The regression equations for the skew coefficients were not statistically significant. Equations were derived for four regions of the United States. As an example, the regression equations for the mean ($\overline{X}$) and standard deviation (S) for rural watersheds in the eastern region are

$$\overline{X} = 0.00264A^{1.01}P^{1.58} \tag{7.16}$$

and

$$S = 0.0142A^{0.99}P^{0.85}, \tag{7.17}$$

in which A is the drainage area in square miles, and P is the mean annual precipitation in inches.

For given values of A and P, Equations 7.16 and 7.17 can be used to estimate the mean and standard deviation of the logarithms and compute a log-normal frequency curve. Since the skew is 0, the values of K in Equation 7.15 become the standard normal deviate.

For a 100-mi^2 watershed in the eastern United States where the mean annual precipitation is 42 in., Equations 7.16 and 7.17 yield

$$\overline{X} = 0.00264(100)^{1.01}(42)^{1.58} = 101 \text{ ft}^3\text{/sec} \tag{7.18}$$

and

$$S = 0.0142(100)^{0.99}(42)^{0.85} = 32.5 \text{ ft}^3\text{/sec.} \tag{7.19}$$

The log-normal flood-frequency curve is computed using the values shown in Table 2.2.

EXERCISES

7.5.1 Assume regression equations for computing the log mean ($\overline{Y}$) and log standard deviation (S_y) are

$$\overline{Y} = 0.33A^{0.72}$$

and

$$S_y = 0.18A^{0.37}S^{0.29},$$

where A is the drainage area (mi^2), and S is the channel slope (ft/ft). Develop a log-normal frequency curve for a 12.6 mi^2 watershed with a 1.4% slope.

7.5.2 Using the data shown in Table 7.3, develop an equation that relates the 2-yr discharge to the drainage area. Assume a power-model relationship. Use the relationship to estimate the 2-yr discharge for a 57.5 mi^2 watershed. Compare the coefficient with that for the 2-yr equation with the three predictors (see Table 7.4a).

7.6 RATIONAL METHOD

The methods outlined in the three previous sections are based on the analysis of stream-gage data. For small watersheds, especially those undergoing urban/suburban development, regional equations that are appropriate for assessing the impact of development on peak discharges are not available, with the possible exception of the USGS regression equations discussed in Section 7.3.3. However, these equations are not widely used because they do not include variables that typically are used to reflect changes in watershed conditions. Thus, methods that provide peak-discharge estimates using readily available input data, such as watershed and design-storm rainfall characteristics, are needed in design. The remainder of this chapter introduces a few of these methods.

TABLE 7.9 Runoff Coefficients for the Rational Formula versus Hydrologic Soil Group (A, B, C, D) and Slope Range

