site, a single intensity is usually used. Thus, in designing the storm drainage system for a housing area, the engineer might select a 10-yr intensity from the U.S. National Weather Service (35). Later we will discuss methods for choosing the proper duration.

For a project involving a drainage area larger than several square miles, it is necessary to develop a design storm with a duration as long as 24 to 72 hr. Intensity is constant only for durations of a few minutes and varies significantly over a long duration. Past design practices used the most severe recorded storms

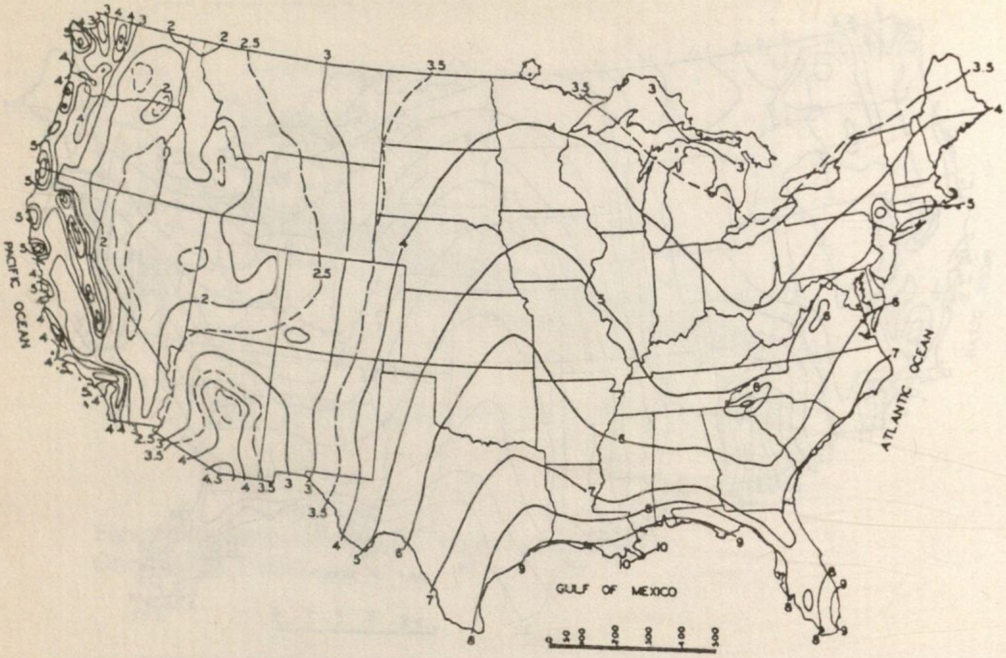


Figure 2-18 10-year 6-hour rainfall for the United States (36)

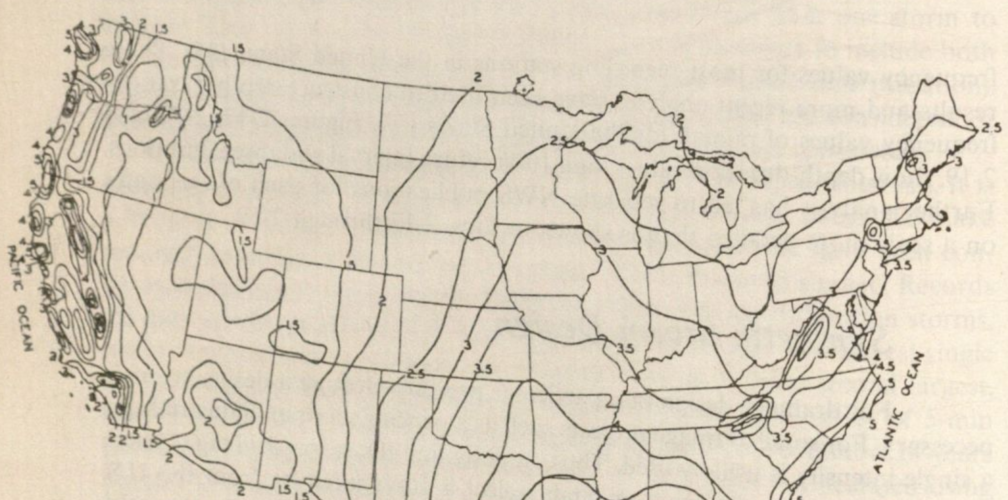


Figure 2-19 10-year 24-hour rainfall for the United States (36)

Table 2-5 24-Hour Rainfall Depths as a Function of 2-Year, 24-Hour Depths (10)

Return Period (yr)	Depth (% of 2-yr depth)
1	0.78
2	1.00
5	1.40
10	1.60
25	1.82
50	2.00
100	2.22

in the region. However, to develop a design storm having a given return interval, it is usually necessary to develop a hypothetical or synthetic storm. Since precipitation records are generally available in many more locations than stream gauges, it is often necessary to develop design floods using calculated runoff from these design storms.

To construct a design storm for a given return interval, select the depths of precipitation for each duration from maps like those shown in Figs. 2-17, 2-18, and 2-19. Table 2-5 provides acceptable empirical guides to proportion 24-hr rainfalls for various return intervals.

The depths can be plotted against duration for various return intervals as shown in Fig. 2-20. Smooth curves drawn through the points provide the means to interpolate for different durations or return periods. Table 2-6 shows tabulated values of rainfall for a 100-yr 24-hr storm at the common eastern corner of Arkansas and Missouri. Values in the table (column 2) were interpolated from Fig. 2-20.

The sequence of rainfall depths in a synthetic storm is more or less arbitrary. As shown in Fig. 2-12, the time distribution of an actual storm can be irregular. Nevertheless, the hydrologist must rearrange the incremental values in column 3 of Table 2-6 to represent a reasonable storm pattern. In general, the peak increment should be placed at approximately the one-third point of the storm's duration. The other increments are arranged more or less arbitrarily to provide for a continuous increase in intensity before the peak and a continuous decrease afterward. Column 4 of Table 2-6 shows a possible distribution. Specific arrangements have been adopted by certain firms and agencies. Table 2-7 shows a distribution used by the U.S. Soil Conservation Service for storms west of the Sierra Nevada and Cascade Mountains (Type 1) and storms in other parts of the United States (Type 2). Later, when we discuss runoff, further reasoning will be given for the distribution of incremental rainfall depths in a design storm.

Point rainfall during a storm is generally larger than the average rainfall over the area. Thus, values taken from Figs. 2-17, 2-18, and 2-19, which are point-rainfall values, must be corrected to represent averages over the drainage area. A study made by the NWS led to the development of Fig. 2-21, which

Table 2-6 Construction of a 100-Year Storm for a 100 Square-Mile Drainage Area at the Common Eastern Corner of Arkansas and Missouri

(1) Time (hr)	(2) Accumulated Rainfall (in.)	(3) Increment (in.)	(4) Selected Rainfall Sequence* (in.)	(5) Rainfall Sequence Average Over Area (in.)
0	0	0	0	0
2	4.10	4.10	0.10	0.09
4	4.80	0.70	0.20	0.19
6	5.40	0.60	0.30	0.28
8	5.85	0.45	0.45	0.42
10	6.15	0.30	4.10	3.85
12	6.40	0.25	0.70	0.66
14	6.65	0.25	0.60	0.56
16	6.90	0.25	0.25	0.24
18	7.10	0.20	0.25	0.24
20	7.30	0.20	0.25	0.24
22	7.40	0.10	0.20	0.19
24	7.50	0.10	0.10	0.09
Totals	7.50	7.50	7.50	7.05

* This selected distribution is a rearrangement of incremental values from column 3 to provide a realistic sequence representative of actual storms.

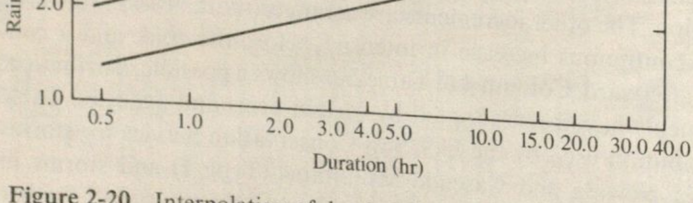


Figure 2-20 Interpolation of depth-duration-frequency for rainfall for a point at the common eastern corner of Arkansas and Missouri.*

* Values shown as ⦿ are taken from Figs. 2-17, 2-18, and 2-19. Values shown as • are computed from 10-yr values using Table 2-5. The lines for 100-yr and 2-yr values were drawn parallel to that for the 10-yr values.

Table 2-7 Accumulation of Rainfall up to 24 hours (34)

Time (hr)	P_x/P_{24}* Type 1	P_x/P_{24}* Type 2	Time (hr)	P_x/P_{24}* Type 1	P_x/P_{24}* Type 2
0	0	0	11.0	0.624	0.235
2.0	0.035	0.022	11.5	0.654	0.283
4.0	0.076	0.048	11.75	. . .	0.387
6.0	0.125	0.080	12.0	0.682	0.663
7.0	0.156	. . .	12.5	. . .	0.735
8.0	0.194	0.120	13.0	0.727	0.772
8.5	0.219	. . .	13.5	. . .	0.799
9.0	0.254	0.147	14.0	0.767	0.820
9.5	0.303	0.163	16.0	0.830	0.880
9.75	0.362	. . .	20.0	0.926	0.952
10.0	0.515	0.181	24.0	1.000	1.000
10.5	0.583	0.204			

* P_x/P_{24} is the ratio of accumulated rainfall at time x to the accumulated rainfall in 24 hours.

has been widely used as a means to adjust point rainfall depths to averages over an area (36). Values in column 5 of Table 2-6 were obtained by multiplying values in column 4 by 0.94, as obtained from Fig. 2-21 for a 100-sq-mi drainage area.

Snowmelt

In many of the colder climates of the world, such as the Sierra Nevada mountains between Nevada and California, a major part of the annual precipitation occurs as snow. Melting snow during the spring and early summer

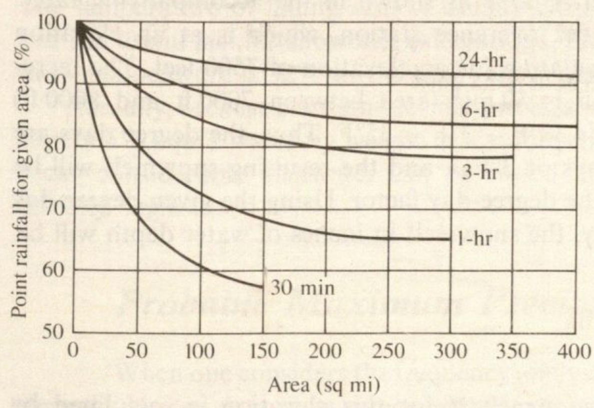


Figure 2-21 Area average versus point rainfall for use with depth-duration values (36)

months forms much of the annual runoff. In climates like this, major floods are often produced by rainfall on snow. The rain containing more heat per unit of mass tends to melt the snow. Newly fallen snow has a specific gravity of approximately 0.1, thus, 10 in. of new snow would have a water equivalent of 10% or only 1 in. As winter progresses, the snow pack compresses and reaches water equivalents of 40 to 50%.

With the arrival of spring, the air temperature warms and the snowpack melts producing streamflow. The scientific basis of snowmelt is a function of at least incoming and back radiation, air temperature, and wind (38). Only rarely are records of all these phenomena available, and snowmelt computations or predictions must usually use approximate analyses. The most common approach is the use of degree days. One *degree day* is defined as a 24-hr period during which the average temperature is 1°F above 32°F. A degree-day factor is determined by dividing the total snowmelt runoff (in inches) from a drainage basin by the number of degree days for that period of runoff. Thus, the degree-day factor is an empirical factor implying the snowmelt (in inches of water) per degree day occurring on a particular drainage basin. Degree-day factors of 0.05 to 0.15 in. per degree day are common. Since temperature varies with altitude, degree-day factor analysis can confidently be used only on basins with uniform snow coverage and moderate relief. Example 2-10 shows a snowmelt computation for a basin with considerable relief.

EXAMPLE 2-10 The accompanying table shows drainage area versus elevation for a particular snow-covered basin. Assuming that temperature decreases at a rate of 3°F/1000 ft, calculate the snowmelt depth to be expected for a given day when the reference mean-daily temperature is 38°F at an elevation of 6500 ft. Assume a degree-day factor of 0.1. The lowest elevation of snow (snowline) is at 5000 ft.

SOLUTION The solution is done as shown in the accompanying table. The temperature is 38°F at the reference station, which is at an elevation of 6500 feet. Consider the calculation for an elevation of 7500 feet. The incremental area for this elevation is 50 mi^2 (area between 7000 ft and 8000 ft) and the average temperature is 38°F − 3°F = 35°F. Thus, the degree days are 1 day·(35°F − 32°F) = 3°F-days or 3°FD, and the resulting snowmelt will be equal to 3°FD multiplied by the degree-day factor. Using the given degree-day factor of 0.1 in. per degree day, the snowmelt in inches of water depth will be

$$3°\text{FD} \times \frac{0.1 \text{ in.}}{°\text{FD}} = 0.3 \text{ in.}$$

The incremental volume of snowmelt $\forall$ for this elevation is calculated by multiplying the area by the depth of melt:

Elevation (ft)	Area Above Elevation (mi^2)	Incremental Area (mi^2)	Average Temperature (°F)	Degree Days	Snowmelt (mi^2-in.)
11,000	0	20	26	0	0
10,000	20	30	29	0	0
9,000	50	40	32	0	0
8,000	90	50	35	3	15.0
7,000	140	60	38	6	36.0
6,000	200	70	41	9*	63.0
5,000	270				Total = 114.0 mi^2-in.
					= 6080 acre-ft

* This calculation assumes that the depth of snowpack is great enough to provide 0.9 in. of water equivalent.

$$\forall = 50 \text{ mi}^2 \times 0.3 \text{ in.}$$
$$= 15.0 \text{ mi}^2\text{-in.}$$
$$= 800 \text{ Acre-ft or } 800 \text{ AF}$$

The complete solution is given in the table above. ■

Rain melts snow in accordance with calorimetric theory, and since 144 BTU of heat are required to melt 1 lb of ice at 32°F, Eq. (2-17) yields the number of inches M of melt produced by a rainfall of R inches having a Fahrenheit temperature Tw:

$$M = \frac{R(Tw - 32)}{144} \tag{2-17}$$

The temperature of falling rain is usually close to the wet-bulb temperature and is a useful fact for snowmelt calculations. Equation (2-17) assumes that the snow temperature is 32°F at the time rain begins.

Actually, because of intermolecular forces, a snowpack can contain approximately 5% liquid water by weight in a stable condition. Thus, for deep snowpacks considerable meltwater can be stored in the snowpack before runoff actually begins.

Probable Maximum Precipitation

When one considers the frequency analysis of rainfall or peak floods, the normally limited data plots somewhat as is shown in Fig. 2-16. For a given duration, the depth of rainfall appears to increase continuously with increasing recurrence interval. How much could fall in the largest possible rainstorm? Extending the frequency curve appears to indicate that the maximum depth would be infinite. However, physical limitations in the occurrence of rainfall limit that depth to what has become known as the *probable maximum precipitation* (PMP).

