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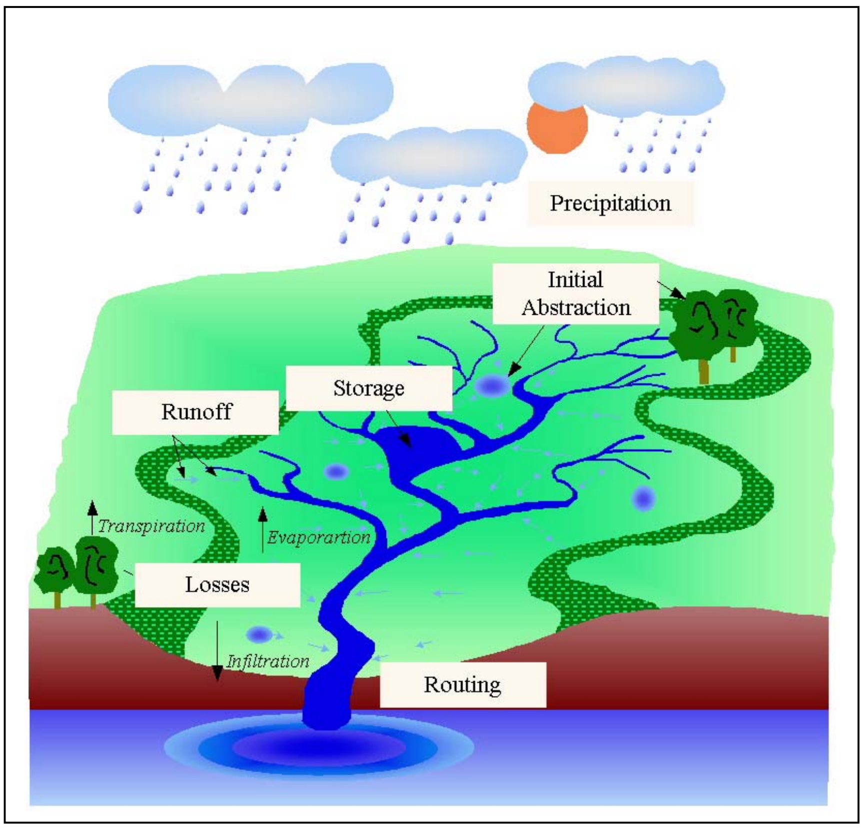

National Highway Institute

CHAPTER 5

PEAK FLOW FOR UNGAGED SITES

While using frequency approaches is almost always the most appropriate means to determine a peak flow, at many stream crossings of interest to the highway engineer, there may be insufficient stream gaging records, or often no records at all, available for making a flood frequency analysis, such as a log-Pearson Type III analysis. Several regional analysis and empirical techniques have been developed and successfully applied to address these situations.

Extrapolation of data from nearby watersheds with comparable hydrologic and physiographic features is referred to as regional analysis and includes regional regression equations and index-flood methods. The USGS has collected a comprehensive series of these regional regression equations into the National Flood Frequency computer program. This tool provides the means for computing a peak discharge for any place in the United States.

Empirical methods include such widely applied techniques as the rational formula and the NRCS (formerly the SCS) graphical method. These methods employ empirical relationships between rainfall and runoff that allow estimation of design discharges on ungaged watersheds by development of parameters describing the watershed. If an engineer has an interest in the magnitude of measured maximum flood flows, peak discharge envelope curves can be used alone or in conjunction with other regional or empirical analyses.

Watershed area plays an important role for each of these ungaged watershed peak flow determination methods. As described in Chapter 2, watershed area is the single most important characteristic for determining runoff peaks. As will be seen, the area of the watershed also provides a basis for determining the limits of applicability for many of these methods.

5.1 REGIONAL REGRESSION EQUATIONS

Regional regression equations are commonly used for estimating peak flows at ungaged sites or sites with insufficient data. Regional regression equations relate either the peak flow or some other flood characteristic at a specified return period to the physiographic, hydrologic, and meteorologic characteristics of the watershed.

5.1.1 Analysis Procedure

The typical multiple regression model utilized in regional flood studies uses the power model structure:

$$Y_T = aX_1^{b_1} X_2^{b_2} \cdots X_p^{b_p} \qquad (5.1)$$

where,

Y_t = the dependent variable
$X_1, X_2, ..., X_p$ = independent variables
a = the intercept coefficient
$b_1, b_2, ..., b_p$ = regression coefficients.

The dependent variable is usually the peak flow for a given return period T or some other property of the particular flood frequency, and the independent variables are selected to characterize the watershed and its meteorologic conditions. The parameters a, $b_1, b_2, ..., b_p$ are determined using a regression analysis. Regression analysis is described in detail by Sanders

(1980), Riggs (1968), and McCuen (1993). The general procedure for making a regional regression analysis is as follows:

1. Obtain the annual maximum flood series for each of the gaged sites in the region.

2. Perform a separate flood frequency analysis (e.g., log-Pearson Type III) on each of the flood series of Step 1 and determine the peak discharges for selected return periods (e.g., the 2-, 5-, 10-, 25-, 50-, 100-, and 500-year discharges are commonly selected).

3. Determine the values of watershed and meteorological characteristics for each watershed for which a flood series was collected in Step 1.

4. Form an (n by p) data matrix of all the data collected in Step 3, where n is the number of watersheds of step 1 and p is the number of watershed characteristics obtained for Step 3.

5. Form a one-dimensional vector with n peak discharges for the specific return period selected.

6. Regress the vector of n peak discharges of Step 5 on the data matrix of Step 4 to obtain the prediction equation.

If more than one return period is of interest, the procedure can be repeated for each return period, with a separate equation developed for each return period. In this case, it is also important to review closely the regression coefficients to ensure that they are rational and consistent across the various return periods. Because of sampling variation, it is possible for the regression analyses to produce a set of coefficients that, under certain sets of values for the predictor variables, result in the computed 10-year discharge, for example, being greater than the computed 25-year discharge. In such cases, the irrational predictions can be eliminated by smoothing the coefficients. If the coefficients need to be smoothed, the goodness-of-fit statistics should be recomputed using the smoothed coefficients. The problem can usually be prevented by using the same predictor variables for all of the equations.

The most important watershed characteristic is usually the drainage area and almost all regression formulas include drainage area above the point of interest as an independent variable. The choice of the other watershed characteristics is much more varied and can include measurements of channel slope, length, and geometry, shape factors, watershed perimeter, aspect, elevation, basin fall, land use, and others. Meteorological characteristics that are often considered as independent variables include various rainfall parameters, snowmelt, evaporation, temperature, and wind.

As many independent variables as desired can be used in a regression analysis although it would be unlikely that more than one measure of any particular characteristic would be included. The statistical significance of each independent variable can be determined and those that are statistically insignificant at a specified level of significance (e.g., 5 percent) can be eliminated. In addition to statistical criteria, it is also important for all coefficients to be reasonable.

The specific predictor variables to be included in a regression equation are usually selected using a stepwise regression analysis (McCuen, 1989). While a 5 percent level of significance is sometimes used to make the decision, it is better to select only those variables that are easily obtained and necessary to provide both a reasonable level of accuracy and rational coefficients. When stepwise regression analysis is used to select variables for a set of equations for different return periods, the same independent variables should be used in all of the equations. In a few cases, this may cause some equations in the set to have less accuracy than would be possible, but it is usually necessary to ensure consistency across the set of equations.

5.1.2 USGS Regression Equations

In a series of studies by the USGS, the Federal Highway Administration, and State Highway Departments, statewide regression equations have now been developed throughout the United States. The highway community has made a significant contribution to acquiring additional stream flow data through funding USGS stream gaging station studies throughout the country since the 1960s. Highway interests have supported these research endeavors with expenditures of $14 million. These equations permit peak flows to be estimated for return periods varying from 2 to 500 years. The published equations (Jennings, et al., 1994) are included in the National Flood Frequency (NFF) Program discussed in Section 5.1.5.2.

Typically, each state is divided into regions of similar hydrologic, meteorologic, and physiographic characteristics as determined by various hydrological and statistical measures. Using a combination of measured data and rainfall-runoff simulation models such as that of Dawdy, et al. (1972), long-term records of peak annual flow were synthesized for each of several watersheds in a defined region. Each record was subjected to a log-Pearson Type III frequency analysis, adjusted as required for loss of variance due to modeling, and the peak flow for various frequencies determined.

Multiple regression was then used on the logarithmically transformed values of the variables to obtain regression equations of the form of Equation 5.1 for peak flows of selected frequencies. Only those independent variables that were statistically significant at a predetermined level of significance were retained in the final equations.

5.1.2.1 Hydrologic Flood Regions

In most statewide flood-frequency reports, the analysts divided the state into separate hydrologic regions. Regions of homogeneous flood characteristics were generally determined by using major watershed boundaries and an analysis of the areal distribution of the regression residuals, which are the differences between regression and station (observed) T-year estimates. In some instances, the hydrologic regions were also defined by the mean elevation of the watershed or by statistical tests such as the Wilcoxon signed-rank test.

Regression equations are defined for 210 hydrologic regions throughout the Nation, indicating that, on average, there are about four regions per state. Figure 5.1 gives the NFF statewide results for Maine and is used to illustrate the content for one of the 210 regions. Some areas of the Nation, however, have inadequate data to define flood-frequency regions. For example, there are regions of undefined flood frequency in Florida, Texas, and Nevada. For the state of Hawaii, regression equations are only provided for the island of Oahu.

Summary

Maine is considered to be a single hydrologic region. The regression equations developed for the state are for estimating peak discharges (QT) having recurrence intervals T that range from 2 to 100 years. The explanatory basin variables used in the equations are drainage area (A), in square miles; channel slope (S), in feet per mile; and storage (St), which is the area of lakes and ponds in the basin in percentage of total area. The constant 1 is added to St in the computer application of the regression equations. The user should enter the actual value of St. All variables can be measured from topographic maps. The regression equations were developed from peak-discharge records through 1974 for 60 sites with records of at least 10 years in length. The regression equations apply to streams having drainage areas greater than 1 square mile and virtually natural flood flows. Standard errors of estimate of the regression equations range from 31 to 49 percent.

Procedure

Topographic maps and the following equations are used to estimate the needed peak discharges QT, in cubic feet per second, having selected recurrence intervals T.

$$Q2 = 14.0A^{0.962}S^{0.268}ST^{-0.212}$$

$$Q5 = 21.2A^{0.946}S^{0.298}ST^{-0.239}$$

$$Q10 = 26.9A^{0.936}S^{0.315}ST^{-0.252}$$

$$Q25 = 35.6A^{0.923}S^{0.333}ST^{-0.266}$$

$$Q50 = 42.7A^{0.915}S^{0.346}ST^{-0.275}$$

$$Q100 = 50.9A^{0.907}S^{0.358}ST^{-0.282}$$

Reference

Morrill, R.A., 1975. “A Technique for Estimating the Magnitude and Frequency of Floods in Maine.” U.S. Geological Survey Open-File Report No. 75-292.

Figure 5.1. Description of NFF regression equations for rural watersheds in Maine (Jennings, et al., 1994).

Example 5.1. To illustrate the use of regional regression equations for estimating peak flows, consider the following example.