	A			B			C			D		
Land Use	0–2%	2–6%	6%+	0–2%	2–6%	6%+	0–2%	2–6%	6%+	0–2%	2–6%	6%+
Cultivated land	0.08[a]	0.13	0.16	0.11	0.15	0.21	0.14	0.19	0.26	0.18	0.23	0.31
	0.14[b]	0.18	0.22	0.16	0.21	0.28	0.20	0.25	0.34	0.24	0.29	0.41
Pasture	0.12	0.20	0.30	0.18	0.28	0.37	0.24	0.34	0.44	0.30	0.40	0.50
	0.15	0.25	0.37	0.23	0.34	0.45	0.30	0.42	0.52	0.37	0.50	0.62
Meadow	0.10	0.16	0.25	0.14	0.22	0.30	0.20	0.28	0.36	0.24	0.30	0.40
	0.14	0.22	0.30	0.20	0.28	0.37	0.26	0.35	0.44	0.30	0.40	0.50
Forest	0.05	0.08	0.11	0.08	0.11	0.14	0.10	0.13	0.16	0.12	0.16	0.20
	0.08	0.11	0.14	0.10	0.14	0.18	0.12	0.16	0.20	0.15	0.20	0.25
Residential lot size 1/8 acre	0.25	0.28	0.31	0.27	0.30	0.35	0.30	0.33	0.38	0.33	0.36	0.42
	0.33	0.37	0.40	0.35	0.39	0.44	0.38	0.42	0.49	0.41	0.45	0.54
Residential lot size 1/4 acre	0.22	0.26	0.29	0.24	0.29	0.33	0.27	0.31	0.36	0.30	0.34	0.40
	0.30	0.34	0.37	0.33	0.37	0.42	0.36	0.40	0.47	0.38	0.42	0.52
Residential lot size 1/3 acre	0.19	0.23	0.26	0.22	0.26	0.30	0.25	0.29	0.34	0.28	0.32	0.39
	0.28	0.32	0.35	0.30	0.35	0.39	0.33	0.38	0.45	0.36	0.40	0.50
Residential lot size 1/2 acre	0.16	0.20	0.24	0.19	0.23	0.28	0.22	0.27	0.32	0.26	0.30	0.37
	0.25	0.29	0.32	0.28	0.32	0.36	0.31	0.35	0.42	0.34	0.38	0.48
Residential lot size 1 acre	0.14	0.19	0.22	0.17	0.21	0.26	0.20	0.25	0.31	0.24	0.29	0.35
	0.22	0.26	0.29	0.24	0.28	0.34	0.28	0.32	0.40	0.31	0.35	0.46
Industrial	0.67	0.68	0.68	0.68	0.68	0.69	0.68	0.69	0.69	0.69	0.69	0.70
	0.85	0.85	0.86	0.85	0.86	0.86	0.86	0.86	0.87	0.86	0.86	0.88
Commercial	0.71	0.71	0.72	0.71	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72
	0.88	0.88	0.89	0.89	0.89	0.89	0.89	0.89	0.90	0.89	0.89	0.90
Streets	0.70	0.71	0.72	0.71	0.72	0.74	0.72	0.73	0.76	0.73	0.75	0.78
	0.76	0.77	0.79	0.80	0.82	0.84	0.84	0.85	0.89	0.89	0.91	0.95
Open space	0.05	0.10	0.14	0.08	0.13	0.19	0.12	0.17	0.24	0.16	0.21	0.28
	0.11	0.16	0.20	0.14	0.19	0.26	0.18	0.23	0.32	0.22	0.27	0.39
Parking	0.85	0.86	0.87	0.85	0.86	0.87	0.85	0.86	0.87	0.85	0.86	0.87
	0.95	0.96	0.97	0.95	0.96	0.97	0.95	0.96	0.97	0.95	0.96	0.97

[a] Runoff coefficients for storm-recurrence intervals less than 25 years

[b] Runoff coefficients for storm-recurrence intervals of 25 years or longer

7.6.1 Procedure. The most widely used uncalibrated equation is the Rational Method. Mathematically, the Rational Method relates the peak discharge (q_p, ft^3/sec) to the drainage area (A, acres), the rainfall intensity (i, in./hr), and the runoff coefficient (C):

$$q_p = CiA. \tag{7.20}$$

The rainfall intensity is obtained from an IDF curve (see Figure 4.4) using both the return period and a duration equal to the time of concentration as input. The value of the runoff coefficient is a function of the land use, cover condition, soil group, and watershed slope. Table 7.9 is an example of a table of C values. Table 7.9 suggests that the runoff coefficient varies with the soil group, the watershed slope, and the exceedence probability, as well as the landcover. The C values are shown for discrete categories for the continuous variables of exceedence probability and slope, which can lead to misapplication. For example, if a frequency curve is needed for return periods from the 2-yr to the 100-yr return period, then application of Table 7.9 would lead to a discontinuity at a return period of 25 years. For such cases, it is best to use a constant C or to let C vary smoothly across return periods. One possibility is to plot the value in Table 7.9 for low return periods at the 0.2 exceedence probability, plot the value for large return periods at the 0.02 exceedence probability, and then take the values for other return periods from the straight-line frequency curve drawn with these two points.

Table 7.10 is a commonly used summary of C values. A problem with tables such as Table 7.10 is that for each land use a range of values is provided, which can lead to inconsistency in application. As a general rule, the mean of the range should be used unless a different value can be fully justified. It would be improper for a low value to be selected to reduce the size and therefore the cost of the drainage system.