Figure 2-11 shows a plot of the world's record rainfall. The straight line drawn upward to the right in Fig. 2-11, page 43, falls just above all the plotted points. The equation of this line is

$$R = 15.3D^{0.486} \tag{2-18}$$

where R is rainfall depth in inches, and D is duration in hours. Rainfall cannot occur faster than moisture can be supplied to the air column above the point of interest. Thus, the volume of moisture (usually expressed as depth) in the air column, the rate that wind brings moisture into the column, and the physical efficiency of the condensation-rainfall process limit the maximum depth of rainfall that can occur at a point for a given duration. Looking at frequency curves like Fig. 2-11 leads to the conclusion that there is an upper limit to precipitation depth for a given frequency.

Computing PMP uses records of intense storms for which relative humidity data are available. Actual moisture available in a column of air is computed based on the relative humidity occurring at the time (37). Thus, the steps in estimating a PMP depth are as follows:

1. Obtain records of rainfall depth R and relative humidity for severe storms that have occurred at or near the site.
2. Calculate the moisture P available in a column of air at the time the data on rainfall depth R were taken (36).
3. Calculate the probable maximum moisture M that could have been available had the relative humidity been a maximum. Maximum relative humidity must be determined on the basis of records at or near the site.
4. Calculate a maximizing factor as $K = M/P$.
5. Estimate the PMP as PMP $= R \times K$.

Many specific considerations go into estimating the PMP, and the illustration given is only a simple consideration. The severe storms chosen must be exceptional storms for which the rainfall efficiency can be assumed to be as high as that which would occur under probable maximum conditions. If a storm is to be transposed to another area, you must be certain that the storm could occur in the area of interest. This judgment requires the knowledge of an experienced meteorologist.

In some areas, generalized estimates of PMP have been prepared by analyzing storms in regions. These studies have been done by the U.S. National Weather Service (28, 29). Figure 2-22 shows the results of one of these studies. To use Fig. 2-22 to construct a probable maximum storm sequence, one must

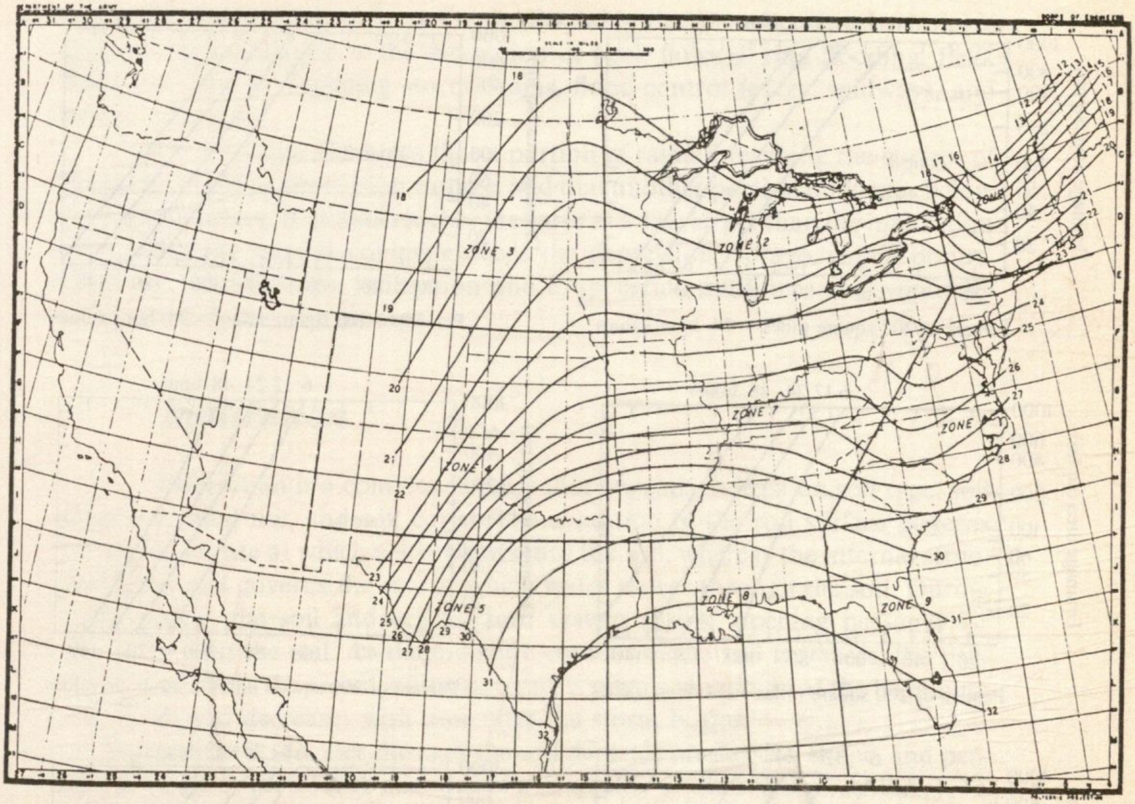


Figure 2-22 Probable maximum 24-hr precipitation for 200 square miles for the United States east of the 105th meridian (29)

adjust the precipitation for the size of the drainage area involved and for duration. Figure 2-23 provides the means to make such adjustments (29).

The PMP is used as a design event when a large flood would result in hazards to life or great economic loss. Thus, large dams upstream from populated centers, under current design standards, must be designed with spillways adequate to protect the dam against failure during the flood produced by the PMP falling on the drainage area.

2-6 Surface Runoff

As we mentioned in our discussion of the hydrologic cycle, a portion of precipitation falling on a drainage area runs off. That portion moves initially as overland flow in very shallow depths before eventually reaching a stream channel where streamflow is produced. In engineering, this portion of the hydrologic cycle is of the greatest interest, since it is surface runoff that fills reser-

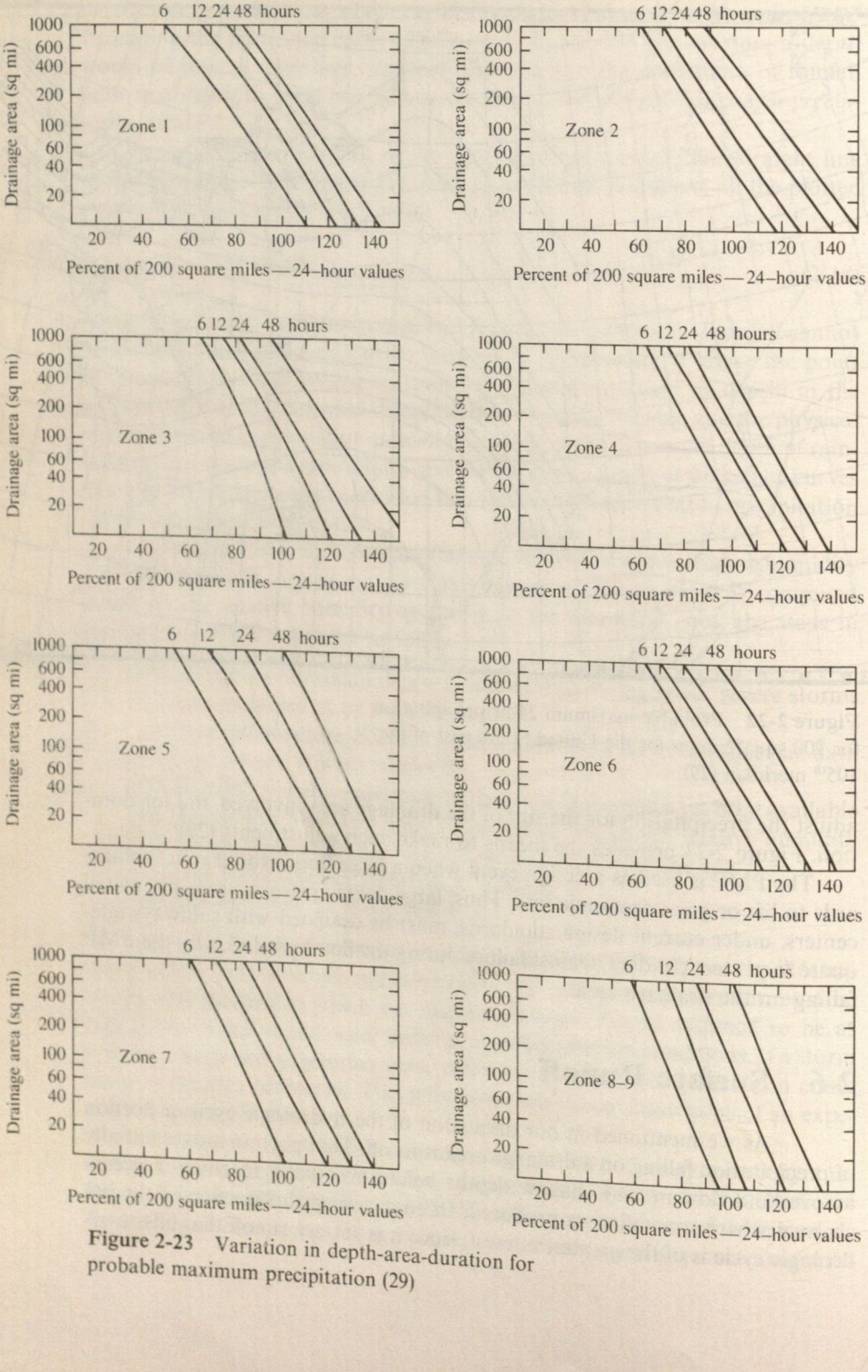


Figure 2-23 Variation in depth-area-duration for probable maximum precipitation (29)

voirs, fills rivers, and produces floods. Probably the most frequent hydrologic engineering calculation is the estimation of peak flows during flooding. Peak flows are used in designing storm drains, flood-control levees, spillways, and bridge openings.

Figure 2-4, page 25, shows that a portion of rainfall reaching the surface of the earth is held as depression storage, and that another portion infiltrates below the earth's surface. If rainfall reaches the earth at a rate larger than the infiltration rate, and if the rainfall volume exceeds the depression storage, runoff occurs. Once the rainfall stops, infiltration and evaporation continue and eventually exhaust the depression storage.

Infiltration

Infiltration is a complex process that depends at least on soil type, soil grain size, land use, and soil cover. The condition of the soil surface governs the ability or rate at which water passes into the soil, whereas the internal structure of the soil governs the rate at which water moves through the soil. During a dry period, the soil and vegetal root system shrink, opening passages for easy entry into the soil. As the moisture content of the soil increases, the passages close. Thus, the actual infiltration rate is often a maximum at the beginning of a storm and decreases with time after the storm begins.

Transmission of water through the soil depends on the size, shape, and percent of voids in the soil. A measure of this quality is the soil's *permeability*.

Vegetation covering the soil tends to increase total infiltration during a given storm, since raindrops are broken up and prevented from directly striking the ground, compacting the soil and closing its voids. Thus, the actual infiltration rate is greater on a vegetated soil than it would be for the same soil without cover, and since some rain is also intercepted and held by the vegetation, surface runoff is a greater percentage of total rainfall for bare ground.

The actual infiltration rate depends on both soil and cover conditions and rainfall intensity. The maximum rate at which infiltration could occur is the *infiltration capacity*. The actual rate of infiltration can approach the infiltration capacity only if the rainfall intensity is large enough to exceed the rate at which rainfall is intercepted and retained as depression storage.

A basic formulation for the infiltration capacity as a function of time was developed by Robert Horton (12) as

$$f = f_c + (f_0 - f_c)e^{-k_f t} \qquad (2\text{-}19)$$

where f = maximum infiltration rate at time t (infiltration capacity)

f_0 = initial infiltration rate at $t = 0$

f_c = minimum infiltration rate

k_f = a constant

e = the natural logarithm base (2.718)

Table 2-8 Soils Groups and Corresponding Minimum Infiltration Rates (34)

Group	Minimum Infiltration Rate (in./hr)	(mm/hr)	Soil Description
A	0.30–0.45	7.6–11.4	Soils having a high infiltration rate. They are chiefly deep, well-drained sands or gravels, deep loess, or aggregated silts. They have *low runoff potential.*
B	0.15–0.30	3.8–7.6	Soils having a moderate infiltration rate when thoroughly wet. They are chiefly moderately deep, well-drained soils of moderately fine to moderately coarse texture such as shallow loess and sandy loam.
C	0.50–0.15	1.2–3.8	Soils having a slow infiltration rate when wet. They are soils with a layer that impedes downward movement of water and soils of moderately fine to fine texture such as clay loams, shallow sandy loam, soils low in organic content, and soils high in clay content.
D	0.00–0.05	0.00–1.2	Soils having a very slow infiltration rate. They are chiefly clay soil with a high swelling potential, soils with a permanent high water table, soils with a claypan at or near the surface, shallow soils over nearly impervious material, heavy plastic clays, and certain saline soils. They have *high runoff potential.*

Although this equation is generally accepted as an empirical but reasonable quantification of the infiltration process, it is difficult to use since values for the constants f_c, f_0, and k_f are not easily determined. Minimum infiltration rates have been evaluated by the U.S. Soil Conservation Service (34) and are shown in Table 2-8 as a function of soil type. The value of f_0 depends on the surface condition of the soil as well as on the content of soil moisture and, therefore, varies with time since the last rain.

An average infiltration capacity called the ϕ index has been widely used because of its simplicity. The ϕ index is simply an average infiltration capacity that when applied to a particular rainfall event, yields the proper volume of runoff. Figure 2-24 illustrates the ϕ index and Horton's relationship. In separating runoff from infiltration, the value of ϕ is subtracted from each incremental rainfall amount with any resulting negative values set to 0.

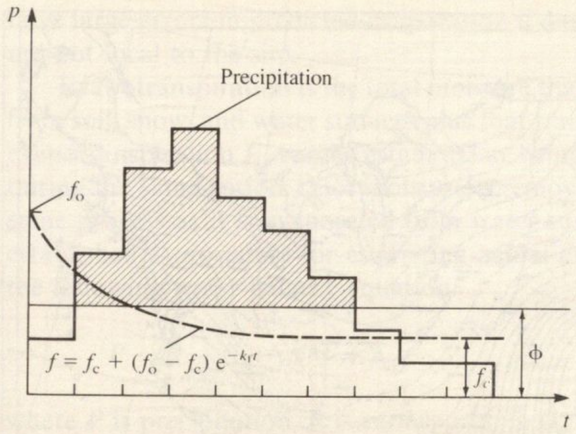


Figure 2-24 Illustration of ϕ index and Horton's equation for infiltration*

* The ordinate is precipitation in inches or millimeters per hour while the abscissa is time. Runoff is indicated by the area above the ϕ index line (shaded) or by the area above the curved line (Horton's Equation).

Another method commonly used considers infiltration to be made up of a fixed initial volume (abstraction) plus continuing infiltration at a fixed rate. This method is preferred by some over the ϕ index method because the abstraction represents an approximation of the noninfiltration quantities of interception and detention.