It is desired to renovate a bridge at a highway crossing of the Seco Creek at D'Hanis, TX. The site is ungaged and the design return period is 25 years. The site lies in Region 5 as defined by Schroeder and Massey (1970). The equations have the following form:

$$Q_T = a\,A^{b_1}\,S^{\,b_2} \qquad (5.2)$$

where,

- Q_T = peak annual flow for the specified return periods, m^3/s (ft^3/s)
- A = drainage area contributing surface runoff above the site, km^2 (mi^2)
- S = average slope of the streambed between points 10 and 85 percent of the distance along the main stream channel from the site to the watershed divide, m/km (ft/mi).

The coefficients of Equation 5.2 are given in Table 5.1. The range of application of the above equations was specified as:

Variable	Value in SI	Value in CU
Drainage Area (A)	$2.80 < A\ (km^2) < 5{,}040$	$1.08 < A\ (mi^2) < 1{,}950$
Slope (S)	$1.7 < S\ (m/km) < 14.5$	$9.2 < S\ (ft/mi) < 76.8$

By measuring the drainage area above the site from a topographic map, the area A is found to be 545.5 km^2 (210.6 mi^2) and the channel slope between the 10 and 85 percent points is 2.833 m/km (14.96 ft/mi). Using Equation 5.2 and the coefficients of Table 5.1, the 25-year peak flow is:

Variable	Value in SI	Value in CU
$Q_{25} = a_{25}\ A^{0.776}\ S^{0.554}$	$= 6.13(545.5)^{0.776}(2.833)^{0.554}$ $= 1450\ m^3/s$	$= 180(210.6)^{0.776}\ (14.96)^{0.554}$ $= 51{,}200\ ft^3/s$

Table 5.1. Regression Coefficients for Texas, Region 5

Return Period, T (years)	Regression Coefficients				Standard Error (%)*
	a (SI)	a (CU)	b_1	b_2	
2	0.319	4.82	0.799	0.966	62.1
5	1.60	36.4	0.776	0.706	46.6
10	3.15	82.6	0.776	0.622	42.6
25	6.13	180	0.776	0.554	41.3
50	8.96	278	0.778	0.522	42.0
100	12.3	399	0.782	0.497	44.1

* Standard errors were computed using the logarithmic regression and are given as a percentage of the mean.

5.1.2.2 Assessing Prediction Accuracy

In most cases, regional regression equations are given with associated standard errors, which are indicators of how accurately the regression equation predicts the observed data used in their development. The standard error of estimate is a measure of the deviation of the observed data from the corresponding predicted values and is given by the basic equation:

$$S_e = \left[\frac{\Sigma(\hat{Q}_i - Q_i)^2}{n - q} \right]^{0.5} \qquad (5.3)$$

where,

Q_i = observed value of the dependent variable (discharge)

$\hat{Q}_i$ = corresponding value predicted by the regression equation

n = number of watersheds used in developing the regression equation

q = number of regression coefficients (i.e., a, b_1, ..., b_p).

In a manner analogous to the variance, the standard error can be expressed as a percentage by dividing the standard error S_e by the mean value ($\overline{Q}_T$) of the dependent variable:

$$V_e = \frac{S_e}{\overline{\overline{Q}}_T} \times 100\% \qquad (5.4)$$

where,

V_e = coefficient of error variation.

V_e of Equation 5.4 has the form of the coefficient of variation of Equation 4.14. The standard error of regression S_e has a very similar meaning to that of the standard deviation, Equation 4.13, for a normal distribution in that approximately 68 percent of the observed data should be contained within ±1 standard error of the regression line.

When S_e is computed for regional regression equations, it is usually computed using the logarithms of the flows. Thus, $\hat{Q}_i$ and Q_i of Equation 5.3 are logarithms of the corresponding flows. This is believed to be necessary because the errors (i.e., $\hat{Q}_i - Q_i$) have a constant variance when expressed from the logarithms.

5.1.2.3 Comparison with Gaged Estimates

Because of the extensive use now being made of USGS regression equations, it is of interest to compare peak discharges estimated from these equations with results obtained from a formal flood frequency analysis as described in Chapter 4. A direct comparison cannot be made with the previously used Medina River data because of storage and regulation upstream of the gage.

Since regression equations apply only to totally unregulated flow, Station 08179000, Medina River near Pipe Creek, Texas, has been selected for comparison. This gage has 43 years of record, drains an area of 1,228 km^2 (474 mi^2), is totally unregulated, and has station and generalized skews of -0.005 and -0.234, respectively. The data were analyzed with a log-Pearson Type III distribution, and the 10-, 25-, 50- and 100-year peak discharges estimated using the USGS Bulletin 17B (1982) weighted skew option (G_L = -0.2). These values together with peak flows determined from a frequency curve through the systematic record are summarized in Table 5.2.

The Pipe Creek gage is located in Region 5 in Texas and the regression equations given for the Seco Creek example above are applicable. The watershed has an average slope of 3.07 m/km (16.2 ft/mi) between 10 and 85 percent points along the main stream channel. The corresponding peak flows calculated from the appropriate regression equations are also summarized in Table 5.2.

The peak discharges estimated from the regression equations are all substantially higher than the comparable values determined from the log-Pearson Type III analysis, although all are within the USGS Bulletin 17B, upper 95-percent confidence limits. Further review of the data at this station indicates that a frequency curve constructed using the systematic record plots above the log-Pearson Type III distribution curves at least over the range of frequencies considered in the above comparison. This is partially a result of a peak flow in 1978 in excess of 7960 m^3/s (281,000 ft^3/s), which, according to the log-Pearson Type III analysis, is an event approaching the 500-year peak flow.

It has been suggested by some experienced hydrologists that regression equations may give better estimates of peak flows of various frequencies than formal statistical frequency analyses. They reason that regression equations more nearly reflect the potential or capacity of the watershed to experience a peak flow of given magnitude, whereas a frequency analysis is biased by what has been recorded at the gage. Some justification exists for this argument as there are many examples throughout the country of adjacent watersheds of comparable size and physiographic and hydrologic characteristics experiencing the same storm patterns, but wherein only one has recorded major floods. This is obviously a function of where the storm occurs, but frequency analyses of gaged data from the different watersheds may give very different peak flows for the same frequencies. On the other hand, regression equations will give comparable flood magnitudes at the same frequencies for each watershed, all other factors being approximately equal.

This is not to suggest that regional regression equations should take precedence over frequency analysis, especially when sufficient data are available. Regression equations, however, do serve as a basis for comparison of statistically determined peak flows of specified frequencies and provide for further evaluation of the results of a frequency analysis. They may be used to add credence to historical flood data or may indicate that historical records should be sought out and incorporated into the analysis. Regression equations can also provide insight into the treatment of outliers beyond the purely statistical methods discussed in Section 4.3.6.1. As demonstrated by the above discussion, comparison of the peak flows obtained by different methods may indicate the need to review data from other comparable watersheds within a region and the desirability of transposing or extending a given record using data from other gages.

Sauer (1973) has proposed a methodology for weighting the log-Pearson Type III result with the regression equation estimate for the gaged watershed based on the gage record length and the equivalent record length for the regression equation as follows:

$$Q_{gw} = \frac{Q_g N_g + Q_r N_r}{N_g + N_r} \tag{5.5}$$

where,

Q_{gw} = weighted peak flow estimate at the gage
Q_g = log-Pearson Type III peak flow estimate at the gage
Q_r = regression equation peak flow estimate at the gage
N_g = number of years of record at the gage
N_r = equivalent record length of the regression equation.

This methodology seeks to use information in the gage record as well as similar gaged watersheds in the region via the regression equations. It is presented in many of the USGS reports documenting development of the regression equations.

Table 5.2. Comparison of Peak Flows from Log-Pearson Type III Distribution and USGS Regional Regression Equation

	Peak Discharge (m^3/s)			Peak Discharge (ft^3/s)		
Return Period (years)	Log-Pearson Type III Frequency	Systematic Record	USGS Regression Equations	Log-Pearson Type III Frequency	Systematic Record	USGS Regression Equations
10	1,210	1,420	1,580	42,700	50,300	55,700
25	1,950	2,520	2,850	68,900	89,000	100,000
50	2,630	3,640	4,070	92,900	129,000	144,000
100	3,420	5,080	5,590	120,900	179,000	197,000

5.1.2.4 Application and Limitations

Several points should be kept in mind when using regional regression equations. For the most part, the state regional equations are developed for unregulated, natural, nonurbanized watersheds. They separate out mixed populations (i.e., rain produced floods from snowmelt floods or hurricane associated storms). The equations are regionalized so that it is incumbent on the user to carefully define the hydrologic region and to define the dependent and independent variables in the exact manner prescribed for each set of regional equations. This includes applying the equations to basins that fall within the range of characteristics for basins used to develop the equations. The designer is also cautioned to apply these equations within or close to the range of independent variables utilized in the development of the equations.

Although not a serious problem, the designer should be alert to any discrepancies in results from regression equations when applied at regional boundaries and especially near state boundaries. Within-state regional boundaries generally define hydrologic regions with similar characteristics, and regression equations may not give comparable results near regional boundaries.

Hydrologic regions also may cross state boundaries, and regression equations for adjacent regions in different states can give substantially different peak flows for the same frequency. When working near within-state regional and state boundaries, regression equations for adjacent regions should be checked and any serious discrepancies reconciled.

The following additional limitations should be observed:

- Rural equations should only be used for rural areas and should not be used in urban areas unless the effects of urbanization are insignificant.

- Regression equations should not be used where dams, flood-detention structures, and other human-made works have a significant effect on peak discharges.

- The magnitude of the standard errors can be larger than the reported errors if the equations are used to estimate flood magnitudes for streams with variables outside the ranges for the necessary input variables as stated in the applicable report.

- Drainage area should always be determined. Although a hydrologic region might not include drainage area as a variable in the prediction equation to compute a frequency curve, the drainage area may be used for determining the maximum flood envelope discharge from Crippen and Bue (1977) and Crippen (1982), as well as weighting of curves for watersheds in more than one region.

- Frequency curves for watersheds contained in more than one region cannot be computed if the regions involved do not have corresponding T-year equations. Failure to observe this limitation will lead to erroneous results. Frequency curves are weighted by the percentage of drainage area in each region. No provision is provided for weighting frequency curves for watersheds in two different states.

- In some instances, the maximum flood envelope value might be less than some T-year computed peak discharges for a given watershed. The T-year peak discharge is the discharge that will be exceeded as an annual maximum peak discharge, on average, once every T years. The engineer should carefully determine which maximum flood-region contains the watershed being analyzed and is encouraged to consult Crippen and Bue (1977) and Crippen (1982) for guidance and interpretations.

- The engineer should be cautioned that some hydrologic regions do not have prediction equations for peak discharges as large as the 100-year peak discharge. The engineer is responsible for the assessment and interpretation of any interpolated or any extrapolated T-year peak discharge. Examination of plots of the frequency curves is highly desirable.

Maximum flood envelopes are discussed later in this chapter.

5.1.3 USGS Urban Watershed Studies

In 1978, the Federal Highway Administration contracted with the USGS to conduct a nationwide survey of flood frequencies under urban conditions. The purposes of the study were to: review the literature of urban flood studies, compile a nationwide data base of flood frequency characteristics including land use variables for urban watersheds, and define estimating techniques for ungaged urban areas. Results of the study are described in detail in USGS Water Supply Paper 2207 (Sauer, et al., 1983).

A review of nearly 600 urbanized sites resulted in a final list of 269 sites that met criteria wherein at least 15 percent of the drainage area was covered with commercial, industrial, or residential development; reliable flood data were available for 10 or more years (either actual peak flow data or synthesized data from a calibrated rainfall-runoff model); and the period of record was coincident with a period of relatively constant urbanization. The complete data base, including topographic and climatic variables, land use variables, urbanization indices, and flood frequency estimates are available from the USGS National Center, Reston, VA.