A primary use of the Rational Method has been for design problems for small urban areas such as the sizing of inlets and culverts, which are characterized by small drainage areas and short times of concentration. For such designs, short-duration storms are critical, which is why the time of concentration is used as the input duration for obtaining i from the IDF curve. If the storm duration occurs at a constant rate i and occurs uniformly over the entire watershed, the volume of rainfall would equal iAt_c, which would have units of acre-inches when t_c is expressed in hours. The runoff coefficient then becomes a scaling factor that converts the volume rate (i.e., iA in acre-in./hr) of rainfall to a peak discharge. A more detailed discussion of the conceptual basis of the Rational Method is given in Chapter 9.

Example 7.9

Consider the design problem where a peak discharge is required to size a storm-drain inlet for a 2.4-acre parking area in Baltimore, with a time of concentration of 0.1 hr and a slope of 1.5%. For a 25-yr design return period, the rainfall intensity (see Figure 4.4) is 8.6 in./hr, and the runoff coefficient (see Table 7.9) is 0.95. Therefore, the design discharge is

$$q_p = 0.95(8.6)(2.4) = 20 \text{ ft}^3\text{/sec}. \tag{7.21}$$

Some drainage policies provide for a minimum time of concentration, with 15 to 20 min often being specified. If the preceding design were for a project where the minimum t_c was 15 min, the design intensity would be 6.5 in./hr, and the peak discharge would be 15 ft^3/sec.

TABLE 7.10 Runoff Coefficients for the Rational Method

Description of Area	Range of Runoff Coefficients	Recommended Value*
Business		
Downtown	0.70–0.95	0.85
Neighborhood	0.50–0.70	0.60
Residential		
Single-family	0.30–0.50	0.40
Multiunits, detached	0.40–0.60	0.50
Multiunits, attached	0.60–0.75	0.70
Residential (suburban)	0.25–0.40	0.35
Apartment	0.50–0.70	0.60
Industrial		
Light	0.50–0.80	0.65
Heavy	0.60–0.90	0.75
Parks, cemeteries	0.10–0.25	0.20
Playgrounds	0.20–0.35	0.30
Railroad yard	0.20–0.35	0.30
Unimproved	0.10–0.30	0.20

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surface in the drainage area. This procedure often is applied to a typical sample block as a guide to the selection of reasonable values of the coefficient for an entire area. Coefficients with respect to surface type currently in use are listed below.

Character of Surface	Range of Runoff Coefficients	Recommended Value*
Pavement		
Asphaltic and concrete	0.70–0.95	0.85
Brick	0.75–0.85	0.80
Roofs	0.75–0.95	0.85
Lawns, sandy soil		
Flat, 2%	0.05–0.10	0.08
Average, 2 to 7%	0.10–0.15	0.13
Steep, 7%	0.15–0.20	0.18
Lawns, heavy soil		
Flat, 2%	0.13–0.17	0.15
Average, 2 to 7%	0.18–0.22	0.20
Steep, 7%	0.25–0.35	0.30

The coefficients in these two tabulations are applicable for storms of 5- to 10-year frequencies. Less frequent, higher intensity storms will require the use of higher coefficients because infiltration and other losses have a proportionally smaller effect on runoff. The coefficients are based on the assumption that the design storm does not occur when the ground surface is frozen.

* Recommended value not included in original source.

Source: American Society of Civil Engineers. *Design and Construction of Sanitary and Storm Sewers*. New York, 1969: 332.

7.6.2 Runoff Coefficients for Nonhomogeneous Areas. The runoff coefficients shown in Table 7.9 reflect the effects of land use, soil, and slope on runoff potential. The use of Equation 7.20 assumes that the watershed is homogeneous in these characteristics so that the runoff coefficient used provides unbiased estimates. Where a drainage area is characterized by distinct subareas that have different runoff potentials, the watershed should be subdivided, and the equation should be applied separately to each area; the procedure for this is discussed in Section 7.6.3. Where a watershed is not homogeneous, but is characterized by highly dispersed areas that can be characterized by different runoff coefficients, a weighted runoff coefficient should be determined. The weighting is based on the area of each land use and is found by the equation

$$C_w = \frac{\sum_{j=1}^{n} C_j A_j}{\sum_{j=i}^{n} A_j}, \tag{7.22}$$

in which A_j is the area for landcover j, C_j is the runoff coefficient for area j, n is the number of distinct landcovers within the watershed, and C_w is the weighted runoff coefficient. The weighted coefficient can be used with Equation 7.20. The denominator of Equation 7.22 equals the total drainage area, so Equation 7.22 can be substituted into Equation 7.20, which yields the following:

$$q_p = i \sum_{j=1}^{n} C_j A_j. \tag{7.23}$$

Example 7.10

Equation 7.23 will be illustrated using the data shown in Table 7.11. It is assumed that the different land uses are scattered throughout the watershed, and therefore it is impractical to subdivide the watershed. Equation 7.22 can be used to compute a weighted runoff coefficient:

$$\begin{aligned} C_w &= \frac{0.19(14.2) + 0.14(11.6) + 0.32(8.9) + 0.89(4.3) + 0.82(3.9)}{42.9} \\ &= 0.33. \end{aligned} \tag{7.24}$$

TABLE 7.11 Example: Calculation of Weighted Runoff Coefficients

Land Use	C_i	A_i (acres)
Open space	0.19	14.2
Forest	0.14	11.6
Residential (1/2 acre)	0.32	8.9
Light commercial	0.89	4.3
Streets	0.82	3.9
		42.9

For a 25-yr rainfall intensity of 3.6 in./hr, the peak discharge would be

$$q_p = 0.33(3.6)(42.6) = 51 \text{ ft}^3/\text{sec}. \tag{7.25}$$

Equation 7.23 will provide the same estimate of q_p.

7.6.3 Designs on Subdivided Watersheds. The discussion to this point concerning the Rational Method has used the method only to compute the peak discharge for a contributing area. The method can also be used for nonhomogeneous watersheds in which the watershed is divided into homogeneous subareas and where multiple inlets and pipe systems are involved. Where a watershed has distinct areas of nonhomogeneity, every attempt should be made to subdivide the watershed into homogeneous subareas and then use the Rational Method for each subarea or group of subareas.

A number of methods have been proposed for solving such problems. One method will be described here and examples provided for illustration. A second method is given in Chapter 9.

The method described here is an attempt to provide an equal level of protection to each structural element of the total drainage area. It is based on the following two rules for using Equation 7.20:

1. For each inlet area at the headwater of a drainage area, the Rational Method (Equation 7.20) is used to compute the peak discharge.
2. For locations where drainage is arriving from two or more inlet areas, the longest time of concentration is used to find the design intensity, a weighted runoff coefficient is computed, and the total drainage area to that point is used with Equation 7.20.

It is important to emphasize that Equation 7.20 is not used to compute the discharge from each inlet area and the discharges summed, since the differences in timing of runoff that exist for the different subareas would be ignored. The procedure behind these two steps will be illustrated with two examples.

Example 7.11

Figure 7.1 shows the schematic of a drainage area that has been divided into three subareas, with the characteristics of each shown. Beginning with the upstream subarea, the discharge into inlet 1 can be determined. Since the time of concentration is less than 15 min, a duration of 15 min will be used to obtain the rainfall intensity from Figure 4.4. The intensity for a 10-yr event is 5.4 in./hr; therefore, the peak discharge into inlet 1 is

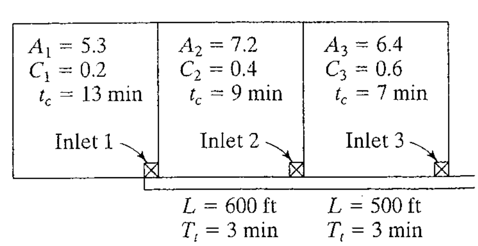

Figure 7.1 Schematic diagram of a drainage area.

$$q_{p1} = 0.2(5.4 \text{ in./hr})(5.3 \text{ acres}) = 5.7 \text{ ft}^3/\text{sec.} \tag{7.26}$$

The runoff into inlet 1 flows through a pipe, which is 600 ft in length and has a travel time of 3 min. The peak discharge to the inlet from subarea 2 can also be computed from the Rational Formula:

$$q_{p2} = 0.4(5.4 \text{ in./hr})(7.2 \text{ acres}) = 15.6 \text{ ft}^3/\text{sec.} \tag{7.27}$$