Modern methods of determining the parameters for use in the estimation of infiltration include analysis of the physical properties of the soil and the use of experimental infiltrometers. The experimental infiltrometers include flooding and sprinkling instruments (1).

Evaporation and Evapotranspiration

Evaporation from water and soil surfaces and transpiration through plants, as shown in Figs. 2-3 and 2-4, can account for significant volumes of water. Evaporation is the process by which water transforms into vapor. The process occurs at the water surface where molecules of water develop sufficient energy to escape bonds with the water and become vapor molecules in the air. Evaporation from a water body is a function of air and water temperatures, the moisture gradient at the water surface, and wind. Wind moves the moisture away from the lake's surface and, thus, increases the moisture gradient, increasing the rate of evaporation. Natural evaporation rates vary from as much as 90 in. per year in the hot dry climate of southeast California to 20 in. per year in the cool climate of northern Maine or the wet climate of northwest Washington. Figure 2-25 shows average annual lake evaporation in the United States.

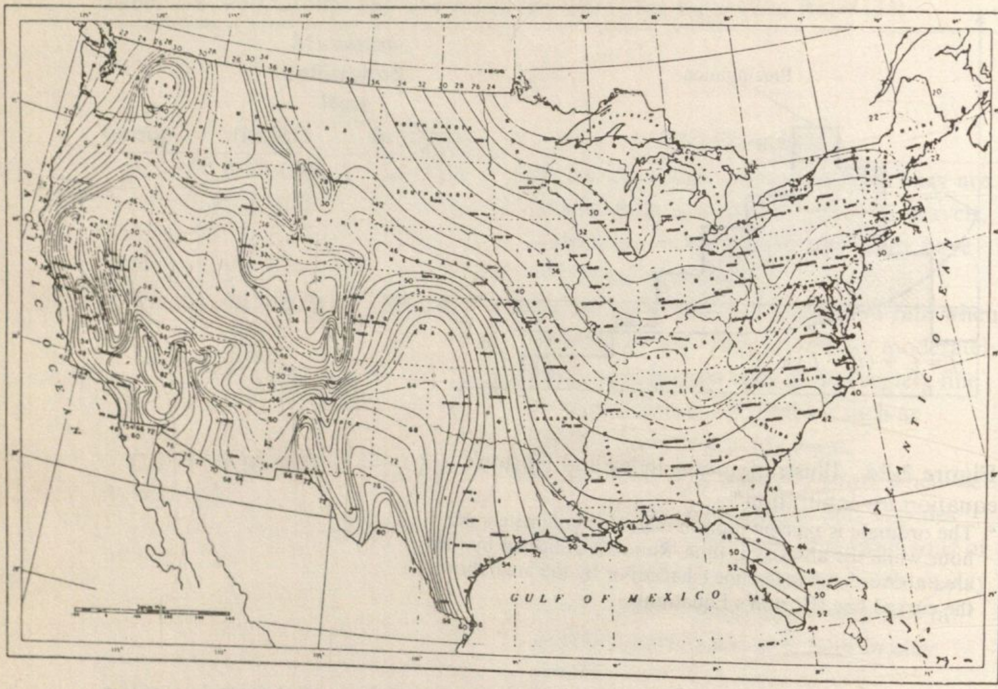


Figure 2-25 Average annual lake evaporation in the United States in inches (6) (Enlarged map on page 650)

Several different methods are used to estimate evaporation. The so-called Standard Class A Land Pan is used by the U.S. National Weather Service at its official stations. It is a metal pan 4 ft in diameter and 10 in. deep. Evaporation from the pan is measured daily by measuring the water level in the pan using a hook gage. The pan is mounted on a wooden frame 4 in. off the ground. The pan is drastically different aerodynamically and thermodynamically from a lake and yields a higher rate of evaporation than is normally experienced by a lake. An analysis of pan and lake evaporation by Kohler (15) showed that multiplication of measured pan evaporation by a "pan coefficient" equal to 0.7 yields annual lake evaporation rates accurate to within about 15% if the pan is subject to the same climatic conditions as the lake.

Extensive field investigations of evaporation, made at Lake Hefner, Oklahoma, from 1950 to 1953, yielded the following equation for lake evaporation, E:

$$E = 0.00241(e_s - e_8)V_8 \tag{2-20}$$

where E is the rate of evaporation in inches/day, e_s is the vapor pressure in inches of mercury at the water surface, and e_8 and V_8 are the vapor pressure and wind velocity (miles per day) 8 m above the lake surface. It is possible to have large errors in predicted evaporation if data are not taken accurately and are not local to the site.

Evapotranspiration is the total moisture that leaves an area by evaporation from soil, snow, and water surfaces plus that transpired by plants. The potential evapotranspiration E_p can be estimated as being equal to the lake evaporation during the same period, since moisture is removed from leaves of plants by the same process as it is evaporated from water surfaces. C.W. Thornthwaite (24) established a procedure for estimating actual evapotranspiration E_A by using the following water-balance equation:

$$P - R - G_o - \Delta M = E_A \tag{2-21}$$

where P is precipitation, R is surface runoff, G_o is subsurface outflow, and ΔM is the change in moisture storage within the soil. Equation (2-21) is applied to a given area, and moisture must be measured in the underlying soil. Consistent units must be used for each variable to yield E_A in either a volume or a depth of evapotranspiration in a given time.

Rainfall-Runoff Relations

The portion of rainfall that enters the stream quickly is called surface runoff. When rainfall starts, detention storage is filled first, and if the rainfall intensity is larger than the infiltration capacity, surface runoff occurs. The surface runoff flows first as a thin sheet overland and eventually reaches a rivulet or channel where the flow concentrates. Because the relative importance of viscous resistance to flow decreases as the hydraulic radius of the channel increases, the flow velocity in the channel is greater than that of the overland flow. As the flow moves downstream, flows from other channels join the stream, and the flow rate increases. The time required, after beginning of rainfall, for the most distant point in the drainage area to begin contributing runoff at the outlet of the basin is called the *time of concentration* t_c.

The continuous record of rate of flow as a function of time is called a stream *hydrograph*. Figure 2-26 shows a hydrograph for the Sacramento River at Red Bluff, California. The shape of the hydrograph reflects physical characteristics of the drainage basin as well as that of the storm producing the hydrograph. If the drainage basin is compact around the point of interest, streamflow at the outlet of the basin will peak rapidly since all points in the basin tend to be close to the outlet. By contrast, for a basin that is not compact, streamflow will peak more slowly. Figure 2-27 shows typical hydrographs for characteristic drainage basins having equal areas but different shapes.

Physical laws dictate that all other things being equal, steep slopes will produce more runoff than flat slopes; vegetated drainage areas will produce less runoff than bare areas; and areas where soils are relatively impermeable such as compacted clay will produce more runoff than sandy soils with high per-

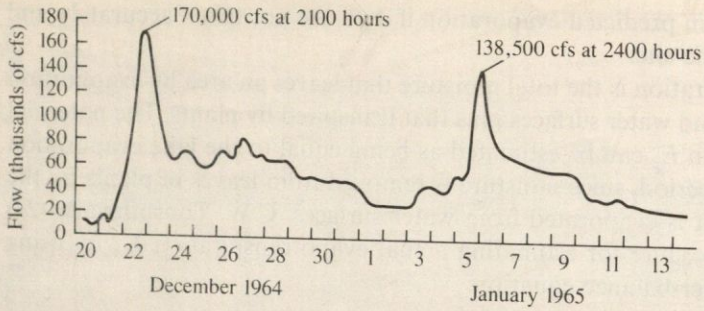


Figure 2-26 Hydrograph for the Sacramento River at Red Bluff, California, 1964–1965 flood. (23)

meability. Analytical methods for computing runoff cannot yet consider all characteristics of a drainage area exactly. However, many empirical methods have been developed with which to estimate runoff and a great deal of experience has been accumulated in using these methods, which provides confidence in their application. The most simple of these is the so-called rational equation mentioned earlier:

$$Q_p = CIA \tag{2-1}$$

where Q_p is the peak rate of runoff, I is rainfall intensity, and A is drainage area. This equation has been widely used in the design of small drainage systems, such as those for airports, city blocks, or parking lots. The runoff coefficient C is essentially the proportion of rainfall volume that runs off the area. Thus, this runoff coefficient has a value approaching unity for smooth, impermeable surfaces such as small areas covered by concrete surfaces. Conversely, C has a much

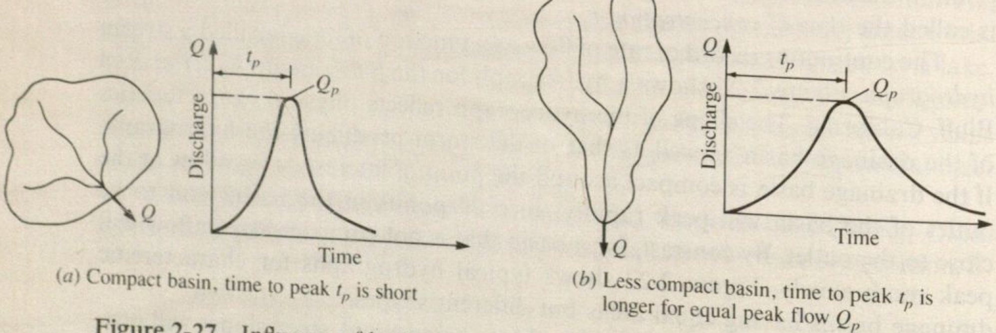


Figure 2-27 Influence of basin shape on runoff hydrograph. (*a*) Area A_2—Compact basin, short time to peak. (*b*) Area A_1—Less compact basin, longer time to peak

Table 2-9 Runoff Coefficients Recommended for Use in the Rational Equation (19)

Type of Area or Development	C
Type of development	
Urban business	0.70–0.95
Commercial office	0.50–0.70
Residential development	
Single-family homes	0.30–0.50
Condominiums	0.40–0.60
Apartments	0.60–0.80
Suburban residential	0.25–0.40
Industrial development	
Light industry	0.50–0.80
Heavy industry	0.60–0.90
Parks, greenbelts, cemetaries	0.10–0.30
Railroad yards, playgrounds	0.20–0.40
Unimproved grassland or pasture	0.10–0.30
Type of surface areas	
Asphalt or concrete pavement	0.70–0.95
Brick paving	0.70–0.80
Roofs of buildings	0.80–0.95
Grass-covered sandy soil	
Slopes 2% or less	0.05–0.10
Slopes 2% to 8%	0.10–0.16
Slopes over 8%	0.16–0.20
Grass-covered clay soils	
Slopes 2% or less	0.10–0.16
Slopes 2% to 8%	0.17–0.25
Slopes over 8%	0.26–0.36

smaller value for permeable surfaces and approaches 0 for sandy desert areas. Table 2-9 lists runoff coefficients for various surfaces.

Equation (2-1) is known as the rational equation primarily because it is dimensionally homogeneous in contrast to many other empirical equations for runoff that have been proposed and used in drainage design. The runoff coefficient C is a dimensionless constant. Intensity has the dimensions of depth (length) divided by time and is usually used in inches per hour or millimeters per hour. The area A is commonly used in acres or square miles. Thus, if intensity I is expressed in inches per hour and the area A is in acres, the runoff rate Q_p will be computed in acre-inches per hour, which is nearly equivalent numerically to cubic feet per second.

Since only peak rate of flow is computed by Eq. (2-1), use of the rational equation assumes that rainfall duration is at least equal to the time of concentration. If rainfall intensity were actually constant, a constant discharge would occur only after all storage and initial losses have been satisfied and flow is being contributed from all parts of the basin. The time of concentration for a drainage basin is made up of the longest combination of overland flow time plus the accumulated flow time in the stream channels to the outlet of the basin. To estimate times of concentration, several empirical expressions have been developed, including the following form of Kirpich's equation (14):

$$t_c = \left(\frac{3.35 \times 10^{-6} L^3}{h}\right)^{0.385} \tag{2-22}$$

where t_c = time of concentration in minutes
L = stream length in feet
h = difference in elevation in feet between the upper and lower limits of the drainage basin

A second empirical expression is the following equation modified from the original proposed by Hathaway (9):

$$t_c = \left(\frac{2Ln}{3\sqrt{S}}\right)^{0.47} \tag{2-23}$$

where t_c = time of concentration in minutes
L = channel length in feet
S = mean slope of the basin
n = Manning's roughness coefficient

Table 2-10 provides values of n to be used in Eq. (2-23).

In using the rational method, it is first necessary to compute the time of concentration. Either Eq. (2-22) or (2-23) may be used initially. However, because these equations are truly empirical, considerable care must be exercised before accepting their results. It is always advisable to make a check calculation by measuring the longest length of stream and determining the flow time in the stream using an estimated average velocity of flow. The average velocity of flow can be approximated by using the Manning equation (Eq. 4-7a). A reasonable assumption for the roughness coefficient must be made using values from

Table 2-10 Values of the Roughness Coefficient n to be Used in Eq. (2-23)

Surface	n
Smooth pavements	0.02
Bare packed soil, free of stones	0.10
Poor grass cover or moderately rough surface	0.20
Average grass cover	0.40
Dense grass cover	0.80

Table 4-1. One-dimensional flow with a depth of 6 to 12 in. can be assumed in the velocity calculation. The velocity calculation is described in Sec. 4-2 of Chapter 4.

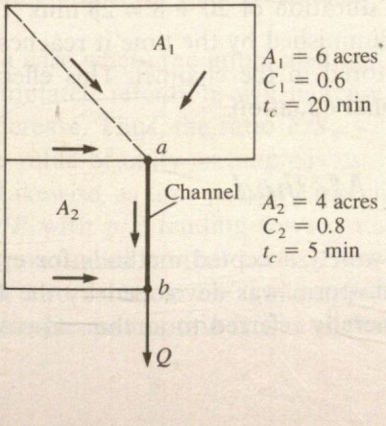
Once a time of concentration has been computed and accepted, a precipitation intensity is chosen from the previously determined intensity-duration values, such as the graph shown in Fig. 2-16 using the calculated time of concentration as the duration. Incorporating the drainage area, the determined rainfall intensity, and the properly chosen runoff coefficient in Eq. (2-1) yields the peak rate of runoff.

Because of its simplicity, the rational equation is widely used, particularly for calculations of urban runoff. However, there are some inherent inaccuracies in the use of the equation:

1. The runoff coefficient C cannot have a constant value throughout the storm, since as we mentioned, certain storages must be filled by rainfall before runoff can begin. Moreover, antecedent moisture conditions govern the initial infiltration rate.
2. In computing a peak flow from a precipitation intensity of a given return period, it is tacitly assumed that the return interval of the peak flow is the same as that of the chosen precipitation intensity. This is not strictly true, but for the general design range up to approximately 20 yr, it has been found to be approximately true (19).