The USGS study developed a procedure for quantifying the effects of urbanization on peak discharge and flood volume. Regression equations relate the peak discharge at a specified frequency to: (l) drainage area, (2) peak discharge for the same watershed in a rural condition, and (3) a basin development factor (BDF). The basin development factor is a measure of the degree of urbanization that exists (or might exist in the future) in the watershed. The BDF is discussed in more detail in Section 5.1.4.

The USGS regression equations can be used to estimate the peak discharge for existing conditions of urbanization, and they can also be used to estimate the peak discharge for future conditions. The urban peak flow equations are applicable to a wide variety of geographic and climatologic conditions. They can provide useful estimates of the relative impact that varying amounts of urbanization have on peak discharge. However, these estimates cannot be treated as absolutes, and some judgment must be exercised in their application.

5.1.3.1 Peak Discharge Equations

Initially, the USGS study developed regression equations for urban peak flow discharge in terms of seven independent variables. Subsequently, it was found that by eliminating the less significant independent variables from the regression analyses, simpler equations could be obtained without appreciably increasing the standard error of regression. Ultimately, the following family of three-parameter equations was developed by the USGS for peak discharges in urbanized watersheds:

$$UQ_T = a_T \; A^{C_{1T}} (13 - BDF)^{C_{2T}} \; RQ_T^{C_{3T}} \tag{5.6}$$

where,

UQ_T = peak discharge of recurrence interval, T, for an urbanized condition, m^3/s (ft^3/s)
T = recurrence interval ranging from 2 to 500 years
A = drainage area of the basin, km^2 (mi^2)
BDF = basin development factor as defined below
RQ_T = peak discharge of recurrence interval, T, for rural conditions, m^3/s (ft^3/s).
A_T, C_{1T}, C_{2T}, and C_{3T} = regression constants summarized in Table 5.3.

This equation is applicable for watersheds between 0.5 and 260 km^2 (0.2 and 100 mi^2).

Table 5.3. Unit Conversion Constants for the USGS Urban Equations

Return Period	a_T (SI)	a_T (CU)	C_{1T}	C_{2T}	C_{3T}
2	4.13	13.2	0.21	-0.43	0.73
5	4.12	10.6	0.17	-0.39	0.78
10	3.86	9.51	0.16	-0.36	0.79
25	3.69	8.68	0.15	-0.34	0.80
50	3.54	8.04	0.15	-0.32	0.81
100	3.52	7.70	0.15	-0.32	0.82
500	3.38	7.47	0.16	-0.30	0.82

5.1.3.2 Basin Development Factor

Several indices of urbanization were evaluated in the course of the USGS study including percentage of basin occupied by impervious surfaces, population and population density, basin response time, and basin development factor. The BDF, which provides a measure of the efficiency of the drainage system within an urbanizing watershed, was selected for a number of reasons. The BDF was highly significant in the regression equations, compared to the other measures of urbanization, and its value may be determined from topographic maps, storm drain maps, and field surveys.

To determine the BDF, the basin is first divided into three sections as shown in Figure 5.2. Each section contains approximately one-third of the drainage area of the watershed. Travel time is given consideration when drawing these boundaries so that the travel distances along two or more streams within a particular third are about equal. This does not mean that the travel distances of all three subareas are equal, only that within a particular subarea the travel distances are approximately equal.

Within each section of the basin, four aspects of the drainage system are evaluated and assigned a code:

1. **Channel modifications**. If channel modifications such as straightening, enlarging, deepening, and clearing are prevalent for the main drainage channel and principal tributaries (those that drain directly into the main channel), a code of 1 is assigned. Any one, or all, of these modifications would qualify for a code of 1. To be considered significant, at least 50 percent of the main drainage channels and principal tributaries must be modified to some extent over natural conditions. If channel modifications are not prevalent, a code of 0 is assigned.

2. **Channel linings**. If more than 50 percent of the main drainage channel and principal tributaries have been lined with an impervious material, such as concrete, a code of 1 is assigned. If less than 50 percent of these channels are lined, a code of 0 is assigned. The presence of channel linings would probably indicate the presence of channel improvements as well. Therefore, this is an added factor and indicates a more highly developed drainage system.

3. **Storm drains or storm sewers**. Storm drains are defined as enclosed drainage structures (usually pipes), frequently used on the secondary tributaries where the drainage is received directly from streets or parking lots. Quite often these drains empty into the main tributaries and channels that are either open channels or in some basins may be enclosed as box or pipe culverts. When more than 50 percent of the secondary tributaries within a section consist of storm drains, a code of 1 is assigned. If less than 50 percent of the secondary tributaries consist of storm drains, a code of 0 is assigned. It should be noted that if 50 percent or more of the main drainage channels and principal tributaries are enclosed, the aspects of channel improvements and channel linings would also be assigned a code of 1.

4. **Urbanization/Curb and gutter streets**. If more than 50 percent of a subarea is urbanized (covered by residential, commercial, and/or industrial development), and if more than 50 percent of the streets and highways in the subarea is constructed with curbs and gutters, a code of 1 should be assigned. Otherwise, a code of 0 is assigned. Frequently, drainage from curb and gutter streets will empty into storm drains.

The above guidelines for determining the various drainage system codes are not intended to be precise measurements. Practical determination involves a certain amount of subjectivity and engineering judgment. It is recommended that field checking be performed to obtain the best estimate. The BDF is computed as the sum of the assigned codes. With three subareas per basin, and four drainage aspects to which codes are assigned in each subarea, the maximum

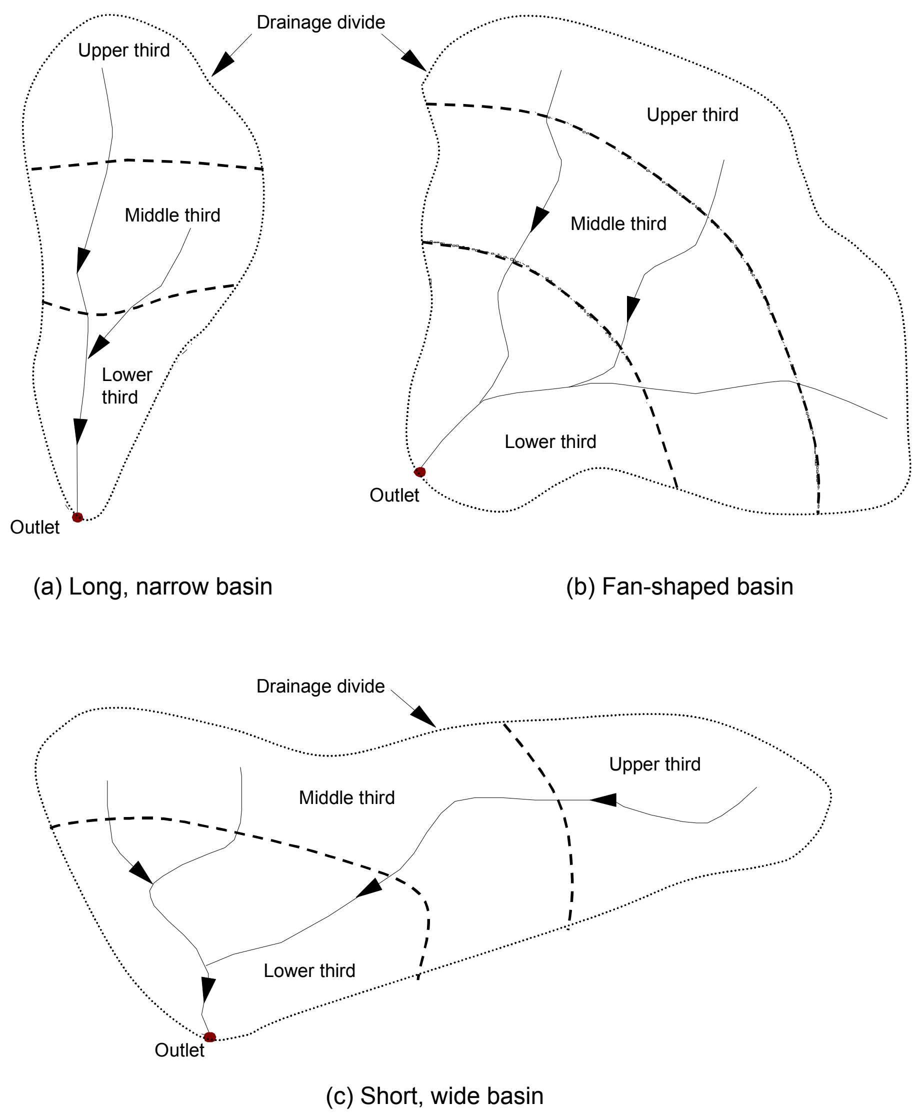

Figure 5.2. Subdivision of watersheds for determination of BDF

value for a fully developed drainage system would be 12. Conversely, if the drainage system has not been developed, a BDF of 0 would result. Such a condition does not necessarily mean that the basin is unaffected by urbanization. In fact, a basin could be partially urbanized, have some impervious area, and have some improvements to secondary tributaries, and still have an assigned BDF of 0. It will be shown later that such a condition will still frequently cause increases in peak discharges.

The BDF is a fairly easy index to estimate for an existing urban basin. The 50 percent guideline is usually not difficult to evaluate because many urban areas tend to use the same design criteria throughout, and therefore the drainage aspects are similar throughout. Also, the BDF is convenient to use for projecting future development. Full development and maximum urban effects on peaks would occur when BDF = 12. Projections of full development, or intermediate stages of development, can usually be obtained from city development plans.

<u>Example 5.2 (SI)</u>. Information is first collected from topographic maps and a field survey for the 99.3-ha watershed. The watershed is divided into three subareas of approximately equal area. The separation is based on homogeneity of hydrologic conditions, with the following values measured:

Subarea	**Area (ha)**	**Main Channel Length (m)**	**Length of Secondary Tributaries (m)**	**Road Length (m)**	**Length of Channel Modified (m)**	**Length of Channel Lined (m)**	**Storm Drains (m)**	**Curb and Gutter (m)**
Upper	29.2	780	1,580	870	140	0	410	210
Middle	36.3	1,140	1,200	1,430	615	540	680	920
Lower	33.8	910	660	1,710	525	480	460	970
Sum	99.3	2,830						

The BDF is determined as follows:

Channel Modifications
Upper Third: 140 m have been straightened and deepened — Code = 0
[140/780 < 50%]
Middle Third: 615 m have been straightened and deepened — = 1
[615/1140 > 50%]
Lower Third: 525 m have been straightened and widened — = 1
[525/910 > 50%]

Channel Linings
Upper Third: 0 m of channel are lined — Code = 0
[0/780 < 50%]
Middle Third: 540 m of channel are lined — = 0
[540/1140 < 50%]
Lower Third: 480 m of channel are lined — = 1
[480/910 > 50%]

Storm Drains on Secondary Tributaries
Upper Third: 410 m have been converted to drains — Code = 0
[410/1580 < 50%]
Middle Third: 680 m have been converted to drains — = 1
[680/1200 > 50%]
Lower Third: 460 m have been converted to drains — = 1
[460/660 > 50%]

Curb and Gutter Streets
Upper Third: 20% urbanized with 210 m curb/gutter — Code = 0
[210/870 < 50%]
Middle Third: 70% urbanized with 920 m curb/gutter — = 1
[920/1430 > 50%]
Lower Third: 55% urbanized with 970 m curb/gutter — = 1
[970/1710 > 50%]