Because the rational equation assumes that rainfall duration is at least equal to the time of concentration, it should not be used for areas greater in size than about 1 sq mi. For larger areas the entire drainage area almost certainly will not be covered by rainfall of equal intensity, thus violating the assumptions inherent in the rational equation. Other methods presented in the next section should be used for larger areas.

EXAMPLE 2-11 Use the rational equation to find the 10-yr peak rate of runoff for the two areas with the characteristics shown in the accompanying figure and for the channel at point b. The arrows show that all flow from area A_1



enters the channel at point a, and that all flow from area A_2 enters the channel at point b. Assume that the intensity-duration-frequency curves of Fig. 2-16 are applicable for the site, and that the time of flow from a to b is 8 min. *Note*: In using the rational equation, the units of Q are dependent on the units used for I and A.

SOLUTION Figure 2-16, page 50, shows the intensity of rainfall is 1.2 in. per hr, for a duration of 20 min. For area A_1, the peak rate of runoff occurs when the storm duration is equal to the time of concentration, t_c (20 min). Thus, the peak rate of flow from area A_1 into the channel at a is

$$Q_a = CIA = 0.6(1.2)6 = 4.3 \text{ acre-in./hr}$$

$$= 4.3 \text{ cfs}$$

Similarly, for area A_2, the peak rate of runoff entering the channel at b is

$$Q_b = 0.8(2.3)4 = 7.4 \text{ cfs}$$

since the intensity of rain is 2.3 in. per hr for a duration (time of concentration, t_c) of 5 min.

To find the peak rate of flow at b, we must consider both areas simultaneously. The critical time of concentration for area A_1 plus the time of flow from a to b is

$$t_b = 20 + 8 = 28 \text{ min}$$

We must add the rate of flow from area A_2 for a duration of 28 min to the peak rate of flow from area A_1 for a duration of 20 min. Thus, the peak flow at b is

$$Q_b = 4.3 + 0.8(1.0)6 = 4.3 + 4.8 = 9.1 \text{ cfs}$$

because the rainfall intensity for a 28-min duration is 1.0 in. per hr. We use the peak rate of flow from area A_1 for the 20-min duration because it takes that peak inflow from A_1 8 min to flow from a to b, at which point it combines with the flow coming from area A_2 after a duration of 20 + 8 = 28 min. The peak flow at point a is actually somewhat diminished by the time it reaches point b because part of the flow volume is stored in the channel. This effect, called routing, is neglected in using the rational equation. ■

Soil-Cover Complex Method

One of the more recent and widely accepted methods for estimating the amount of runoff from a given rainstorm was developed by the U.S. Soil Conservation Service (SCS) and is generally referred to as the soil-cover complex method, or more commonly, the curve-number method (34). The method is used where runoff records are not available and assumes that runoff can be determined directly in terms of a single parameter called a curve number (CN), a quantitative parameter for the drainage area of interest. Values of the curve number, and the method in general, were developed based on field measurements of the amount of runoff from drainage areas for which the rainfall, soil characteristics, cover, and usage were studied. The curve number attempts to account for both the *initial abstraction* I_a of rainfall (interception and depression storage plus the amount of infiltration that occurs before runoff begins) and the infiltration rate after runoff begins. Values of the curve number CN, have been developed and evaluated in terms of soil type, soil cover, land use, hydrologic condition, and antecedent moisture. Table 2-8, page 62, classifies different soils into groups in terms of their minimum infiltration rate, and Table 2-11, page 72, lists CN values for each soil group in terms of land use and hydrologic condition. The CN values of Table 2-11 are for an antecedent moisture condition, called condition II, which is described by the SCS as a condition that has been found to frequently precede the occurrence of maximum annual floods.

The analytical development of the curve-number method is empirical and is based on the observed behavior of runoff as a function of precipitation. If accumulated runoff is plotted against accumulated precipitation, the initial abstraction I_a (interception plus depression storage) causes the accumulated runoff to initially be zero because I_a is equal to the accumulated precipitation before runoff begins. As time passes and precipitation continues, the interception and depression storage volumes are filled, and if the rainfall rate is greater than the infiltration capacity of the soil, surface runoff begins. If the rainfall intensity continues to be larger than the infiltration capacity, the runoff rate will also increase. The SCS curve number method assumes that, as time passes and the rainfall rate continues to exceed the infiltration capacity, the curve developed by plotting accumulated runoff versus accumulated precipitation will become parallel to a 45° line (on an arithmetic plot). That is, this model assumes that the infiltration rate ultimately becomes zero, and thus, the incremental increase in rainfall excess (surface runoff) becomes equal to the incremental rainfall. This assumption, of course, is not valid, but it does make the analysis more simple and does yield reasonable results, particularly for short-duration storms.

For a case where the initial abstraction is zero ($I_a = 0$), it is known that F, the accumulated retention, will increase with time but at an ever-decreasing rate of increase. Thus, the ratio F/S_* will start with a zero value and increase toward a value of unity as time passes. Here S_* is the potential maximum retention. Likewise, as time passes, p (precipitation excess) will increase with time as will p/P with p/P tending toward unity as t becomes very large (P is the potential maximum rainfall excess). Since both of these ratios, F/S_* and p/P, start with zero values at $t = 0$ and approach unity at large values of time, one possible simple assumption is that their values are equal at all times:

Table 2-11 Curve Numbers for Soil Groupings in Terms of Use for Antecedent Moisture Condition II (34)

Cover			Hydrologic Soil Group			
Land Use	Treatment or Practice	Hydrologic Condition	A	B	C	D
Fallow	Straight row	—	77	86	91	94
Row crops	Straight row	Poor	72	81	88	91
	Straight row	Good	67	78	85	89
	Contoured	Poor	70	79	84	88
	Contoured	Good	65	75	82	86
	Contoured and terraced	Poor	66	74	80	82
	Contoured and terraced	Good	62	71	78	81
Small Grain	Straight row	Poor	65	76	84	88
	Straight row	Good	63	75	83	87
	Contoured	Poor	63	74	82	85
	Contoured	Good	61	73	81	84
	Contoured and terraced	Poor	61	72	79	82
	Contoured and terraced	Good	59	70	78	81
Close-seeded Legumes* or Rotation Meadow	Straight row	Poor	66	77	85	89
	Straight row	Good	58	72	81	85
	Contoured	Poor	64	75	83	85
	Contoured	Good	55	69	78	83
	Contoured and terraced	Poor	63	73	80	83
	Contoured and terraced	Good	51	67	76	80
Pasture or Range		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
	Contoured	Fair	25	59	75	83
	Contoured	Good	6	35	70	79
Meadow		Good	30	58	71	78
Woods		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads		—	59	74	82	86
Roads						
Dirt†		—	72	82	87	89
Hard surface†		—	74	84	90	92

* Close-drilled or broadcast-seeded

† Including right-of-way

$$\frac{F}{S_*} = \frac{p}{P} \tag{2-24}$$

In years of extensive use, the assumption of the equality of Eq. (2-24) has been shown to be reasonable. However, the assumption does lead to inaccuracies about infiltration rates, particularly for storms of several days duration.

For Eq. (2-24), S_* is assumed to be a constant for a given storm and is the maximum that can occur if the storm continues indefinitely. The accumulated retention F is the difference between the potential accumulated rainfall excess and the actual. Thus,

$$F = P - p \tag{2-25}$$

Equation (2-24) can therefore be written as

$$\frac{P - p}{S_*} = \frac{p}{P} \tag{2-26}$$

and can be solved for p as

$$p = \frac{P^2}{P + S_*} \tag{2-27}$$

Equation (2-27) represents the relationship between accumulated excess precipitation and accumulated precipitation for the particular case where the initial interception and depression storage volumes are zero.

If the initial abstraction I_a (initial interception plus depression storage) is considered, Eq. (2-24) can be modified as

$$\frac{F}{S} = \frac{p}{P - I_a} \tag{2-28}$$

where S is the sum of S_* and the initial abstraction, or

$$S = S_* + I_a \tag{2-29}$$

For Eq. (2-28) to hold, F/S and $p/(P - I_a)$ must be less than unity. The equivalents to Eqs. (2-25) and (2-26) are then

$$F = (P - I_a) - p \tag{2-30}$$

and $$\frac{(P - I_a) - p}{S} = \frac{p}{P - I_a} \tag{2-31}$$

Finally, Eq. (2-31) can be solved for the precipitation excess to give

$$p = \frac{(P - I_a)^2}{(P - I_a) + S} \tag{2-32}$$

Equation (2-32) provides a relationship between accumulated rainfall excess and accumulated rainfall for particular values of the initial abstraction and the potential maximum retention.

Investigations of small drainage areas have shown that a relationship between the initial abstraction and the potential maximum accumulated retention can be approximated by

$$I_a = 0.2S \tag{2-33}$$

Using Eq. (2-33), the precipitation excess can be expressed in terms of accumulated precipitation depth P as

$$p = \frac{(P - 0.2S)^2}{P + 0.8S} \tag{2-34}$$

At this point, the curve number CN is introduced to approximately relate hydrologic parameters to drainage-basin parameters. CN is defined in terms of S such that a number CN value of 100 will yield 100% surface runoff ($p = P$). The empirical relationship between CN and S is

$$CN = \frac{1000}{10 + S} \tag{2-35}$$

Equations (2-34) and (2-35) have been combined and plotted in Fig. 2-28 for ready use. In that figure, the curve number 100 corresponds to a case of zero initial abstraction and zero infiltration, and the rate of runoff is equal to the rate of rainfall. The curve numbers of less than 100 correspond to soil conditions where the initial abstraction is greater than zero. For example, consider the curve number 70. Figure 2-28 shows that for such a soil, the initial abstraction I_a is approximately 1 in. (no runoff occurs until rainfall exceeds approximately 1 in.) Then, as the rainfall continues (P increases), the rainfall excess p becomes an increasingly larger portion of the rainfall. This process indicates that the infiltration rate is steadily decreasing with additional rainfall, which is in agreement with Horton's concept of infiltration as shown in Fig. 2-24. For the curve number 70, Fig. 2-28 indicates that rainfall excess (runoff) would be approximately 6.2 in. for a total rainfall of 10 in.

The variable S, used in developing Eq. (2-34) is called the potential maximum retention and includes all the precipitation that does not run off (interception, depression storage, and infiltration). Examining Eq. (2-34) shows that

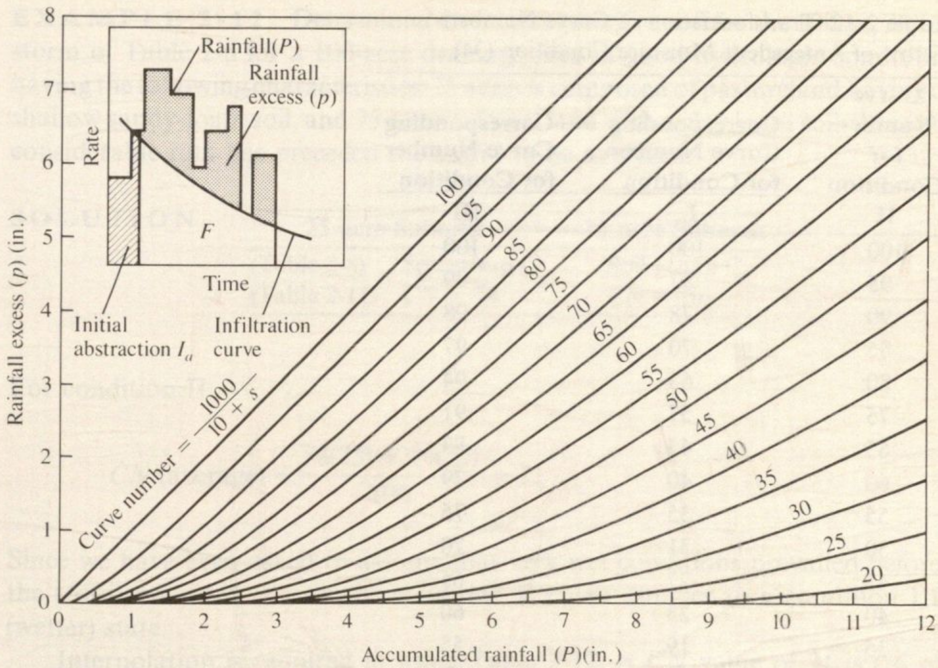


Figure 2-28 Rainfall excess in terms of rainfall and curve number for $I_a = 0.2S$ (34)

rainfall excess (runoff) will equal precipitation if $S = 0$, which is the condition for $CN = 100$. Equation (2-35) shows that $CN = 100$ for $S = 0$. Thus, the runoff model given by Eqs. (2-34) and (2-35) is seen to be very much empirical but generally in agreement with the accepted concept of infiltration and other losses of rainfall.*

The curve numbers given in Table 2-11 are for an antecedent moisture content referred to as condition II. Condition II is a state that has been found to occur frequently before many large storms. If drier conditions or wetter conditions occur before the storm, then less or more runoff will occur, respectively. An empirical method has been developed to obtain curve numbers for drier (condition I) or wetter (condition III) antecedent moisture. The conversion is given in Table 2-12, page 76.

The procedure for using the curve number method is as follows:

1. Study the soil, cover, and use characteristics of the drainage area. Then select the proper soil group from Table 2-8, page 62. For this process, it may be desirable to subdivide the drainage area into subareas having

* To gain further insight into the development of this useful concept, refer to the *National Engineering Handbook* of the U.S. Soil Conservation Service (34).

Table 2-12 Transformation of Curve Numbers in Terms of Antecedent Moisture Condition (34)

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

reasonably homogeneous conditions and a separate soil-group classification may be necessary for each.

2. Select a *CN* value from Table 2-11, where the definition of "hydrologic conditions" is as follows:
   Poor: Heavily grazed, no mulch, or less than 1/2 of area with plant cover
   Fair: Moderately grazed with plant cover on 1/2 to 3/4 of the area
   Good: Lightly grazed with plant cover over more than 3/4 of the area
3. If the runoff event to be analyzed is for a drier (less conservative in terms of runoff volume) or a wetter (more conservative) estimate, select a different *CN* value using Table 2-12. If the drainage area has been subdivided, a separate *CN* value will be required for each subarea.
4. Determine an average *CN* value for the entire area by calculating an area-weighted average of subarea values.
5. Tabulate the accumulated precipitation of the storm from which runoff is to be estimated.
6. Estimate the accumulated runoff (rainfall excess) using the selected *CN* and Fig. 2-28.

EXAMPLE 2-12 Determine 2-hr increments of runoff for the 100-yr, 24-hr storm of Table 2-6 for a 100-acre drainage area in good hydrologic condition having the following characteristics: 25 acres is composed of pastureland having a shallow sandy-loam soil, and 75 acres is forest land with a clay loam soil. Assume considerable rain has preceded the storm to be analyzed.

SOLUTION

	25-acre Subarea	75-acre Subarea
(Table 2-8)	Soil group C	Soil group C
(Table 2-11)	$CN = 74$	$CN = 70$

For condition II:

$$CN \text{ (average)} = \frac{74(25) + 70(75)}{100} = 71$$

Since we have been asked to assume that very wet conditions prevailed before the storm, we must correct the condition II curve number to a condition III (wetter) state.