Total BDF = 7

Example 5.2 (CU). Information is first collected from topographic maps and a field survey for the following watershed. The watershed is divided into three subareas of approximately equal area. The separation is based on homogeneity of hydrologic conditions, with the following values measured:

Subarea	Area (ac)	Main Channel Length (ft)	Length of Secondary Tributaries (ft)	Road Length (ft)	Length of Channel Modified (ft)	Length of Channel Lined (ft)	Storm Drains (ft)	Curb and Gutter (ft)
Upper	72.2	2,560	5,180	2,850	460	0	1,350	690
Middle	89.7	3,740	3,940	4,690	2,020	1,770	2,230	3,020
Lower	83.5	2,990	2,170	5,610	1,720	1,570	1,510	3,180
Sum	245.4	9,290						

The BDF is determined as follows:

Channel Modifications
Upper Third: 460 ft have been straightened and deepened Code = 0
[460/2,560 < 50%]
Middle Third: 2,020 ft have been straightened and deepened = 1
[2,020/3,740 > 50%]
Lower Third: 1,720 have been straightened and widened = 1
[1,720/2,990 > 50%]

Channel Linings
Upper Third: 0 ft of channel are lined Code = 0
[0/2,560 < 50%]
Middle Third: 1,770 ft of channel are lined = 0
[1,770/3,740 < 50%]
Lower Third: 1,570 of channel are lined = 1
[1,570/2,990 > 50%]

Storm Drains on Secondary Tributaries
Upper Third: 1,350 ft have been converted to drains Code = 0
[1,350/5,180 < 50%]
Middle Third: 2,230 ft have been converted to drains = 1
[2,230/3,940 > 50%]
Lower Third: 1,510 ft have been converted to drains = 1
[1,510/2,170 > 50%]

Curb and Gutter Streets
Upper Third: 20% urbanized with 690 ft curb/gutter Code = 0
[690/2,850 < 50%]
Middle Third: 70% urbanized with 3,020 ft curb/gutter = 1
[3,020/4,690 > 50%]
Lower Third: 55% urbanized with 3,180 ft curb/gutter = 1
[3,180/5,610 > 50%]

Total BDF = 7

Example 5.3. The 25-year peak discharge is computed for an urban watershed of 67 km^2 (26 mi^2) with a BDF of 4. The percentage increase over the undeveloped rural condition is also computed.

1. Determine the equivalent rural discharge using the published USGS statewide regression equation. For this site, the 25-year peak discharge for the rural conditions is determined from the following equation:

Variable	Value in SI	Value in CU
$RQ_{25} = a_{25} A^{C_T}$	$= 4.21(67)^{0.666} = 69\ m^3/s$	$= 280(26)^{0.666} = 2450\ ft^3/s$

2. Determine the urbanized discharge:

$$UQ_{25} = a_{25}\ A^{C_{1,25}}\ (13 - BDF)^{C_{2,25}}\ RQ_{25}^{C_{3,25}}$$

Value in SI	Value in CU
$UQ_{25} = 3.69 A^{0.15} (13 - BDF)^{-0.34} RQ^{0.80}$ $= 3.69(67)^{0.15}(13-4)^{-0.34}(69)^{0.80}$ $= 97\ m^3/s$	$UQ_{25} = 8.68 A^{0.15} (13 - BDF)^{-0.34} RQ^{0.80}$ $= 8.68(26)^{0.15}(13-4)^{-0.34}(2{,}450)^{0.80}$ $= 3{,}450\ ft^3/s$

3. Determine the percent change:

Variable	Value in SI	Value in CU
$\%\ change = \frac{UQ_{25} - RQ_{25}}{RQ_{25}} \times 100\%$	$= \frac{97 - 69}{69} \times 100\% = 41\%$	$= \frac{3450 - 2450}{2450} \times 100\% = 41\%$

5.1.3.3 Effects of Future Urbanization

The regression equations can also be used to determine the effects of future urbanization upon peak discharges. This calculation is simplified by performing some algebraic manipulation of the regression equations. This is illustrated by showing the impact on the 5-year peak discharge when the BDF changes from 5 to 10.

For the present and future conditions, the 5-yr peak discharge is computed with Equation 5.6:

$$UQ_5 = a_5\ A^{0.17}\ (13 - BDF_i)^{-0.39}\ RQ_5^{0.78}$$

where i = p and i = f for the present and the future BDF, respectively. The change in the BDF is:

$$\Delta BDF = (\ BDF_f - BDF_p\ ) \qquad (5.7)$$

which can be rearranged to:

$$BDF_f = BDF_p + \Delta BDF \tag{5.8}$$

The ratio of the future UQ_{5f} to the present UQ_{5p} is:

$$\frac{UQ_{5f}}{UQ_{5p}} = \frac{a_5 \ A^{0.17} \ \left[13 - \left(BDF_p + \Delta BDF\right)\right]^{-0.39} RQ_5^{0.78}}{a_5 \ A^{0.17} \ (13 - BDF_p)^{0.39} RQ_5^{0.78}} \tag{5.9}$$

Canceling the common terms and rearranging yields:

$$\frac{UQ_{5f}}{UQ_{5p}} = \left[1 - \frac{\Delta BDF}{13 - BDF_p}\right]^{-0.39} \tag{5.10}$$

For this example, BDF_p = 5 and ∆BDF = (10 - 5); therefore:

$$\frac{UQ_{5f}}{UQ_{5p}} = \left[1 - \frac{5}{8}\right]^{-0.39} = 1.47$$

Thus, the future 5-year peak discharge is 47 percent higher than the present 5-year peak discharge.

The same approach can be applied to the other recurrence intervals yielding the following general equation:

$$\frac{UQ_f}{UQ_p} = \left[1 - \frac{\Delta BDF}{13 - BDF_p}\right]^{C_{2T}} \tag{5.11}$$

where C_{2T} varies with recurrence intervals as given in Table 5.3.

5.1.3.4 Local Urban Equations

Many of the USGS regression studies include additional equations for some cities and metropolitan areas that were developed for local use in those designated areas only. These local urban equations can be used in lieu of the nationwide urban equations, or they can be used for comparative purposes. It would be highly coincidental for the local equations and the nationwide equations to give identical results.

Therefore, it is advisable to compare results of the two (or more) sets of urban equations, and to also compare the urban results to the equivalent rural results. Ultimately, it is the engineer's decision as to which urban results to use.

Local urban equations are available in many cities throughout the United States. In addition, some of the rural reports contain estimation techniques for urban watersheds. Several of the rural reports suggest the use of the nationwide equations given by Sauer, et al. (1983).

5.1.4 National Flood Frequency Program

Because of the common usage of the USGS equations developed for individual states and regions, the USGS has developed software called the National Flood Frequency Program (Jennings, Thomas, and Riggs, 1994). The USGS, in cooperation with the Federal Highway Administration and the Federal Emergency Management Agency, has compiled all of the current (as of September 1993) statewide and metropolitan area regression equations into a microcomputer program titled the National Flood Frequency Program. This program summarizes regression equations for estimating flood-peak discharges and techniques for estimating a typical flood hydrograph for a given recurrence interval or exceedence probability peak discharge for unregulated rural and urban watersheds. The report summarizes the statewide regression equations for rural watersheds in each state, summarizes the applicable metropolitan area or tatewide regression equations for urban watersheds, describes the National Flood Frequency software for making these computations, and provides much of the reference information and input data needed to run the computer program.

Since 1973, regression equations for estimating flood-peak discharges for rural, unregulated watersheds have been published, at least once, for every state and the Commonwealth of Puerto Rico. For some areas of the Nation, however, data are still inadequate to define flood-frequency characteristics. Regression equations for estimating urban flood-peak discharges for many metropolitan areas are also available. Typical flood hydrographs corresponding to a given rural and urban peak discharge can also be estimated by procedures described in the NFF report.

Information on computer specifications and the computer program is presented in appendices of the NFF report. Instructions for installing NFF on a personal computer are also given, in addition to a description of the NFF program and the associated database of regression statistics.

5.1.5 FHWA Regression Equations

In 1977, the Federal Highway Administration published a two-volume report by Fletcher, et al. (1977) that presents nationwide regression equations for predicting runoff from small rural watersheds (<130 km^2 or <50 mi^2). This method is not the equivalent of the USGS regression equations. While it was used rather widely at first, it is rarely used today. The procedure is similar in concept to that of Potter (1961). It was developed using frequency analyses of data in over 1000 small watersheds throughout the United States and Puerto Rico to relate peak flows to various hydrographic and physiographic characteristics. Three-, five-, and seven-parameter regression equations were developed for the 10-year peak runoff for each of 24 hydrophysiographic regions. Since the standard errors of estimate were found to be approximately the same for each regression equation option, the following discussion is limited to the three-parameter equations only.

If a drainage structure is to be designed to carry the probable maximum flood peak, $Q_{p(max)}$ in m^3/s (ft^3/s), Fletcher, et al. (1977) give the equation:

$$Q_{p(max)} = 10^{[C_0 + C_1 \log A + C_2 (\log A)^2]} \qquad (5.12)$$

where,

log A = base-10 logarithm of the drainage area, km^2 (mi^2)

$Q_{p(max)}$ = discharge, m^3/s (ft^3/s)

C_0 , C_1 , and C_2 = regression coefficients equal to 2.031, 0.8389, and –0.0325, respectively, in SI units and 3.920, 0.8120, and –0.0325, respectively, in CU units.

If it is feasible to construct a very large drainage structure to handle this probable maximum flow, the hydrologic analysis is essentially complete. Similarly, if a minimum size drainage structure is specified, and its carrying capacity is greater than $Q_{p(max)}$, no further analysis is required.

A more common problem in highway drainage is that the structure must be designed to handle a flow of specified frequency. This can be accomplished with the three-parameter FHWA regression equations. The basic form of these equations is:

$$\hat{q}_{10} = a\, A^{b_1} R^{b_2} E_c^{b_3} \quad (5.13)$$

where,

$\hat{q}_{10}$ = 10-year peak discharge, m^3/s (ft^3/s)

A = drainage area, km^2 (mi^2)

R = isoerodent factor defined as the product of the mean annual rainfall kinetic energy and the maximum respective 30-minute annual maximum rainfall intensity

E_c = difference in elevation measured along the main channel from the drainage structure site to the drainage basin boundary, m (ft)

a, b_1, b_2, and b_3 = regression coefficients.

Values of the drainage area and elevation difference are readily determined from topographic maps and R is taken from individual state isoerodent maps given by Fletcher, et al. (1977).

Two options are available to use the three-parameter regression equations. The first involves the application of an equation of the same form as Equation 5.13 for a specific hydrophysiographic zone. Twenty-four zones are defined covering the United States and Puerto Rico and each has its own regression equation for q_{10}. The second option involves the use of an all-zone equation developed from all of the data. The all-zone, three-parameter equation for the 10-year peak discharge, $q_{10(3AZ)}$, is:

$$\hat{q}_{10(3AZ)} = 0.02598\ A^{0.56172}\ R^{0.94356}\ E_c^{0.16887} \quad (5.14)$$

For each of the 24 hydrophysiographic zones, is a correction equation is presented to adjust Equation 5.15 for zonal bias. These correction equations have the form:

$$\hat{q}_{10} = a_1\ \hat{q}_{10(3AZ)}^{b_1} \quad (5.15)$$

where,

a_1 and b_1 = regression coefficients.