Interpolation is required in using Table 2-12. A CN value of 71 is 0.6 of the interval between CN values of 65 and 75. Thus, for condition III, the desired value is 0.6 of the interval between 83 and 91. For condition III (From Table 2-12):

$$CN = 83 + (91 - 83)0.6 = 88$$

The values of time and accumulated precipitation in the accompanying table come from columns 1 and 5, respectively, of Table 2-6. Each value of runoff

Time (hr)	Accumulated Precipitation (in.)	Runoff (in.)
0	0	0
2	0.09	0
4	0.28	0
6	0.56	0.01
8	0.98	0.25
10	4.83	3.55
12	5.49	4.15
14	6.05	4.70
16	6.29	4.95
18	6.53	5.20
20	6.77	5.40
22	6.96	5.62
24	7.05	5.70

for each value of accumulated precipitation is read from Fig. 2-28 using a curve number of 88 (interpolated between $CN = 85$ and $CN = 90$).

Thus, the total runoff from the 100-yr, 24-hr storm is 5.70 in., and 1.35 in. (7.05 − 5.70) was lost to interception, depression storage, and infiltration. The runoff was 5.70/7.05 = 0.81 or 81% of rainfall. ■

The CN method is a reasonable and widely used representation of the runoff process and the only procedure that provides a means of assessing runoff on the basis of drainage-area characteristics without recorded data on runoff and rainfall for actual storms. In practice, the procedure has a basic fault in that it theoretically assumes that the infiltration rate eventually goes to zero. Theoretically, the actual infiltration rate should probably approach a constant minimum rate, as is indicated in Table 2-8. Thus, the curve number method may be slightly conservative when used for predicting runoff from long-duration storms. Because of this limitation, its use is probably questionable for areas greater than perhaps 5 to 10 sq mi since drainage areas that size or larger have times of concentration that may be longer than the time required for the infiltration capacity to reach a minimum. The method, however, has been widely used for much larger areas.

2-7 Streamflow

As we discussed in Sec. 2-2, hydrologic design criteria for an engineering project usually requires the determination of a peak streamflow that will be equalled or exceeded on the average only once in a specified number of years. As we mentioned in Sec. 2-4, that peak flow can be determined through statistical means, provided sufficient streamflow data are available for the watershed in question. More often, however, it is necessary to determine a peak rate of flow or a streamflow hydrograph when a peak rainfall rate or the variable rainfall rate of a design storm is known, but streamflow records are not available. In Sec. 2-6, we discussed the rational method, by which the peak rate of flow from a small drainage area can be determined based on the time of concentration and the degree of imperviousness of the surface. In this section, we discuss the development of the streamflow hydrograph, given a particular rain storm.

Hydrographs

The response of each watershed to rainfall tends to be unique and physically quite complex, but the general characteristics can be readily described. Figure 2-29 represents streamflow occurring at a particular location on a stream as a result of rainfall on the drainage area. The hydrograph is generally divided into the baseflow, the rising limb, the peak segment, and the falling limb, as shown in Fig. 2-29. Segment AB, the baseflow segment, is a recession curve,

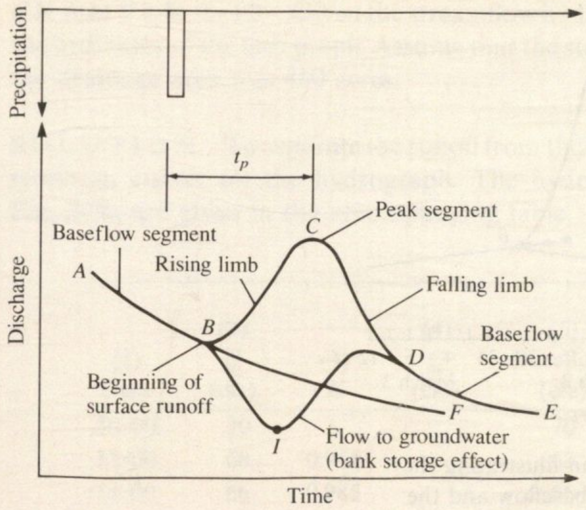


Figure 2-29 Hypothetical single-peaked hydrograph

which is generally due to flow of groundwater into the stream when no surface runoff is occurring at the time. As runoff begins, water first flows overland to the stream and then flows downstream to the point of measurement. The elapsed time between the occurrence of rainfall and the time the peak streamflow occurs is the *time to peak* t_p and is generally measured from the centroid of the rainstorm to the peak flow of the hydrograph.

The hydrograph shown in Fig. 2-29 is typical of that produced by a short rainfall of nearly uniform intensity. Line *BF* represents the streamflow that would have occurred had no rainfall occurred. Point *B* is the point in time at which surface runoff first reaches the observation point. At point *C*, the streamflow has peaked, rainfall has stopped, and the rate of runoff begins to decrease.

Once streamflow begins to increase (point *B*), the water depth in the stream increases as well. This increase in depth may begin to counteract the flow of groundwater into the channel. If the depth increases enough, water will flow from the stream into the bank creating *bank storage*. This flow into the bank effectively reduces the baseflow as shown by curve *BID* in Fig. 2-29. Once streamflow passes the peak, the flow into the bank begins to decrease (18). At point *D*, the surface runoff has ceased, and the streamflow follows a recession curve approximately geometrically similar to that occurring before the storm.

The streamflow hydrograph integrates all the physical properties of the drainage area (for example, soil, size, shape, slope) that govern the process of runoff. For that reason, it has been used as a single characteristic of the drainage area in what has been called the *unit hydrograph* (20). The unit hydrograph, or unit graph, is defined as the timewise distribution of 1 in. of surface runoff from a given drainage area for a particular rainfall duration. Theory of the unit graph also implies that two storms of equal duration but different intensities will pro-

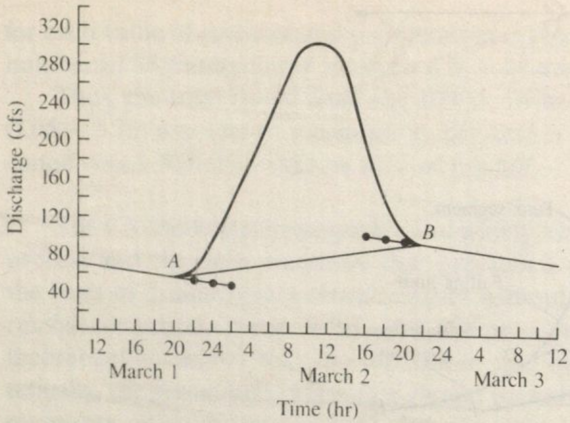


Figure 2-30 Hydrograph used in illustrating the separation of surface runoff and baseflow and the construction of a unit hydrograph (Points plotted near A and B are those computed in column 4 of Example 2-13)

duce hydrographs having similar shapes but with flow ordinates in direct proportion to the total volume of runoff.

Two procedures are used for constructing a unit hydrograph. The first to be discussed involves developing a unit hydrograph using an actual stream hydrograph for which recorded information is available relative to the rainfall that produced the hydrograph. Since the unit graph represents surface runoff, it is necessary to separate baseflow and surface runoff for the given hydrograph. This process is subject to much interpretation, but a reasonable method assumes that a straight line can be drawn from the beginning of runoff (point A in Fig. 2-30) to the end of runoff (point B in Fig. 2-30). Locating points A and B involves analyzing the recession curve. The recession curve must be extrapolated forward from point A and backward from point B, as is shown in Fig. 2-30. One method of doing this assumes that the recession curve can be expressed as

$$Q_{t+\Delta t} = Q_t \quad K^{\Delta t} \tag{2-36}$$

where Q_t = the flow rate at time t
$Q_{t+\Delta t}$ = the flow rate at time $t + \Delta t$
K = an empirical constant
Δt = an increment in time

Equation (2-36) is used to evaluate the coefficient K using the recorded values of Q once Δt is chosen. It is convenient to choose $\Delta t = 1$. In computing values of K, it is important to use values of Q only from those segments of the hydrograph that actually represent recession curves. The following example illustrates the use of Eq. (2-36).

EXAMPLE 2-13 Given the streamflow hydrograph of Fig. 2-30, determine the ordinates of the unit graph. Assume that the storm duration was 4 hr and that the drainage area was 460 acres.

SOLUTION To separate the runoff from the baseflow, we must examine the recession curves on the hydrograph. The hydrograph ordinates taken from Fig. 2-30, are given in the accompanying table.

(1) Time	(2) Q (cfs)	(3) K	(4) Q (cfs)	(5) Baseflow (cfs)	(6) Surface Runoff (cfs)	(7) Unit Graph Ordinate (cfs)
10:00	70			70	0	0
12:00	68	0.985		68	0	0
14:00	66	0.985		66	0	0
16:00	64	0.985		64	0	0
18:00	62	0.984		62	0	0
20:00	60	0.984		60	0	0
22:00	64	1.030	58	61	3	0.5
24:00	94	1.210	56	64	30	4.9
2:00	140		55	67	73	11.9
4:00	190			70	120	19.5
6:00	238			73	165	26.9
8:00	280			76	204	33.2
10:00	297			78	219	35.7
12:00	300			80	220	35.8
14:00	280			83	197	32.1
16:00	210	0.866	104	86	124	20.2
18:00	142	0.822	101	89	53	8.6
20:00	109	0.876	98	92	17	2.8
22:00	95	0.933		95	0	0
24:00	92	0.984		92	0	0
2:00	89	0.984		89	0	0
4:00	87	0.989		87	0	0
6:00	84	0.983		84	0	0
8:00	82	0.988		82	0	0
10:00	79	0.982		79	0	0
12:00	77	0.987		77	0	0
				Totals	1425	232.1

Values of K have been determined using Eq. (2-36) with $\Delta t = 2$ and are shown in column 3. Note that the value of K changes drastically between 20:00 and 24:00 on March 1. The consistent values of K average 0.985. The values at 16:00 and 18:00 of March 2, being much smaller, indicate that they are within the period when surface runoff is in recession. To determine the points where runoff begins (point A) and ends (point B), Eq. (2-36) is used to extend the recession curves both ahead and backward in time. The calculated values are shown in column 4 and are plotted in Fig. 2-30. Points A and B are thus located by the point at which the extrapolated recession curves deviate from the hydrograph. A straight line AB is then drawn to approximately separate surface runoff from baseflow. Runoff can be calculated as the ordinate between the hydrograph and the straight line and is shown in column 6. Column 5 shows the baseflow or the ordinates below the straight line AB. The total surface runoff volume is calculated by multiplying the sum of column 6 by 2, the increment in time used in calculation.

$$1425 \times 2 = 2850 \text{ cfs-hours} = 118.75 \text{ cfs-days}$$
$$= 2826.4 \text{ acre-in.}$$

The average depth of precipitation excess over the drainage area is the runoff volume divided by the area or $2826.4/460 = 6.14$ in. The unit graph ordinates are obtained by dividing the original runoff ordinates by 6.14, the depth of runoff. Thus, the total of the unit graph ordinates (as given in column 7) should indicate a total runoff of 1.0 in. Check this:

$$\text{Unit graph volume} = 232.1 \times 2 = 464.2 \text{ cfs-hours} = 19.3 \text{ cfs-days}$$
$$= 460 \text{ acre-in.}$$

■

Once completed, the unit hydrograph can be used to develop a streamflow hydrograph for increments of rainfall excess having the same duration as that for which the unit graph was developed but having depths of rainfall greater or less than unity. Unfortunately, storms of the same duration and average intensity do produce different unit graphs due to different directions of storm travel, sequence of rainfall increments, antecedent moisture conditions, and seasons. A certain amount of nonlinearity exists in the process although the assumption is made that the hydrograph ordinates are linearly proportional to rainfall excess. To offset this tendency, several unit graphs should be developed and an average unit graph constructed from them. Figure 2-31 illustrates this process. In developing the average unit graph, the times to peak and the peak flows are averaged, and the average graph is then sketched. The resulting ordinates should be checked and, if necessary, adjusted to make certain that the average unit graph does contain a runoff volume of 1 in.

Synthetic Unit Graphs

Because of the lack of either rainfall or average streamflow records for many drainage basins of interest, constructing a unit graph by the foregoing

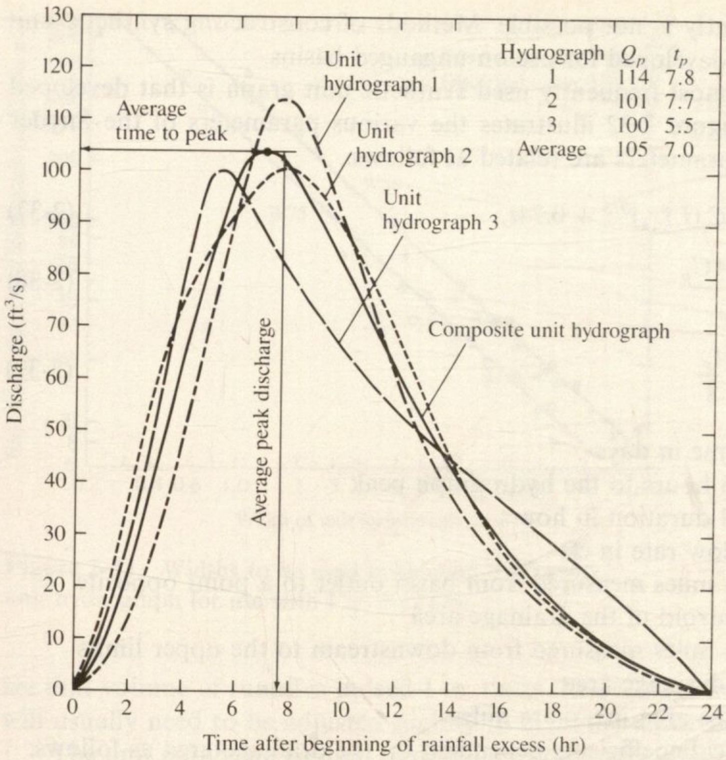


Figure 2-31 Averaging time to peak and peak rate of flow to develop composite unit hydrograph

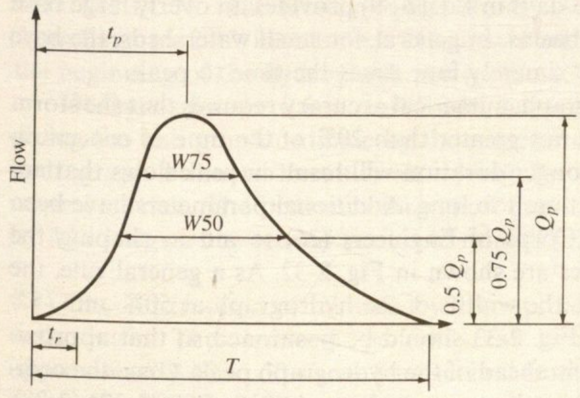


Figure 2-32 Snyder unit hydrograph

procedures frequently is not possible. Methods of constructing synthetic unit graphs have been developed for use on ungauged basins.