If the surface area of surface water storage is more than about 4 percent of the total drainage area, it is recommended that the value of q_{10} computed from an individual zone equation or the corrected all-zone equation be further adjusted with a storage-correction multiplier given with the equations.

Fletcher, et al. (1977) presented the following equations from which a frequency curve can be drawn on any appropriate probability paper:

$$Q_{2.33} = 0.47329\ \hat{q}_{10}^{1.00243} \tag{5.16}$$

$$Q_{50} = 1.58666\ \hat{q}_{10}^{1.02342} \tag{5.17}$$

$$Q_{100} = 1.82393\ \hat{q}_{10}^{1.02918} \tag{5.18}$$

where,

$Q_{2.33}$ = mean annual peak flow taken at a return period of 2.33 years
Q_{50} and Q_{100} = 50- and 100-year peak flows, respectively.

From this curve, the flow for any other selected design frequency can be determined.

The concept of risk can also be incorporated into the FHWA regression equations. Recall that risk is the probability that one or more floods will exceed the design discharge within the life of the project. Methods presented by Fletcher, et al. (1977) permit the return period of the design flood to be adjusted according to the risk the designer can accept. The concept of the probable maximum peak flow is also useful because it represents the upper limit of flow that might be expected. It can, therefore, have application to situations where the consequences of failure are very large or unacceptable.

5.2 SCS GRAPHICAL PEAK DISCHARGE METHOD

For many peak discharge estimation methods, the input includes variables to reflect the size of the contributing area, the amount of rainfall, the potential watershed storage, and the time-area distribution of the watershed. These are often translated into input variables such as the drainage area, the depth of rainfall, an index reflecting land use and soil type, and the time of concentration. The SCS graphical peak discharge method is typical of many peak discharge methods that are based on input such as that described.

5.2.1 Runoff Depth Estimation

The volume of storm runoff can depend on a number of factors. Certainly, the volume of rainfall will be an important factor. For very large watersheds, the volume of runoff from one storm event may depend on rainfall that occurred during previous storm events. However, when using the design storm approach, the assumption of storm independence is quite common.

In addition to rainfall, other factors affect the volume of runoff. A common assumption in hydrologic modeling is that the rainfall available for runoff is separated into three parts: direct (or storm) runoff, initial abstraction, and losses. Factors that affect the split between losses and direct runoff include the volume of rainfall, land cover and use, soil type, and antecedent moisture conditions. Land cover and land use will determine the amount of depression and interception storage.

In developing the SCS rainfall-runoff relationship, the total rainfall was separated into three components: direct runoff (Q), actual retention (F), and the initial abstraction (I_a). The retention F was assumed to be a function of the depths of rainfall and runoff and the initial abstraction. The development of the equation yielded:

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \tag{5.19}$$

where,

P = depth of precipitation, mm (in)
I_a = initial abstraction, mm (in)
S = maximum potential retention, mm (in)
Q= depth of direct runoff, mm (in).

Given Equation 5.19, two unknowns need to be estimated, S and I_a. The retention S should be a function of the following five factors: land use, interception, infiltration, depression storage, and antecedent moisture.

Empirical evidence resulted in the following equation for estimating the initial abstraction:

$$I_a = 0.2S \tag{5.20}$$

If the five factors above affect S, they also affect I_a. Substituting Equation 5.20 into Equation 5.19 yields the following equation, which contains the single unknown S:

$$Q = \frac{(P - 0.2\ S)^2}{P + 0.8\ S} \tag{5.21}$$

Equation 5.21 represents the basic equation for computing the runoff depth, Q, for a given rainfall depth, P. It is worthwhile noting that while Q and P have units of depth, Q and P reflect volumes and are often referred to as volumes because it is usually assumed that the same depths occurred over the entire watershed.

Additional empirical analyses were made to estimate the value of S. The studies found that S was related to soil type, land cover, and the hydrologic condition of the watershed. These are represented by the runoff curve number (CN), which is used to estimate S by:

$$S = \alpha \left[\frac{1000}{CN} - 10 \right] \tag{5.22}$$

where

CN = index that represents the combination of a hydrologic soil group and a land use and treatment class

α = unit conversion constant equal to 25.4 in SI units and 1.0 in CU units.

Empirical analyses suggested that the CN was a function of three factors: soil group, the cover complex, and antecedent moisture conditions.

5.2.2 Soil Group Classification

SCS developed a soil classification system that consists of four groups, which are identified by the letters A, B, C, and D. Soil characteristics that are associated with each group are as follows:

Group A: deep sand, deep loess; aggregated silts

Group B: shallow loess; sandy loam

Group C: clay loams; shallow sandy loam; soils low in organic content; soils usually high in clay

Group D: soils that swell significantly when wet; heavy plastic clays; certain saline soils

The SCS soil group can be identified at a site using either soil characteristics or county soil surveys. The soil characteristics associated with each group are listed above and provide one means of identifying the SCS soil group. County soil surveys, which are made available by Soil Conservation Districts, give detailed descriptions of the soils at locations within a county; these surveys are usually the better means of identifying the soil group. Many of the more recent reports actually categorize the soils into these four groups.

5.2.3 Cover Complex Classification

The SCS cover complex classification consists of three factors: land use, treatment or practice, and hydrologic condition. Many different land uses are identified in the tables for estimating runoff curve numbers. Agricultural land uses are often subdivided by treatment or practices, such as contoured or straight row; this separation reflects the different hydrologic runoff potential that is associated with variation in land treatment. The hydrologic condition reflects the level of land management; it is separated into three classes: poor, fair, and good. Not all of the land uses are separated by treatment or condition.

5.2.4 Curve Number Tables

Table 5.4 shows the SCS CN values for the different land uses, treatments, and hydrologic conditions; separate values are given for each soil group. For example, the CN for a wooded area with good cover and soil group B is 55; for soil group C, the CN would increase to 70. If the cover (on soil group B) is poor, the CN will be 66.

Table 5.4. Runoff Curve Numbers (average watershed condition, $I_a = 0.2S$)(After: SCS, 1986)

Cover Type		Curve Numbers for Hydrologic Soil Group			
		A	B	C	D
Fully developed urban areas[a] (vegetation established)					
Lawns, open spaces, parks, golf courses, cemeteries, etc.					
Good condition; grass cover on 75% or more of the area		39	61	74	80
Fair condition; grass cover on 50% to 75% of the area		49	69	79	84
Poor condition; grass cover on 50% or less of the area		68	79	86	89
Paved parking lots, roofs, driveways, etc. (excl. right-of- way)		98	98	98	98
Streets and roads					
Paved with curbs and storm sewers (excl. right-of-way)		98	98	98	98
Gravel (incl. right-of-way)		76	85	89	91
Dirt (incl. right-of-way)		72	82	87	89
Paved with open ditches (incl. right-of-way)		83	89	92	93
	Average % impervious[b]				
Commercial and business areas	85	89	92	94	95
Industrial districts	72	81	88	91	93
Row houses, town houses, and residential with lots sizes 0.05 ha or less (0.12 acres or less)	65	77	85	90	92
Residential: average lot size					
0.1 ha (0.25 acres)	38	61	75	83	87
0.135 ha (0.33 acres)	30	57	72	81	86
0.2 ha (0.5 acres)	25	54	70	80	85
0.4 ha (1.0 acres)	20	51	68	79	84
0.8 ha (2.0 acres)	12	46	65	77	82
Western desert urban areas:					
Natural desert landscaping (pervious areas only)		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 25- to 50-mm (1- to 2-in) sand or gravel mulch and basin borders)		96	96	96	96
Developing urban areas[c] (no vegetation established) Newly graded area		77	86	91	94

Table 5.4. Runoff Curve Numbers (Cont'd)

Cover Type		Hydrologic Condition[d]	Curve Numbers for Hydrologic Soil Group			
			A	B	C	D
Cultivated Agricultural Land: Fallow						
Straight row or bare soil			77	86	91	94
Conservation tillage		Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row	Poor	72	81	88	91
		Good	67	78	85	89
	Conservation tillage	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured	Poor	70	79	84	88
		Good	65	75	82	86
	Contoured and tillage	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured and terraces	Poor	66	74	80	82
		Good	62	71	78	81
	Contoured and terraces and conservation tillage	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	Straight row	Poor	65	76	84	88
		Good	63	75	83	87
	Conservation tillage	Poor	64	75	83	86
		Good	60	72	80	84
	Contoured	Poor	63	74	82	85
		Good	61	73	81	84
	Contoured and tillage	Poor	62	73	81	84
		Good	60	72	80	83
	Contoured and terraces	Poor	61	72	79	82
		Good	59	70	78	81
	Contoured and terraces and conservation tillage	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadows[e]	Straight row	Poor	66	77	85	89
		Good	58	72	81	85
	Contoured	Poor	64	75	83	85
		Good	55	69	78	83
	Contoured and terraces	Poor	63	73	80	83
		Good	57	67	76	80
Noncultivated agricultural land						
Pasture or range	No Mechanical treatment[i]	Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
		Fair	25	59	75	83
		Good	6	35	70	79
Meadow - continuous grass, protected from grazing and generally mowed for hay			30	58	71	78

Table 5.4. Runoff Curve Numbers (Cont'd)

Cover Type	Hydrologic Condition[d]	Curve Numbers for Hydrologic Soil Group			
		A	B	C	D
Forestland - grass or orchards - evergreen or Deciduous	Poor	55	73	82	86
	Fair	44	65	76	82
	Good	32	58	72	79
Brush - brush-weed-grass mixture with brush the major element[g]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30[f]	48	65	73
Woods	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30[f]	55	70	77
Woods - grass combination (orchard or tree farm)[h]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Farmsteads		59	74	82	86
Forest-range					
Herbaceous - mixture of grass, weeds, and low-growing brush, with brush the minor element	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Oak-aspen - mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple and other brush	Poor		66	74	79
	Fair		48	57	63
	Good		30	41	48
Pinyon - juniper - pinyon, juniper, or both grass understory)	Poor		75	85	89
	Fair		58	73	80
	Good		41	61	71
Sage-grass	Poor		67	80	85
	Fair		51	63	70
	Good		35	47	55
Desert shrub - major plants include saltbush, greasewood, creosotebush, blackbrush, bursage, palo verde, mesquite, and cactus	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84

a For land uses with impervious areas, curve numbers are computed assuming that 100 percent of runoff from impervious areas is directly connected to the drainage system. Pervious areas (lawn) are considered to be equivalent to lawns in good condition and the impervious areas have a CN of 98.

b Includes paved streets.

c Use for the design of temporary measures during grading and construction. Impervious area percent for urban areas under development vary considerably. The user will determine the percent impervious. Then using the newly graded area CN, the composite CN can be computed for any degree of development.

d For conservation tillage poor hydrologic condition, 5 to 20 percent of the surface is covered with residue (less than 850 kg/ha (760 lbs/acre) row crops or 350 kg/ha (310 lbs/acre) small grain). For conservation tillage good hydrologic condition, more than 20 percent of the surface is covered with residue (greater than 850 kg/ha (760 lbs/acre) row crops or 350 kg/ha (310 lbs/acre) small grain).

e Close-drilled or broadcast.
For noncultivated agricultural land:
Poor hydrologic condition has less than 25 percent ground cover density.
Fair hydrologic condition has between 25 and 50 percent ground cover density.
Good hydrologic condition has more than 50 percent ground cover density.
For forest-range.
Poor hydrologic condition has less than 30 percent ground cover density.
Fair hydrologic condition has between 30 and 70 percent ground cover density.
Good hydrologic condition has more than 70 percent ground cover density.

f Actual curve number is less than 30: use CN = 30 for runoff computations.

g CNs shown were computed for areas with 50 percent woods and 50 percent grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.

h Poor: < 50 percent ground cover.
Fair: 50 to 75 percent ground cover.
Good: > 75 percent ground cover.

i Poor: < 50 percent ground cover or heavily grazed with no mulch.
Fair: 50 to 75 percent ground cover and not heavily grazed.
Good: > 75 percent ground cover and lightly or only occasionally grazed.

5.2.5 Estimation of CN Values for Urban Land Uses

The CN table (Table 5.4) includes CN values for a number of urban land uses. For each of these, the CN is based on a specific percentage of imperviousness. For example, the CN values for commercial land use are based on an imperviousness of 85 percent. Curve numbers for other percentages of imperviousness can be computed using a weighted CN approach, with a CN of 98 used for the impervious areas and the CN for open space (good condition) used for the pervious portion of the area. Thus CN values of 39, 61, 74, and 80 are used for hydrologic soil groups A, B, C, and D, respectively. These are the same CN values for pasture in good condition. Thus the following equation can be used to compute a weighted CN:

$$CN_w = CN_p\left(1 - f\right) + f\left(98\right) \tag{5.23}$$

in which f is the fraction (not percentage) of imperviousness. To show the use of Equation 5.23, the CN values for commercial land use with 85 percent imperviousness are:

A soil: 39(0.15) + 98(0.85) = 89
B soil: 61(0.15) + 98(0.85) = 92
C soil: 74(0.15) + 98(0.85) = 94
D soil: 80(0.15) + 98(0.85) = 95

These are the same values shown in Table 5.4.

Equation 5.23 can be placed in graphical form (see Figure 5.2a). By entering with the percentage of imperviousness on the vertical axis at the center of the figure and moving horizontally to the pervious area CN, the composite CN can be read. The examples above for commercial land use can be used to illustrate the use of Figure 5.2a for 85 percent imperviousness. For a commercial land area with 60 percent imperviousness of a B soil, the composite CN would be:

$$CN_w = 61(0.4) + 98(0.6) = 83$$

The same value can be obtained from Figure 5.3a.

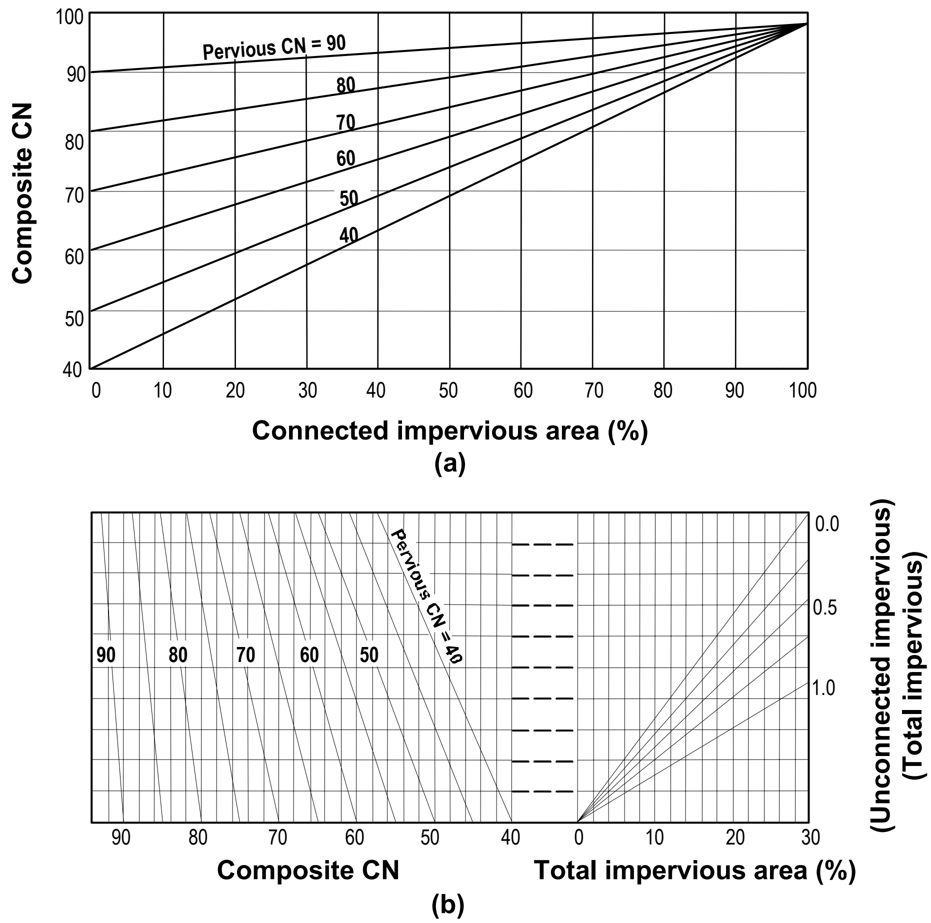

Figure 5.3. Composite curve number estimation:
(a) all imperviousness area connected to storm drains
(b) some imperviousness area not connected to storm drain

5.2.6 Effect of Unconnected Impervious Area on Curve Numbers

Many local drainage policies are requiring runoff that occurs from certain types of impervious land cover (i.e., rooftops, driveways, patios) to be directed to pervious surfaces rather than being connected to storm drain systems. Such a policy is based on the belief that disconnecting these impervious areas will require smaller and less costly drainage systems and lead both to increased ground water recharge and to improvements in water quality. If disconnecting some impervious surfaces will reduce both the peak runoff rates and volumes of direct flood runoff, credit should be given in the design of drainage systems. The effect of disconnecting impervious surfaces on runoff rates and volumes can be accounted for by modifying the CN.

There are three variables involved in the adjustment: the pervious area CN, the percentage of impervious area, and the percentage of the imperviousness that is unconnected. Because Figure 5.3a for computing composite CN values is based on the pervious area CN and the percentage of imperviousness, a correction factor was developed to compute the composite CN. The correction is a function of the percentage of unconnected imperviousness, which is shown in Figure 5.3b. The use of the correction is limited to drainage areas having percentages of imperviousness that are less than 30 percent.

As an alternative to Figure 5.3b, the composite curve number (CN_c) can be computed by:

$$CN_c = CN_p + (P_i/100)(98 - CN_p)(1 - 0.5R) \quad \text{for } P_i \le 30\% \tag{5.24}$$

where,

P_i = percent imperviousness
R = ratio of unconnected impervious area to the total impervious area.

Equation 5.24, like Figure 5.3b, is limited to cases where the total imperviousness (P_i) is less than 30 percent.

5.2.7 I_a/P Parameter

I_a/P is a parameter that is necessary to estimate peak discharge rates. I_a denotes the initial abstraction, and P is the 24-hour rainfall depth for a selected return period. For a given 24-hour rainfall distribution, I_a/P represents the fraction of rainfall that must occur before runoff begins.

5.2.8 Peak Discharge Estimation

The following equation can be used to compute a peak discharge with the SCS method:

$$q_p = q_u A Q \tag{5.25}$$

where,

q_p = peak discharge, m^3/s (ft^3/s)
q_u = unit peak discharge, $m^3/s/km^2/mm$ ($ft^3/s/mi^2/in$)
A = drainage area, km^2 (mi^2)
Q = depth of runoff, mm (in).

The unit peak discharge is obtained from the following equation, which requires the time of concentration (t_c) in hours and the initial abstraction/rainfall (I_a/P) ratio as input:

$$q_u = \alpha\, 10^{C_0 + C_1 \log t_c + C_2 [\log (t_c)]^2} \tag{5.26}$$

where,

C_o, C_1, and C_2 = regression coefficients given in Table 5.5 for various I_a/P ratios

α = unit conversion constant equal to 0.000431 in SI units and 1.0 in CU units.

The runoff depth (Q) is obtained from Equation 5.21 and is a function of the depth of rainfall P and the runoff CN. The I_a/P ratio is obtained directly from Equation 5.20.

Table 5.5. Coefficients for SCS Peak Discharge Method

Rainfall Type	I_a/P	C_0	C_1	C_2
I	0.10	2.30550	-0.51429	-0.11750
	0.20	2.23537	-0.50387	-0.08929
	0.25	2.18219	-0.48488	-0.06589
	0.30	2.10624	-0.45695	-0.02835
	0.35	2.00303	-0.40769	0.01983
	0.40	1.87733	-0.32274	0.05754
	0.45	1.76312	-0.15644	0.00453
	0.50	1.67889	-0.06930	0.0
IA	0.10	2.03250	-0.31583	-0.13748
	0.20	1.91978	-0.28215	-0.07020
	0.25	1.83842	-0.25543	-0.02597
	0.30	1.72657	-0.19826	0.02633
	0.50	1.63417	-0.09100	0.0
II	0.10	2.55323	-0.61512	-0.16403
	0.30	2.46532	-0.62257	-0.11657
	0.35	2.41896	-0.61594	-0.08820
	0.40	2.36409	-0.59857	-0.05621
	0.45	2.29238	-0.57005	-0.02281
	0.50	2.20282	-0.51599	-0.01259
III	0.10	2.47317	-0.51848	-0.17083
	0.30	2.39628	-0.51202	-0.13245
	0.35	2.35477	-0.49735	-0.11985
	0.40	2.30726	-0.46541	-0.11094
	0.45	2.24876	-0.41314	-0.11508
	0.50	2.17772	-0.36803	-0.09525

The peak discharge obtained from Equation 5.26 assumes that the topography is such that surface flow into ditches, drains, and streams is relatively unimpeded. Where ponding or wetland areas occur in the watershed, a considerable amount of the surface runoff may be retained in temporary storage. The peak discharge rate should be reduced to reflect this condition of increased storage. Values of the pond and swamp adjustment factor (F_p) are provided in Table 5.6. The adjustment factor values in Table 5.6 are a function of the percent of the total watershed area in ponds and wetlands. If the watershed includes significant portions of pond and wetland storage, the peak discharge of Equation 5.25 can be adjusted using the following:

$$q_a = q_p F_p \tag{5.27}$$

where,

q_a = adjusted peak discharge, m^3/s (ft^3/s).

Table 5.6. Adjustment Factor (F_p) for Pond and Wetland Areas

Area of Pond and Wetland (%)	**F_p**
0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

The SCS method has a number of limitations. When these conditions are not met, the accuracy of estimated peak discharges decreases. The method should be used on watersheds that are homogeneous in CN; where parts of the watershed have CNs that differ by 5, the watershed should be subdivided and analyzed using a hydrograph method, such as TR-20 (SCS, 1984). The SCS method should be used only when the CN is 50 or greater and the t_c is greater than 0.1 hour and less than 10 hours. Also, the computed value of I_a/P should be between 0.1 and 0.5. The method should be used only when the watershed has one main channel or when there are two main channels that have nearly equal times of concentration; otherwise, a hydrograph method should be used. Other methods should also be used when channel or reservoir routing is required, or where watershed storage is either greater than 5 percent or located on the flow path used to compute the t_c.

Example 5.4. A small watershed (17.6 ha) is being developed and will include the following land uses: 10.6 ha of residential (0.1 ha lots), 5.2 ha of residential (0.2 ha lots), 1.2 ha of commercial property (85 percent impervious), and 0.4 ha of woodland. The development will necessitate upgrading of the drainage of a local roadway at the outlet of the watershed. The peak discharge for a 10-year return period is determined using the SCS graphical method.