Probably the most frequently used synthetic unit graph is that developed by Snyder (22). Figure 2-32 illustrates the various parameters of the Snyder unit graph. The parameters are related as follows:

$$t_p = 0.95C_t(LL_{ca})^{0.3} + 0.74t_r \tag{2-37}$$

$$Q_p = \frac{640AC_p}{t_p} \tag{2-38}$$

$$T = 3 + \frac{t_p}{8} \tag{2-39}$$

where T = base time in days
t_p = time in hours to the hydrograph peak
t_r = rainfall duration in hours
Q_p = peak flow rate in cfs
L_{ca} = stream miles measured from basin outlet to a point opposite the centroid of the drainage area
L = stream miles measured from downstream to the upper limits of the drainage area
A = drainage area in square miles
C_p, C_t = empirical coefficients dependent on the drainage area as follows:

Area	C_p	C_t
Mostly rolling hills such as Appalachian Highlands	0.63	2.0
Steep slopes such as those in the mountains of Southern California	0.94	0.4
Very flat slopes such as those along the Eastern Gulf of Mexico	0.61	8.0

Equation (2-39) provides a reasonable estimate for very large drainage basins. However, the constant 3 days in Eq. (2-39) provides an overly large base for unit hydrographs for small basins. In general, for small watersheds, the base time can be estimated as approximately four times the time to peak.

In constructing any hydrograph, numerical accuracy requires that the storm duration for the unit graph be not greater than 20% of the time of concentration or time to peak. Using a longer duration will result in peak flows that are too high and times to peak that are too long. Additional parameters have been developed by the U.S. Army Corps of Engineers (26) to aid in shaping the Snyder unit graph. These values are shown in Fig. 2-32. As a general rule, the W_{50} and W_{75} widths, which are the widths of the hydrograph at 50% and 75% of Q_p respectively, as given in Fig. 2-33 should be positioned so that approximately one third of their width is ahead of the hydrograph peak. Once the ordinates of the Snyder unit graph have been approximated, using Eqs. (2-37), (2-38), and (2-39) and the parameters in Fig. 2-33, the unit graph must be checked to

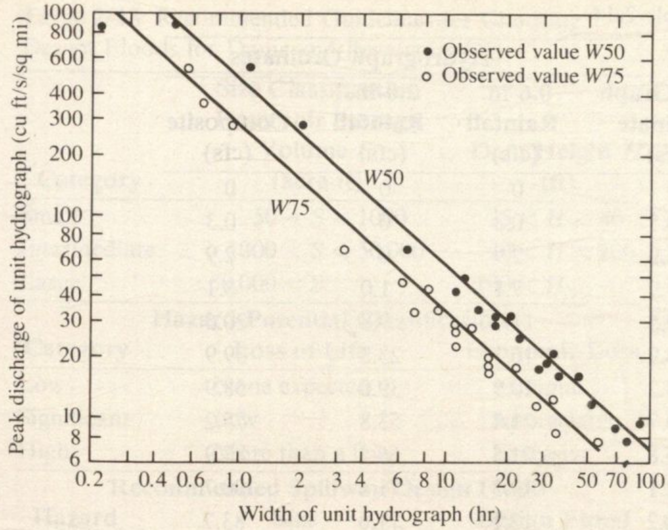


Figure 2-33 Widths to be used in shaping the Snyder unit hydrograph for use with Fig. 2-32 (26).

see that volume of runoff is indeed 1 in. times the drainage area. The ordinates will usually need to be adjusted slightly to meet the unit runoff condition.

The unit graph can be used to develop a composite streamflow hydrograph for a general storm. The process is illustrated in Example 2-14.

EXAMPLE 2-14 Develop a composite hydrograph for an 8-hr storm having two successive 4-hr periods of rainfall excess of 0.6 in. and 2.0 in., respectively. The drainage area is 460 acres, and the unit graph is that computed in Example 2-13.

SOLUTION The table, on page 86, shows the unit hydrograph ordinates (from Example 2-13) in column 2. The time in column 1 is the time measured from the beginning of the hydrograph rise (point A in Fig. 2-29, page 79).

Ordinates for the 0.6-in. and 2.0-in. rainfall excess are obtained by multiplying the unit graph ordinate by the respective rainfall depth. The composite hydrograph ordinates result from adding the two incremental ordinates for common times. The ordinates for runoff due to the 2.0-in. rainfall are lagged by 4 hr because the second period of rainfall follows the first. ■

The construction of the unit hydrograph assumes that rainfall covers the entire basin. Thunderstorms generally can be assumed to cover areas up to approximately 40 to 50 sq mi. For general frontal storms, the coverage can be as much as 2000 to 3000 sq mi. In general, use of the unit graph should be limited to drainage areas less than these limiting sizes for the respective storm types.

Table for Example 2-14

Time (hr)	Unit Graph Ordinate (cfs)	Hydrograph Ordinates 0.6-in. Rainfall (cfs)	2.0-in. Rainfall (cfs)	Composite (cfs)
0	0	0	0	0
2	0.5	0.3	0	0.3
4	4.9	2.9	0	2.9
6	11.9	7.1	1.0	8.1
8	19.5	11.7	9.8	20.5
10	26.9	16.1	23.8	39.9
12	33.2	19.9	39.0	58.9
14	35.7	21.4	53.8	75.2
16	35.8	21.5	66.4	87.9
18	32.1	19.3	71.4	90.7
20	20.2	12.1	71.6	83.7
24	8.6	5.2	64.7	69.0
26	2.8	1.7	40.4	42.1
28	0	0	19.2	19.2
30			5.6	5.6
32			0	0

Probable Maximum Flood Determination

Peak flow of floods of a certain frequency can be calculated statistically according to methods presented in Sec. 2-4. However, the development of a hydrograph of particular return period is somewhat different.

As we mentioned in Sec. 2-5, major projects such as dams whose failure could cause a flood that would jeopardize many people must be designed to be safe from failure during the probable maximum flood (PMF) created by the probable maximum precipitation. Other projects such as nuclear power plants or dikes to contain radioactive material must also be designed to be safe from failure during the probable maximum flood.

In recent years, the safety of dams has become an important issue. To develop a consistent method by which a proper design flood can be chosen, the U.S. Army Corps of Engineers developed the guidelines for design floods for dams shown in Table 2-13.

Table 2-13 recommends that the design flood be chosen in accordance with the hazard that might be created by a dam and the reservoir of water stored behind the dam. In general, hazard is difficult to define quantitatively. Table 2-13 provides guidelines for quantitatively assessing the hazard in terms of reservoir volume and dam height as well as potential loss of life or economic value that might occur if the dam were to fail. Certainly these measures are

Table 2-13 Recommended Guidelines for Choosing Design Floods for Dams and Reservoirs (25)

	Size Classification	
Category	**Reservoir Storage Volume S (acre-ft)**	**Dam Height H (ft)**
Small	$50 < S < 1000$	$25 < H < 40$
Intermediate	$1000 < S < 50{,}000$	$40 < H < 100$
Large	$50{,}000 < S$	$100 < H$
	Hazard-Potential Classification	
Category	**Loss of Life**	**Economic Loss**
Low	None expected	Minimal
Significant	Few	Appreciable
High	More than a few	Excessive
	Recommended Spillway Design Flood	
Hazard	**Size**	**Design Flood**
Low	Small	50 to 100 yr
	Intermediate	100 yr to $\frac{1}{2}$ PMF
	Large	$\frac{1}{2}$ PMF to PMF
Significant	Small	100 yr to $\frac{1}{2}$ PMF
	Intermediate	$\frac{1}{2}$ PMF to PMF
	Large	PMF
Large	Small	$\frac{1}{2}$ PMF to PMF
	Intermediate	PMF
	Large	PMF

inexact, but they do provide guidance for assessing the relative need for large or small design floods. Thus, a 35-ft high dam that stores 900 acre-ft of water upstream from an unoccupied valley could have freeboard and spillway capacity designed for a 50-yr inflow flood. However, at least one half of a probable maximum flood should be used if the dam were located upstream from a community where many people might be killed by the flood resulting from a dam failure.

Return periods of design floods used for other structures and developments that have become more or less standard are indicated in Table 2-14.

Determination of a probable maximum flood follows the same guidelines used to develop a flood hydrograph produced by a given storm:

1. The probable maximum precipitation (PMP) is chosen using methods discussed in Sec. 2-5.
2. A design storm is developed using the total PMP determined in step 1 and procedures discussed in Sec. 2-5.
3. Rainfall excess is calculated using procedures discussed in Sec. 2-6.

Table 2-14 Typical Recurrence Intervals for Design Floods for Various Projects

Type of Project or Structure	Return Period (yr)
Highway culverts	
Rural roads	5–10
Secondary highways	10–25
Interstate highway bridges	100
Airfield drainage	5
Urban storm drainage	2–10
Flood-control levees	
Urban areas	100–250
Agriculture areas	2–50
Design grade elevation	
Industrial plant	50–100
Coal-fired power plant	50–100
Nuclear power plant	PMF

4. The unit hydrograph is chosen for a storm duration not to exceed 20% of the time of concentration for the drainage area of interest.
5. A unit hydrograph is developed using procedures discussed in this section.
6. The unit hydrograph is used to generate hydrographs for each increment of rainfall excess as shown in Example 2-14 on hydrograph construction.
7. The composite hydrograph is constructed from the summation of the hydrographs generated in step 6 as was also done in Example 2-14.

Low-Flow Analysis

Section 2-4 dealt with the determination of recurrence intervals of peak flows or other hydrologic events. The low flows that a stream will experience are also of particular interest, but for an entirely different reason. If water is to be withdrawn from a stream for use in a project as a water supply, it is important for both engineering design and environmental reasons to know the flow rate at low flow and the estimated probability of the actual streamflow being equal to or less than a given magnitude.

In most areas of the world, it is now recognized that streamflow should be retained at or above a particular level, or the instream environment may suffer serious degradation. The instream environment may include resident fish, anadromous fish such as salmon or striped bass, and local resident aquatic life.

Low flows differ from peak flood flows specifically in that they may have a readily apparent value of zero. An ephemeral stream is one that has zero flow for part of most years. Nearly all streams in desert regions are ephemeral, and some are dry except for a few hours during and after a rainstorm. Because the ultimate minimum instantaneous flow of any stream may be zero, it is the duration of any particular flow rate that is important. For example, it is important in a city's planning to know for how many days flow in the stream that is their source of water may be equal to or below a certain value. Frequently, the 7-day low flow is compared between streams and is often quoted in hydrologic descriptions for environmental impact statements or other environmental or engineering documents. The *7-day low flow* represents the average low flow for a 7-day duration. To have complete meaning, it must be associated with a particular recurrence interval such as the 10-year, 7-day low flow.

To develop low-flow frequency curves, the records of average daily flows for the stream of interest are examined for various durations. The record is examined to find first the lowest flow of record for 1 day (the 1-day low flow), and then the second lowest for 1 day, and so on. The apparent probability of occurrence of the flow is calculated using Eq. (2-40) as:

$$P = \frac{M}{N+1} \tag{2-40}$$

where M = the order number (1 for the lowest)
N = the number of years of record.

Next, the 2-day average low flows are examined and ordered in the same fashion. The same procedure is repeated for all durations of interest. Figure 2-34 shows a set of low-flow duration curves calculated for the Eel River at Scotia, California. In the low-flow selections, it is important to understand

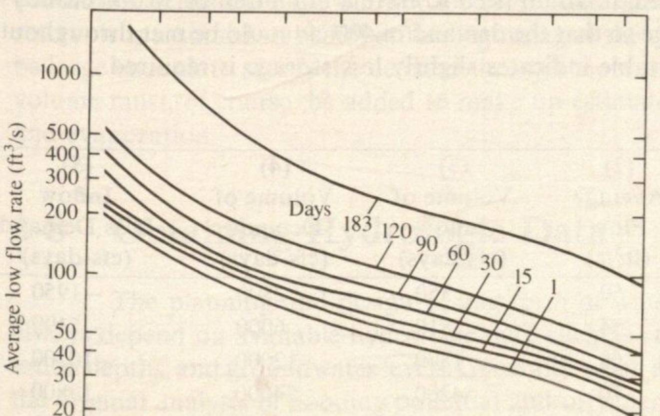


Figure 2-34 Low-flow-duration-frequency curves for the Eel River at Scotia, California 1912–1960 (21)

that each day in the record can appear in only one period for each duration. Thus, a record 700 days in length can be analyzed as not more than 100 7-day durations.

Nonsequential Drought

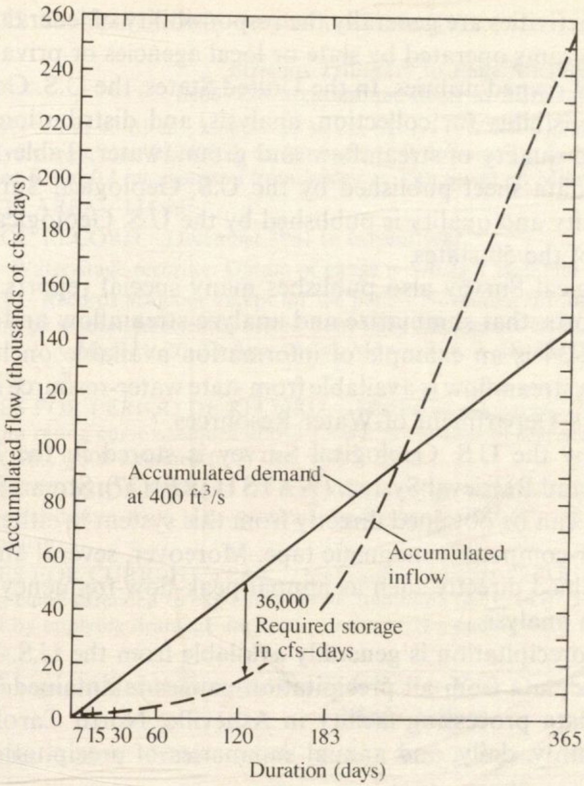
The set of low-flow duration curves can be used to size a reservoir that must be provided to ensure a sufficient supply of water during a drought period. The process is similar to the construction of a design storm of particular recurrence interval. If, for example, it is desired to size a reservoir to provide sufficient water during a 5-yr drought (the dry period that occurs on the average once in 5 yr) the procedure in the following example is followed.

EXAMPLE 2-15 Use the low-flow duration curves of Fig. 2-34 to determine the necessary size of a reservoir to provide an average flow of 400 cfs during a 5-yr drought.

SOLUTION Reading the 5-yr values of Fig. 2-34 provides the low-flow values shown in column 2 of the accompanying table. Column 3 is the accumulated volume of inflow to the reservoir, which is determined by multiplying the entries in column 1 by the flow entries in column 2. Column 4 is the accumulated demand at 400 cfs for the corresponding duration. The difference between inflow (column 3) and demand (column 4) volumes is shown in column 5. Accumulated inflow to the reservoir and accumulated demand are shown in the accompanying figure. The difference between the two curves is the storage required to meet the demand without a shortage. Thus, if the reservoir were full when the drought started, it would need to have a minimum of 36,000 cfs-days (71,405 acre-ft) of storage so that the demand of 400 cfs could be met throughout the drought. *Note*, the table indicates slightly less storage is required.