The weighted CN is computed using the CN values of Table 5.4:

Land Cover	Lot Size (ha)	Lot Size (acres)	Soil Group	5.2.8.1.1	Area (ha)	Area (acres)	A*CN (ha)	A*CN (acres)
Residential	0.2	0.5	B	70	5.2	12.8	364	896
Residential	0.1	0.25	B	75	4.6	11.4	345	855
Residential	0.1	0.25	C	83	6.0	14.8	498	1228
Commercial (85% Imp.)			C	94	1.2	3.0	113	282
Woodland (Good condition)			C	70	0.6	1.5	42	105
Total					17.6	43.5	1,362	3366

The weighted CN is:

Variable	Value in SI	Value in CU
$CN_w = \frac{\sum A * CN}{\sum A}$	$= \frac{1{,}362}{17.6} = 77.4\ (use\ 77)$	$= \frac{3{,}366}{43.5} = 77.4\ (use\ 77)$

The time of concentration is computed using the velocity method for conditions along the principal flowpath:

Conveyance Type	Slope (%)	K	Length (m)	V (m/s)	Length (ft)	V (ft/s)	T_t (h)
Woodland (overland)	2.3	0.152	25	0.23	82	0.76	0.03
Grassed waterway	2.1	0.457	275	0.66	902	2.19	0.12
Grassed waterway	1.8	0.457	250	0.61	820	2.02	0.11
Concrete-lined channel	1.8	-	50	4.62	164	15.1	0.00
			600		1968		0.26

The velocity was computed for the concrete-lined channel using Manning's equation, with n = 0.013 and hydraulic radius of 0.3 m (1ft). The sum of the travel times for the principal flowpath is 0.26 hours.

The rainfall depth is obtained from an IDF curve for the locality using a storm duration of 24 hours and a 10-year return period. (Note that the t_c is not used to find the rainfall depth when using the SCS graphical method. A storm duration of 24 hours is used.) For this example, a 10-year rainfall depth of 122 mm (4.8 in) is assumed. For a CN of 77, S equals 76 mm (3.0 in) and I_a equals 15 mm (0.6 in). Thus, I_a/P is 0.12. The rainfall depth is computed with Equation 5.21:

Variable	Value in SI	Value in CU
$Q = \frac{(P - 0.2S)^2}{P + 0.8\ S}$	$= \frac{(122\ - 0.2(76))^2}{122\ + 0.8(76)} = 62\ mm$	$= \frac{(4.8\ - 0.2(3.0))^2}{4.8\ + 0.8(3.0)} = 2.45\ in$

The unit peak discharge is computed with Equation 5.26 by interpolating c_0 , c_1, and c_2 from Table 5.5 using a type II distribution. The peak discharge is also calculated as follows.

Variable	SI Unit	CU Unit
$q_u = 10^{2.5444 - 0.61587\ \log(0.26) - 0.15928[\log(0.26)]}$	= (0.000431) $10^{2.85}$ = 0.305 $m^3/s/km^2/mm$	= (1) $10^{2.85}$ = 708 $ft^3/s/mi^2/in$
$q_p = q_u A Q$	= 0.305 (0.176 km^2)(62 mm) = 3.3 m^3/s	= 708 (0.068 mi^2) (2.46 in) = 120 ft^3/s

5.3 RATIONAL METHOD

One of the most commonly used equations for the calculation of peak discharges from small areas is the rational formula. The rational formula is given as:

$$Q = \frac{1}{\alpha} C\, i\, A \qquad (5.28)$$

where,

- Q = the peak flow, m^3/s (ft^3/s)
- i = the rainfall intensity for the design storm, mm/h (in/h)
- A = the drainage area, ha (acres)
- C = dimensionless runoff coefficient assumed to be a function of the cover of the watershed and often the frequency of the flood being estimated
- α = unit conversion constant equal to 360 in SI units and 1 in CU units.

5.3.1 Assumptions

The assumptions in the rational formula are as follows:

1. The drainage area should be smaller than 80 hectares (200 acres).

2. The peak discharge occurs when the entire watershed is contributing.

3. A storm that has a duration equal to t_c produces the highest peak discharge for this frequency.

4. The rainfall intensity is uniform over a storm time duration equal to the time of concentration, t_c. The time of concentration is the time required for water to travel from the hydrologically most remote point of the basin to the outlet or point of interest.

5. The frequency of the computed peak flow is equal to the frequency of the rainfall intensity. In other words, the 10-year rainfall intensity, i, is assumed to produce the 10-year peak discharge.

5.3.2 Estimating Input Requirements

The runoff coefficient, C, is a function of ground cover. Some tables of C provide for variation due to slope, soil, and the return period of the design discharge. Actually, C is a volumetric coefficient that relates the peak discharge to the "theoretical peak" or 100 percent runoff, occurring when runoff matches the net rain rate. Hence C is also a function of infiltration and other hydrologic abstractions. Some typical values of C for the rational formula are given in

Table 5.7. Should the basin contain varying amounts of different covers, a weighted runoff coefficient for the entire basin can be determined as:

$$\text{Weighted } C = \frac{\sum C_i A_i}{A} \tag{5.29}$$

where,

C_i = runoff coefficient for cover type i that covers area A_i
A = total area.

5.3.3 Check for Critical Design Condition

When the rational method is used to design multiple drainage elements (i.e. inlets and pipes), the design process proceeds from upstream to downstream. For each design element, a time of concentration is computed, the corresponding intensity determined, and the peak flow computed. For pipes that drain multiple flow paths, the longest time of concentration from all of the contributing areas must be determined. If upstream pipes exist, the travel times in these pipes must also be included in the calculation of time of concentration.

In most cases, especially as computations proceed downstream, the contributing area with the longer time of concentration also contributes the greatest flow. Taking the case of two contributing areas, as shown in Figure 5.3a, the longest time of concentration of the two areas is used to determine the time of concentration for the combined area. When the rainfall intensity corresponding to this time of concentration is applied to the rational equation, as shown below, for the combined area and runoff coefficient, the appropriate design discharge, Q, results.

$$Q = \frac{1}{\alpha}(C_1 A_1 + C_2 A_2)\, i_1 \tag{5.30}$$

However, it may be possible for the larger contributing flows to be generated from the contributing area with a shorter time of concentration. If this occurs, it is also possible that, if the longer time of concentration is applied to the combined drainage area, the resulting design flow would be an underestimate. Therefore, a check for a critical design condition must be made.

$$Q' = \frac{1}{\alpha}\left(C_1 A_1 \frac{t_2}{t_1} + C_2 A_2\right) i_2 \tag{5.31}$$

where,

Q' = design check discharge
t_1 = time of concentration for area 1
t_2 = time of concentration for area 2.

Table 5.7. Runoff Coefficients for Rational Formula (ASCE, 1960)

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

If Q' > Q, Q' should be used for design; otherwise Q should be used. Equation 5.31 uses the rainfall intensity for the contributing area with the shorter time of concentration (area 2) and reduces the contribution of area 1 by the ratio of the times of concentration. This ratio approximates the fraction of the area that would contribute within the shorter duration. This is equivalent to reducing the contributing area as shown by the dashed line in Figure 5.4.

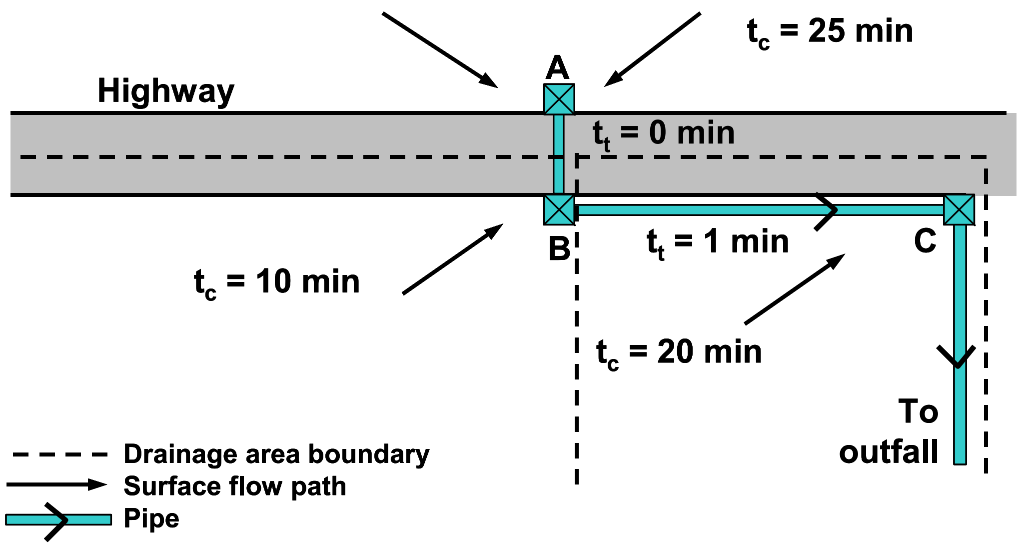

Figure 5.4. Storm drain system schematic

<u>Example 5.5</u>. A flooding problem exists along a farm road near Memphis, Tennessee. A low-water crossing is to be replaced by a culvert installation to improve road safety during rainstorms. The drainage area above the crossing is 43.7 ha (108 acres). The return period of the design storm is to be 25 years as determined by local authorities. The engineer must determine the maximum discharge that the culvert must pass for the indicated design storm.

The current land use consists of 21.8 ha (53.9 acres) of parkland, 1.5 ha (3.7 acres) of commercial property that is 100 percent impervious, and 20.4 ha (50.4 acres) of single-family residential housing. The principal flow path includes 90 m (295 ft) of short grass at 2 percent slope, 300 m (985 ft) of grassed waterway at 2 percent slope, and 650 m (2,130 ft) of grassed waterway at 1 percent slope. The following steps are used to compute the peak discharge with the rational method:

1. **Compute a Weighted Runoff Coefficient**: The tabular summary below uses runoff coefficients from Table 5.7. The average value is used for the parkland and the residential areas, but the highest value is used for the commercial property because it is completely impervious.

Description	C Value	SI Unit		CU Unit	
		Area (ha)	C_iA_i	Area (acres)	C_iA_i
Park	0.20	21.8	4.36	53.9	10.8
Commercial (100% impervious)	0.95	1.5	1.43	3.7	3.5
Single-family	0.40	20.4	8.16	50.4	20.2
Total		43.7	13.95	108.0	34.5

Equation 5.29 is used to compute the weighted C:

Variable	Value in SI	Value in CU
$\text{Weighted } C = \frac{\sum C_i A_i}{A}$	$= \frac{13.95}{43.7} = 0.32$	$= \frac{34.5}{108.0} = 0.32$

2. **Intensity**: The 25-year intensity is taken from an intensity-duration-frequency curve for Memphis. To obtain the intensity, the time of concentration, t_c, must first be estimated. In this example, the velocity method for t_c is used to compute t_c:

Flow Path	Slope(%)	SI Unit		CU Unit	
		Length (m)	Velocity (m/s)	Length (ft)	Velocity (ft/s)
Overland (Short grass)	2	90	0.30	295	1.0
Grassed waterway	2	300	0.64	985	2.1
Grassed waterway	1	650	0.46	2,130	1.5

The time of concentration is estimated as:

Variable	Value in SI	Value in CU
$T_c = \sum\left(\frac{L}{V}\right)$	$= \frac{90\ m}{0.3\ m/s} + \frac{300\ m}{0.64\ m/s} + \frac{650\ m}{0.46\ m/s}$ = 2,180 s = 36 min	$= \frac{295\ ft}{1.0\ ft/s} + \frac{985\ ft}{2.1\ ft/s} + \frac{2{,}130\ ft}{1.5\ ft/s}$ = 2,180 s = 36 min

The intensity is obtained from the IDF curve for the locality using a storm duration equal to the time of concentration:

$$i = 85 \text{ mm/h } (3.35 \text{ in/h})$$

3. **Area (A)**: Total area of drainage basin, A = 43.7 ha (108 acres)

4. **Peak Discharge (Q)**:

Variable	Value in SI	Value in CU
$Q = \frac{1}{\alpha} CiA$	$= \frac{(0.32)(85)(43.7)}{360} = 3.3\ m^3/s$	$= \frac{(0.32)(3.35)(108)}{1} = 116 ft^3/s$

5.4 INDEX FLOOD METHOD

Other methods exist for determining peak flows for various exceedence frequencies using regional methods where no data are available. The USGS index-flood method is representative of this group.

5.4.1 Procedure for Analysis

The index-flood method of regional analysis described by Dalrymple (1960) was used extensively in the 1960s and early 1970s. This method utilizes statistical analyses of data at meteorologically and hydrologically similar gages to develop a flood frequency curve at an ungaged site. There are two parts to the index-flood method. The first consists of developing the basic dimensionless ratio of a specified frequency flow to the index flow (usually the mean annual flood) and the second involves developing the relation between the drainage basin characteristics (usually the drainage area) and the mean annual flood.

The following steps are used to develop a regional flood frequency curve by the index-flood method:

1. Tabulate annual peak floods for all gages within the hydrologically similar region.

2. Select the base period of record. This is usually taken as the longest period of record.

3. Estimate floods for missing years by correlation with other data.

4. Assign an order to all floods (actual and estimated) at each station, compute the plotting positions, and compute and plot frequency curves using the best standard distribution fit for each gage.

5. Determine the mean annual flood for each gage as the discharge with a return period of 2.33 years. This is a graphical mean, which is more stable than the arithmetic mean, and its value is not affected as much by the inclusion or exclusion of major floods. It also gives a greater weight to the median floods than to the extreme floods where sampling errors may be larger. In some cases, the 2- or 10-year flood is used as the index flood.

6. Test the data for homogeneity. This is accomplished in the following manner.

   a. For each gage, compute the ratio of the flood with a 10-year return period, Q_{10}, to the station mean, $Q_{2.33}$. (Both of these values are obtained from the frequency analysis.)

   b. Compute the arithmetic average of the ratio $Q_{10}/Q_{2.33}$ for all the gages considered.

   c. For each gage, compute $Q_{2.33}$ $(Q_{10}/Q_{2.33})_{avg}$ and the corresponding return period.

   d. Plot the values of return period obtained in step c against the effective length of record, L_E, for each gage.

   e. Test for homogeneity by also plotting on this graph, envelope curves determined from Table 5.8, taken from Dalrymple (1960). This table gives the upper and lower limits, T_u and T_L, as a function of the effective length of record. (Table 5.8 applies only to homogeneity tests of the 10-year floods.) Return periods that fail this homogeneity test should be eliminated from the regional analysis.

7. Using actual flood data, compute the ratio of each flood to the index flood, $Q_{2.33}$, for each record.

8. Compute the median flood ratios of the stations retained in the regional analysis for each rank or order m, and compute the corresponding return period by the Weibull formula, $T_r = (n+1)/m$. (It is suggested that the median ratio be determined after eliminating the highest and lowest $Q/Q_{2.33}$ values for each ordered series of data.)

9. Plot the median-flood ratio against the return period on probability paper.

10. Plot the logarithm of the mean annual flood for each gage, $Q_{2.33}$ against the logarithm of the corresponding drainage area. This curve should be nearly a straight line.

11. Determine the flood frequency curve for any stream site in the watershed as follows:

    a. Determine the drainage area above the site.

    b. From Step 10, determine the value of $Q_{2.33}$.

    c. For selected return periods, multiply the median-flood ratio in step 9 by the value of $Q_{2.33}$ from Step 11b.

    d. Plot the regional frequency curve.

Table 5.8. Upper and Lower Limit Coordinates of Envelope Curve for Homogeneity Test (Dalrymple, 1960)

	Return Period Limits, T_r (yrs)	
Effective Length of Record, L_E (Yrs)	**Upper Limit**	**Lower Limit**
5	160	1.2
10	70	1.85
20	40	2.8
50	24	4.4
100	18	5.6

Example problems illustrating the index-flood method are contained in Dalrymple (1960), Sanders (1980), and numerous hydrology textbooks.

5.4.2 Other Considerations

As pointed out by Benson (1962), the index-flood method has some limitations that affect its reliability. The most significant is that there may be large differences in the index or mean annual floods throughout a region. This can lead to considerable variations in the various flood ratios even for watersheds of comparable size. Another shortcoming of the method is that homogeneity is established at the 10-year level, whereas at the higher levels the test may not be sustained. Still another deficiency pointed out by Benson is that all sizes of drainage areas (except the very largest) are included in the index-flood regional analysis. As discussed in Chapter 2, the larger the drainage area, the flatter the frequency curve will be. This effect is most noticeable at the higher return periods.

With the development of regional regression equations for peak-flow in most states, there is only limited application of the index-flood method today. It is used primarily as a check on other solution techniques and for those situations where other techniques are inapplicable or not available.

5.5 PEAK DISCHARGE ENVELOPE CURVES

Design storms are hypothetical constructs and have never occurred. Many design engineers like to have some assurance that a design peak discharge is unlikely to occur over the design life of a project. This creates an interest in comparing the design peak to actual peaks of record.

Crippen and Bue (1977) developed envelope curves for the conterminous United States, with 17 regions delineated as shown in Figure 5.5. Maximum floodflow data from 883 sites that have drainage areas less than 25,900 km^2 (10,000 mi^2) were plotted versus drainage area and upper envelope curves constructed. The curves for the 17 regions were fit to the following logarithmic polynomial model:

$$q_{envlpe} = K_1 A^{K_2} \left[L + A^{0.5}\right]^{K_3} \tag{5.32}$$

where,

q_{envlpe} = maximum flood flow envelope, m^3/s (ft^3/s)
L = length constant, 8.0 km (5.0 mi)
A = drainage area, km^2 (mi^2).

Table 5.9 gives the values of the coefficients (K_1, K_2, and K_3 of Equation 5.32) and the upper limit on the drainage area for each region. The curves are valid for drainage areas greater than 0.25 km^2 (0.1 mi^2). Crippen and Bue did not assign an exceedence probability to the flood flows used to fit the curves, so a probability cannot be given to values estimated from the curves.

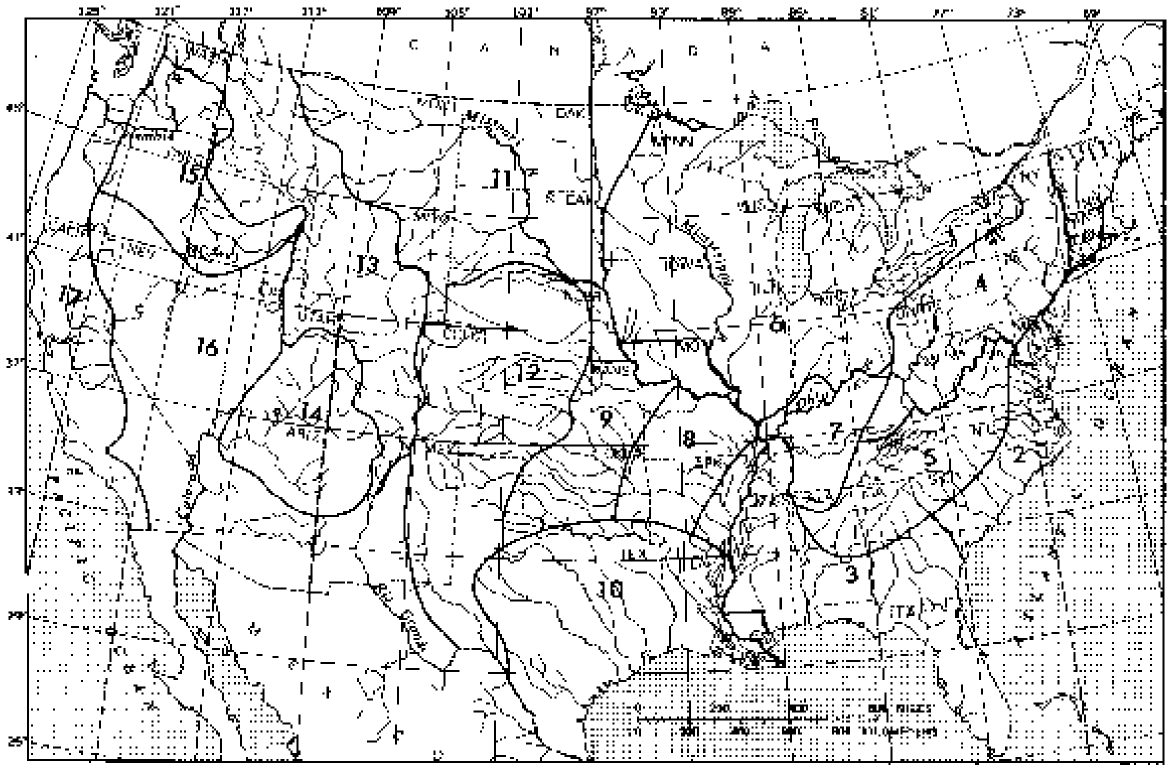

Figure 5.5. Map of the conterminous United States showing flood-region boundaries

Table 5.9. Coefficients for Peak Discharge Envelope Curves

(a) SI Unit

Region	Upper limit (km²)	Coefficients		
		K_1	K_2	K_3
1	26,000	469	0.895	-1.082
2	7,800	584	0.770	-0.897
3	26,000	1229	0.924	-1.373
4	26,000	929	0.938	-1.327
5	26,000	2939	0.838	-1.354
6	26,000	1517	0.937	-1.297
7	26,000	1142	0.883	-1.352
8	26,000	954	0.954	-1.357
9	26,000	1815	0.849	-1.368
10	2,600	1175	1.116	-1.371
11	26,000	917	0.919	-1.352
12	18,100	1944	0.935	-1.304
13	26,000	1504	0.873	-1.338
14	26,000	215	0.710	-0.844
15	50	2533	1.059	-1.572
16	2,600	1991	1.029	-1.341
17	26,000	1724	1.024	-1.461

(b) CU Unit

Region	Upper limit (mi²)	Coefficients		
		K_1	K_2	K_3
1	10,000	23200	0.895	-1.082
2	3,000	28000	0.770	-0.897
3	10,000	54400	0.924	-1.373
4	10,000	42600	0.938	-1.327
5	10,000	121000	0.838	-1.354
6	10,000	70500	0.937	-1.297
7	10,000	49100	0.883	-1.352
8	10,000	43800	0.954	-1.357
9	10,000	75000	0.849	-1.368
10	1,000	62500	1.116	-1.371
11	10,000	40800	0.919	-1.352
12	7,000	89900	0.935	-1.304
13	10,000	64500	0.873	-1.338
14	10,000	10000	0.710	-0.844
15	19	116000	1.059	-1.572
16	1,000	98900	1.029	-1.341
17	10,000	80500	1.024	-1.461