(1) Duration (days)	(2) Average Flow (ft^3/s)	(3) Volume of Inflow (cfs-days)	(4) Volume of Demand (cfs-days)	(5) Inflow Less Demand (cfs-days)
7	50	350	2800	−1950
15	54	810	6000	−5190
30	60	1800	12000	−10200
60	70	4200	24000	−19800
120	100	12000	48000	−36000
183	230	42090	73200	−31110
365	700	255500	146000	109500

■



The volumes used in Example 2-15 are in cfs-days for ease of calculation. The analysis shows that inflow to the reservoir under the assumed drought conditions is less than the outflow for more than one half a year. Thus, if the reservoir is assumed to be full at the beginning of the drought, its volume must be large enough to satisfy the demand through more than 120 days. Additional volume must, of course, be added to make up estimated losses due to seepage and evaporation.

2-8 Obtaining Hydrologic Data

The planning and design of any form of water-resources project will always depend on available hydrologic data such as streamflow rates, precipitation depths, and groundwater levels. Generally, when a major project is started, the original analysis of flooding potential and/or water availability will be performed using data collected at or near the site, but data-collection facilities will be established at pertinent locations in order to collect site-specific data. In any case, locating, obtaining, and analyzing the data will nearly always be an engineering responsibility.

Data-collection activities are generally the responsibility of federal agencies with cooperative programs operated by state or local agencies or private developers such as investor-owned utilities. In the United States, the U.S. Geological Survey has the responsibility for collection, analysis, and distribution of data on both quantity and quality of streamflow and groundwater. Table 2-15 is a copy of a summary data sheet published by the U.S. Geological Survey (10). Data on water quantity and quality is published by the U.S. Geological survey every year for each of the 50 states.

The U.S. Geological Survey also publishes many special reports, such as their Open-File Reports, that summarize and analyze streamflow and groundwater data. Figure 2-34 is an example of information available on low flow. Other information on streamflow is available from state water-resource agencies, such as the California Department of Water Resources.

Data obtained by the U.S. Geological Survey is stored in the National Water Data Storage and Retrieval System (WATSTORE) (27). Streamflow data for the United States can be obtained directly from this system in either printed form or on computer-compatible magnetic tape. Moreover, several analyses of the data can be obtained directly such as annual-peak-flow-frequency analysis or low-flow-duration analysis.

Information on precipitation is generally available from the U.S. National Weather Service. The data from all precipitation gauges maintained by NWS are stored at their data processing facility in Asheville, North Carolina, and are available in monthly, daily, and annual summaries of precipitation depth at each station.

Data on snow depth are available from state water-resource agencies and from the U.S. Soil Conservation Service in Washington, D.C.

In countries outside the United States, water-resource data are generally the responsibility of the national government. For the industrialized nations, data-collection systems are generally good, and records are readily available. For the less developed nations, water data vary greatly in quantity, quality, and availability.

2-9 Computer Programs for Hydrology

Use of modern digital computers is ideally suited to hydrologic calculations where, in general, a large amount of data must be handled and extensive digital calculations are necessary. Programs referred to as *watershed runoff models* have been developed that approximately simulate the part of the hydrologic cycle involving the generation of runoff. The Hydrologic Simulation Program-Fortran (HSPF) is a numerical model that continuously simulates not only surface runoff but the entire part of the hydrologic cycle involving interception, depression storage, evaporation, and infiltration (5). In using this program (or model), recorded average rainfall for the basin is used as input, and resulting runoff and streamflow are calculated. Simultaneous precipitation and runoff

Table 2-15 Example of Daily Streamflow Data (11)

Streams Tributary to Lake Michigan
04087138 Menomonee River at Milwaukee, WI

LOCATION—Lat 43°01′28″, long 87°57′36″, in SE 1/4 NW 1/4 sec. 36, T.7 N., R.21 E., Milwaukee County, Hydrologic Unit 04040003, on left bank 10 ft downstream from pedestrian walkway over the Menomonee River, 0.1 mi upstream from bridge at 35th Street, at Milwaukee

DRAINAGE AREA—134 mi^2

PERIOD OF RECORD—December 1981 to current year

GAUGE—Water-stage recorder. Datum of gauge is 576.23 ft National Geodetic Vertical Datum of 1929

REMARKS—Records are poor except for the period November through April 18, which is fair to good. Stage-discharge relation affected by seiche from Lake Michigan Oct. 1–8, 11–18, 21–31, Dec. 12, 13, Jan. 14, Feb. 2, 3, Mar. 21, 26, 27, Apr. 19–30, May 1, 3–6, 9–18, 27, 28, June 2 to Aug. 16, Aug. 19 to Sept. 5, Sept. 7–30

EXTREMES FOR PERIOD OF RECORD—Maximum discharge, 7240 ft^3/s Aug. 17, 1983, gauge height, 14.66 ft, from rating curve extended above 1500 ft^3/s on basis of four step-backwater determinations, *Q*10, *Q*50, *Q*100, *Q*500 obtained from Oct. 3, 4, 5, 1982

EXTREMES OUTSIDE PERIOD OF RECORD—High water of July 13, 1981, reached a stage of 13.16 ft, present datum, from high-water marks; discharge, 5910 ft^3/s, from rating curve extended as explained above

EXTREMES FOR CURRENT YEAR—Maximum discharge, 7240 ft^3/s Aug. 17, gauge height, 14.66 ft, from rating curve extended as explained above; minimum daily, 14 ft^3/s (seiche affected), Oct. 3, 4, 5, determined by applying drainage area ratio to the corresponding daily discharge for Menomonee River at Wauwatosa

Discharge, in Cubic Feet per Second, Water Year October 1982 to September 1983
Mean Values

Day	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep
1	15	690	72	62	33	104	508	65	155	90	16	22
2	15	335	1180	53	46	108	2390	254	112	38	17	20
3	14	147	983	43	36	120	1650	111	97	32	17	19
4	14	86	425	51	34	130	1090	90	86	212	17	17
5	14	62	693	49	28	128	907	75	74	48	19	19
6	15	49	520	48	28	252	772	82	103	36	17	537
⋮	⋮	⋮	⋮	⋮	⋮	⋮	⋮	⋮	⋮	⋮	⋮	⋮
26	21	59	119	31	121	110	83	107	26	19	34	81
27	20	50	106	29	104	201	77	83	54	19	28	52
28	20	129	217	32	101	171	84	77	62	20	26	50
29	24	92	126	38	—	159	70	399	33	20	24	41
30	21	76	134	51	—	195	65	214	36	20	26	37
31	20	—	79	35	—	318	—	189	—	17	27	—
Total	1457	4100	6718	1297	3347	5591	14110	4469	1965	1210	3391	2205
Mean	47.0	137	217	41.8	120	180	470	144	65.5	39.0	109	73.5
Max	318	690	1180	62	350	409	2390	435	155	212	1930	537
Min	14	41	56	29	28	94	65	59	26	17	15	17
Cfsm	0.35	1.02	1.62	0.31	0.90	1.34	3.51	1.08	0.49	0.29	0.81	0.55
in.	0.40	1.14	1.86	0.36	0.93	1.55	3.92	1.24	0.55	0.34	0.94	0.61
Cal yr	1982	Total	47189	Mean	129	Max	2150	Min 14	Cfsm	0.96	In.	13.10
Wtr yr	1983	Total	49860	Mean	137	Max	2390	Min 14	Cfsm	1.02	In.	13.84

records are required for initial calibration of parameters in the model. Empirical program parameters, which indirectly control the simulation of the various hydrologic components, are adjusted during the calibration process until the program output closely duplicates the measured runoff. Once this procedure is complete, it is assumed that the model can be used to generate runoff from other rainfall events as well. Watershed models, such as HSPF, can be used to generate long sequences of streamflow for a given set of recorded rainfall or can be used to generate a flood hydrograph from a given rain storm. For the latter case, however, special calibration using rainfall and runoff data from severe storms is necessary.

Another watershed model, TR-20 (Computer Program for Project Hydrology) developed by the U.S. Department of Agriculture, Agricultural Research Service, also simulates runoff but uses the curve number method described earlier (5).

Other computer programs for general use are available. The U.S. Army Corps of Engineers Hydrologic Engineering Center (HEC) in Davis, California, has developed several hydrologic programs that have received wide usage. One of these, HEC-1 (Flood Hydrograph Package), has several subroutines that are used to determine an optimal unit hydrograph, loss rates, or streamflow routing parameters by matching recorded and simulated hydrograph values. Other subroutines are used to perform computations for snowmelt, unit hydrograph usage, hydrograph routing and combining, hydrograph balancing, as well as rainfall sequencing and the generation and routing of floods produced by a hypothetical dam failure (5).

Once a basic understanding has been developed, the wide range of computer programs available for hydrologic analysis can and should provide important tools for design and analysis. As the development of microcomputers continues, these programs become easier and cheaper to use and provide an indispensable means for hydrologic analysis.

PROBLEMS

2-1 A reservoir used for both flood control and irrigation contains the following storage volumes: flood control, 210,000 acre-ft; and conservation, 323,100 acre-ft. What are equivalent volumes expressed in ft^3 and m^3?

2-2 In surface-water hydrology, it is convenient to use units that are often unfamiliar. Convert 1310 cfs to the equivalent flow in cms and acre-ft/day.

2-3 A total of 13 in. of precipitation falls on a 35-mi^2 drainage area during a 24-hr period.

a. Calculate the total volume of rainfall in mi^2-inches, acre-ft, ft^3, and m^3.

b. If 70% of rainfall runs off, calculate the total volume of runoff in acre-feet, and the average rate of runoff if runoff lasts for 72 hr.

2-4 A dam has been constructed with a total storage volume of 86,000 acre-ft. If the average annual flow from the drainage area upstream from the dam is 40 cfs, how many years of average runoff will the reservoir hold?

2-5 In the reservoir of Prob. 2-4, if the average annual loss due to seepage and evaporation is 40 in., and the average annual precipitation on the reservoir is 21 in., how many years (on the average) would it take to fill the reservoir? Assume there are no withdrawals, and the average surface area of the reservoir is 1000 acres.

2-6 Calculate the apparent average annual precipitation for Glenwood Springs, Colorado, for the two years given in Fig. 2-2, page 23. Calculate the average monthly precipitation. What is the maximum deviation from the average monthly precipitation?

2-7 A flood control levee is to be built in an urban area to withstand any discharge up to the 75-yr flood magnitude. The planned useful project life of the levee is 60 yr. What is the probability that the levee will be overtopped exactly twice in its 60-yr project life?

2-8 A temporary flood wall has been constructed to protect several homes in a flood plain. The wall was built to withstand any discharge up to the 20-yr flood magnitude. The wall will be removed at the end of the 3-yr period after all the homes have been relocated. Determine the probability in each of a.–d. that

a. The wall will be overtopped in any year
b. The wall will not be overtopped during the relocation operation
c. The wall will be overtopped at least once before all the homes are relocated
d. The wall will be overtopped exactly once before all the homes are relocated.
e. What return period must a highway engineer use in his design of a critical underpass drain if he is willing to accept only a 10% risk that flooding will occur in the next 5 yr?

2-9 You are to design a highway culvert for a secondary highway. Current practice dictates a 25% acceptable risk that flooding will occur at least once over its 30-yr project life. What return period must you use in its design?

2-10 The following parameters were determined for a series of annual maximum stream flows: mean of $Q = 750$ cfs, standard deviation of $Q = 110$ cfs, skew of $Q = 0.0$. Assuming that the data fit the normal distribution, what is the discharge for a flood with a recurrence interval of 100 yr?

2-11 Using Fig. 2-8, what is the apparent annual average precipitation for your hometown? What is the average annual precipitation in Phoenix, Arizona?

2-12 A record of flood flows for Touchet River at Bolles, Washington, is given in the table.

a. Using both the Pearson Type III and the normal distributions, find the magnitude of the 10-, 50-, and 100-year floods using Eq. (2-12).

b. What is the probability of a flood equal to or greater than the 20-yr flood during the next 3 yr?

Peak Annual Flood Flow Rates for Touchet River at Bolles, Washington. (Drainage area = 361 mi^2)

Water Year	Discharge (cfs)	Water Year	Discharge (cfs)
1925	2910	1964	1820
1926	2850	1965	9350
1927	3690	1966	1250
1928	4470	1967	2080
1929	879	1968	2520
1951	—	1969	7160
1952	3440	1970	3570
1953	3030	1971	7140
1954	1810	1972	6110
1955	925	1973	2750
1956	3410	1974	4740
1957	2390	1975	3540
1958	2420	1976	3980
1959	2790	1977	315
1960	1220	1978	2040
1961	2700	1979	2680
1962	2340	1980	2090
1963	2070	1981	3920

2-13 Records for a 110-mi^2 drainage area upstream from a stream gauge on the next page show the following monthly values of average precipitation and evapotranspiration. The average daily flow rate at the stream gauge is also shown.

a. Calculate the total annual volume of precipitation that falls on the drainage basin.

b. Calculate the total volume of water that flows from the basin in each month, and for the year.

c. Calculate the volume of water that does not evaporate from the basin in each month, and for the year.

d. Calculate the percentage of rainfall that flows from the drainage area each month, and for the year.

e. How do you explain that during some months evaporation exceeds rainfall?

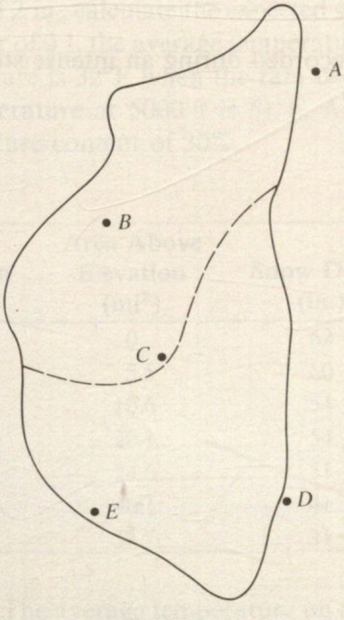
f. How do you explain that during some months total flow out of the basin exceeds rainfall?

Table for Problem 2-13

Month	Precipitation (in.)	Evapotranspiration (in.)	Streamflow (cfs)
Oct	2.2	4.0	37
Nov	4.3	2.1	50
Dec	6.5	0.6	61
Jan	6.8	0.4	11
Feb	5.9	0.5	18
Mar	5.0	1.0	80
Apr	3.1	1.6	350
May	1.4	1.9	465
Jun	0.4	2.2	201
Jul	0.2	2.8	185
Aug	0.2	2.8	147
Sep	0.6	2.0	80

2-14 Using Fig. 2-11, page 43, calculate the total volume of precipitation that would fall on a 10-mi^2 drainage area during a world-record 2-hr storm.

2-15 The accompanying figure shows a drainage area and the location of several precipitation gauges. During a given rainstorm, precipitation depth was recorded at the gauges as follows:



Gauge	Precipitation (in.)
A	3.2
B	2.8
C	4.1
D	1.6
E	2.4

PROBLEM 2-15

Calculate the average precipitation for the basin using the following methods:

a. Station average method
b. Thiessen polygon method

2-16 Using the data and figure for Prob. 2-15, calculate the average precipitation for the basin using the isohyetal method and
a. assume a linear variation between stations.
b. assume that the broken line is the 4.0-in. isohyet.

2-17 For the weather station nearest your hometown (or as specified by your instructor), record the average precipitation for each month of the year, and plot a bar graph (one bar for each month). What other records are taken at that station? The source of this information can be obtained from *Climatological Data*, a U.S. National Weather Service publication.

2-18 Determine from published data the mean annual precipitation at all weather stations or near a river basin close to your hometown (or as specified by your instructor). Choose a basin at least 400 mi^2 in area.

2-19 For the river basin of Prob. 2-18, determine the mean annual precipitation for the basin by
a. The arithmetic mean of stations in or near the basin.
b. The Thiessen polygon method. For the network, use stations in and near the basin. Show all of your work, including the Thiessen polygons and your computations.
c. The isohyetal method. Data from stations near the basin will help you in drawing the isohyets.

2-20 The following rainfall data were recorded during an intense storm:

Time	Accumulated Depth (in.)
12:15	0
12:25	0.1
12:35	0.2
12:45	0.4
12:55	0.7
13:05	1.0
13:15	1.4
13:25	2.0
13:35	2.1
13:45	2.2
13:55	2.2
14:05	2.4
14:15	2.5

Calculate the intensity of rainfall for durations varying from 10 min to 2 hr. Plot a graph using intensity as the ordinate and time as the abscissa.

2-21 Using Fig. 2-17, 2-18, and 2-19 and Table 2-5, pages 51-3, develop 100-yr, 10-yr, and 2-yr depth-duration-frequency curves for your school's location.

2-22 Using the curves developed in Prob. 2-21, tabulate accumulated 10-yr precipitation depths for durations varying from 0 to 12 hr using 1-hr increments.

2-23 Assuming a drainage area of 22 mi^2, tabulate a reasonable rainfall sequence for a 12-hr storm using the depths calculated for Prob. 2-22. Plot the incremental rainfall depth for your selected rainfall sequence versus time. Calculate two new precipitation sequences using the information in Table 2-7, page 55. Plot these sequences on the same graph.

2-24 Construct a 100-yr, 24-hr storm as it might occur in your hometown.

2-25 For the storm of Prob. 2-24, what area factor would you use to apply it over a 200-mi^2 area?

2-26 Estimate the annual water loss due to evaporation (in acre-feet) from Lake Mead (reservoir behind Hoover Dam). Lake Mead has a surface area 162,700 acres when full.

2-27 The table, below left, provides drainage area and snow-pack depth as a function of altitude for a given snow-covered basin. For an assumed rainfall of 2 in., calculate the expected snowmelt in acre-feet. Assume a degree-day factor of 0.1, the average temperature of the falling rain is 41°F, the snow temperature is 32°F when the rain begins, the lapse rate is 3°F/1000 ft, and the temperature at 5000 ft is 51°F. Also assume the original snow pack has a moisture content of 30%.

Elevation (ft)	Area Above Elevation (mi^2)	Snow Depth (in.)
10600	0	62
10000	5.5	60
9000	10.6	54
8000	20.1	54
7000	24.6	51
6000	40.1	41
5000	51.2	31

Day	Avg. Temp. (°F)	Avg. Daily Flow Rate (cfs)
Mon	33	39.2
Tues	38	302.4
Wed	49	952.0
Thur	43	739.2
Fri	39	470.4
Sat	38	336.0
Sun	33	72.8

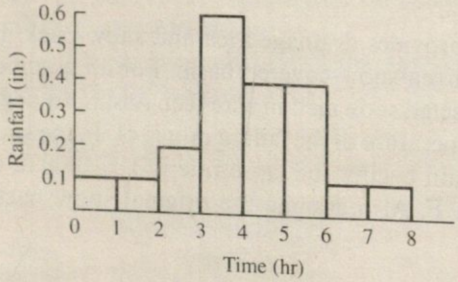
2-28 The average temperature on a snow-covered drainage area for a 1-week period is given in the table above right, along with the average daily flow rate at a stream gauge. Assuming that all streamflow was due to snowmelt, calculate the average degree-day factor for the week. The drainage area is 21 mi^2.

2-29 The average rainfall over a 120-acre watershed for a particular storm was determined to be as follows:

Hour	Hourly Rainfall (in.)
1	0.2
2	0.4
3	1.5
4	1.0
5	0.5
6	0.2
7	0

The volume of runoff from this storm was determined to be 19 acre-ft. What is the ϕ index?

2-30 The hourly rainfall depths for a storm over a 100-acre basin is shown below. If the ϕ index for the basin is assumed to be 0.20 in./hr, what will be the *volume* of surface runoff in acre-feet from the basin for this storm?



PROBLEM 2-30

Hour	Hourly Rainfall (in.)
0	0
1	0.1
2	0.3
3	0.7
4	1.9
5	3.6
6	1.1
7	0.4
8	0

2-31 During an actual storm that occurred on a 200-acre drainage area, the average hourly precipitation depths were as follows: The volume of surface runoff from this storm was measured as 100 acre-ft. Calculate the apparent ϕ index for the drainage area using the table above.

2-32 If in Prob. 2-31, the soil is a clay loam, estimate a correct value for k_f assuming a reasonable value for f_c and assuming f_0 is 1.5 in./hr.

2-33 Using Fig. 2-22, page 59, calculate a 48-hr probable maximum precipitation depth for a 36-mi² area at the southwest corner of Kansas.

2-34 Arrange the 48-hr probable maximum precipitation depth calculated in Prob. 2-33 in a reasonable sequence of 2-hr rainfall increments.

2-35 What is the probable maximum precipitation for a 12-hr storm on a 400-mi^2 drainage basin just north of Houston, Texas?

2-36 Using the soil-cover-complex (SCS) method, determine the total volume of runoff to be expected from a 50-yr, 24-hr rainfall occurring at the southwest corner of the state of Kansas. Assume the soil is a clay loam, the land has been used as pastureland, prior conditions have been wet, the land has been very heavily grazed for some time, and the drainage area is 200 mi^2.

2-37 Repeat Prob. 2-36, but assume the hydrologic conditions are
a. fair.
b. good.

2-38 Using the soil-cover-complex method, determine 2-hr increments of rainfall excess for a 100-yr, 24-hr storm on a 27-mi^2 basin near Spokane, Washington. Assume the soil is claylike when wet. The cover is as follows: 70% wheat, 25% pasture, 5% roads. Assume the basin is in poor hydrologic condition, and considerable rain precedes the 100-yr storm. The terrain in which this basin is located is gentle sloping hills.

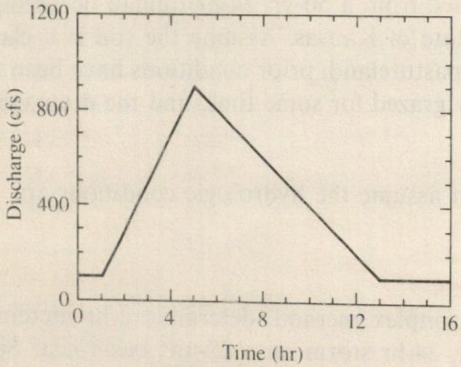
2-39 The following meteorologic and hydrologic information has been collected for a waste-isolation pond.

Month	Surface Inflow (cfs-days)	Average Air Temp. (°F)	Precipitation (in.)	Pan Evaporation (in.)
Mar	190	42	3.1	2.9
Apr	84	49	2.1	4.6
May	82	57	3.6	4.7
Jun	7	72	2.8	5.1
Jul	21	75	1.9	5.2
Aug	9	70	1.3	5.1
Sep	29	66	0.6	4.1
Oct	17	61	0.9	3.0

The pond has an approximately constant surface area of 620 acres and a change in storage capacity of 430 acre-ft/ft of change in depth. Calculate a net change in the volume of water stored in the pond during the 8-month period.

2-40 Calculate the peak rate of runoff from a 20-acre concrete hard-stand area at an airport if the time of concentration for the area is 15 min, and the design rainfall intensity is given as $i = (3.6)/(3 + t)$, where t is the duration of rainfall in minutes.

2-41 Given the hypothetical flood hydrograph for a basin having an area of 10 mi^2, what is the peak discharge for a unit hydrograph obtained from this flood?



PROBLEM 2-41

2-42 Calculate the runoff hydrograph for the 50-yr, 24-hr storm developed in Prob. 2-36. Assume the length of the stream in the drainage is 18 mi, and the difference in elevation between the upper and lower elevations is 180 ft. Use a Snyder unit hydrograph and assume topographic conditions are similar to those of the Appalachian Mountains.

2-43 The following streamflow hydrograph was recorded for a stream draining a drainage area of 50 mi^2 for a 24-hr storm. Develop a unit hydrograph for this basin.

Time (hr)	Q (cfs)	Time (hr)	Q (cfs)	Time (hr)	Q (cfs)
8	54	36	254	64	96
10	54	38	251	66	90
12	53	40	240	68	86
14	53	42	230	70	82
16	52	44	218	72	79
18	58	46	200	74	76
20	66	48	190	76	74
22	76	50	177	78	72
24	88	52	157	80	71
26	108	54	142	82	70
28	138	56	130	84	69
30	178	58	120	86	69
32	208	60	111	88	68
34	233	62	103	90	68

2-44 Use the unit hydrograph developed in Prob. 2-43 to develop a streamflow hydrograph for the following 72-hr rainfall excess.

Hours	August Rainfall (in.)
0–24	1.1
24–48	1.9
48–72	0.7

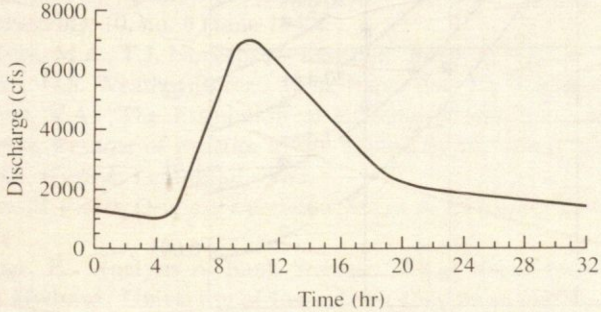
2-45 The data below are the stream discharges from a 4-hr storm on the basin of Prob. 2-36. Plot the flood hydrograph. Determine the direct runoff (in inches) for this storm, and determine and plot the 4-hr unit hydrograph for the basin.

Date	Hour	Q (cfs)	Date	Hour	Q (cfs)
Feb. 16	0200	90		2200	300
	0400	80		2400	245
	0600	75	Feb 17	0200	210
	0800	450		0400	175
	1000	700		0600	145
	1200	750		0800	125
	1400	800		1000	100
	1600	600		1200	85
	1800	450		1400	75
	2000	350		1600	70

2-46 Determine a synthetic unit hydrograph for the basin of Prob. 2-38. For this basin, the stream length is 11 mi and L_{CA} is 5.5 mi.

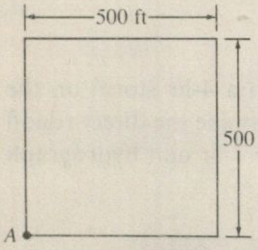
2-47 The hydrograph shown resulted from a 2-hr rainstorm.

a. Using reasonable assumptions, determine the volume of surface runoff and the volume of baseflow. What was the depth of runoff if the drainage area is 8000 acres?

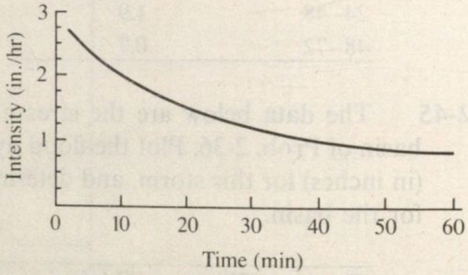


b. Develop and plot a unit hydrograph for the drainage basin.

2-48 The parking lot shown is to have a drainage system designed to convey peak runoff from a 5-yr storm. If the inlet is at A, determine the peak rate of inflow to the pipe. Assume that water flows over the lot at 0.5 ft/s and that the parking lot is paved.

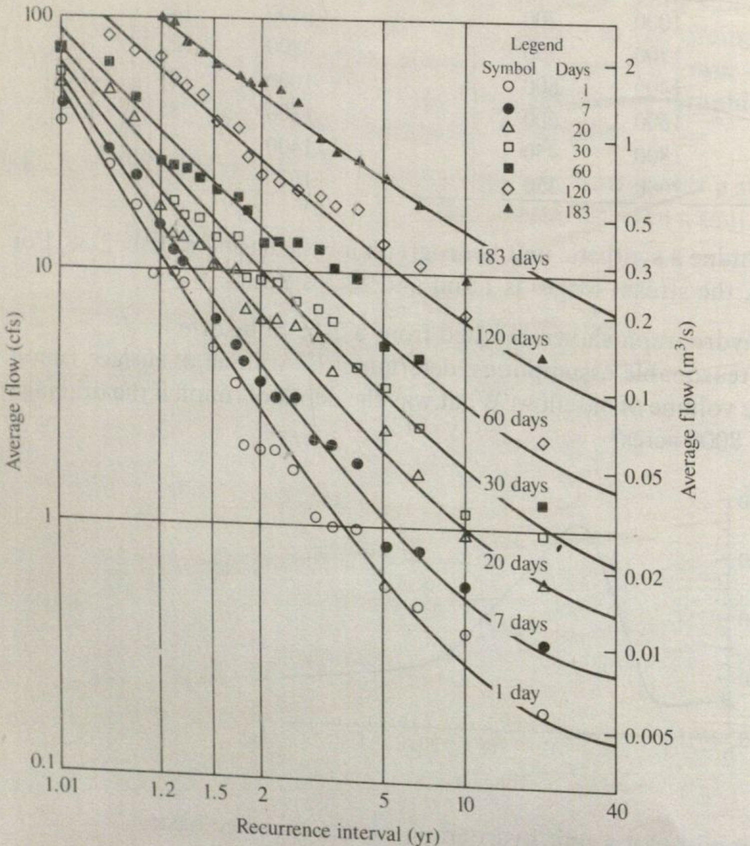


(*a*) Drainage area



(*b*) Intensity-duration curve

2-49 Low-flow-duration curves for Yellow Creek near Hammondsville, Ohio, are given in the accompanying figure.



a. Determine the necessary size of a reservoir (in acre-feet) to provide a flow of not less than 25 cfs during a 6-yr drought.
b. How often on the average might one expect the flow over the driest 30-day period to be equal to or less than 80 acre-ft?

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