However, the pipe between inlets 2 and 3 should not necessarily be designed to carry the sum of these subarea peak discharges (i.e., 21.3 ft^3/sec). Subareas 1 and 2 have different times of concentration, and the flow from subarea 1 must travel through the 600 ft of pipe before arriving at inlet 2. Therefore, the pipe between inlets 2 and 3 will not be subjected to the sum of the two. Instead, it is common practice to recompute the discharge for the total area using a weighted runoff coefficient and a rainfall intensity based on the longest time of concentration. For the drainage area shown in Figure 7.1, the weighted runoff coefficient for subareas 1 and 2 is

$$C_w = \frac{0.2(5.3) + 0.4(7.2)}{5.3 + 7.2} = 0.315. \tag{7.28}$$

The longest time of concentration for the two subareas would be the sum of the drainage time from subarea 1 and the travel time in the pipe between inlets 1 and 2, which is $13 + 3 = 16$ min. From Figure 4.4 the 10-yr intensity is 5.3 in./hr, which yields a peak discharge of

$$q_p = 0.315(5.3)(5.3 + 7.2) = 20.9 \text{ ft}^3/\text{sec.} \tag{7.29}$$

The discharge of Equation 7.29 should be used to size the pipe from inlet 2 to inlet 3.

The data for subarea 3 could be used to size the inlet for that subarea:

$$q_{p3} = 0.6(5.4 \text{ in./hr})(6.4 \text{ acres}) = 20.7 \text{ ft}^3/\text{sec.} \tag{7.30}$$

However, the size of the pipe draining the three subareas can be determined using a discharge estimate obtained using a weighted runoff coefficient, which is

$$C_w = \frac{0.2(5.3) + 0.4(7.2) + 0.6(6.4)}{5.3 + 7.2 + 6.4} = 0.412. \tag{7.31}$$

The longest time of concentration is 19 min, which includes 6 min of travel time in the pipe from inlet 1 to inlet 3. From Figure 4.4, a rainfall intensity of 4.7 in./hr is obtained and used to compute the peak discharge for the entire 18.9 acres:

$$q_p = 0.412(4.8 \text{ in./hr})(18.9 \text{ acres}) = 37.4 \text{ ft}^3/\text{sec.} \tag{7.32}$$

While the sum of the discharges from the individual subareas is greater than the discharge computed in Equation 7.32 (41.8 ft^3/sec versus 37.4 ft^3/sec), the value computed using the approach with the weighted runoff coefficient and the longest time of concentration is an accepted method. It is believed that this approach provides the same level of protection with respect to flood risk at each design point. That is, the inlets and pipe segments would each be designed to pass the flood runoff for the same exceedence frequency, which was 10 years in the example shown in Figure 7.1. This assumption is probably reasonable, although it may not be entirely accurate. A method based on hydrographs, presented in Chapter 9, should be more accurate for the given assumptions.

Example 7.12

Using another hypothetical case, Figure 7.2 shows a 43.8-acre drainage area that includes approximately 11 acres of commercial property, with the remainder in $\frac{1}{4}$-acre and $\frac{1}{8}$-acre parcels with residential land use. The existing slope and grading during development have resulted in an area in which all drainage is contained between Glass Boulevard and Polonsky Alley. The upper end of the drainage area is approximately 800 ft north of 3rd Street. The location of the proposed manholes and storm-drainage system is shown in Figure 7.2. The outlet will be a single pipe that will run under Main Street at the intersection of Glass Boulevard. The drainage policy requires design on a 10-yr exceedence frequency.

The computations for the peak discharges are given in Table 7.12. The computations begin for flow into manhole M_{54}, which is at the intersection of 3rd Street and Polonsky Alley. Calculations are provided for the design discharges of both the inlets and the pipes. For some locations, there may be more than one inlet at the intersection, and so the inlet discharges would have to be divided accordingly. Computations proceed down Polonsky Alley to Main Street. At each manhole, a weighted runoff coefficient is computed, with the weights depending on the drainage area in the different land uses. For example, the weighted runoff coefficient for manhole M_{24} is computed by

$$C_w = \frac{1.8(0.31) + 1.1(0.72) + 1.1(0.72) + 1.6(0.72)}{1.8 + 1.1 + 1.1 + 1.6} = 0.58. \tag{7.33}$$

Since all of the inlet times of concentration were less than 15 min, a minimum of 15 min was used. The total times of concentration for the downstream manholes are the sum of the inlet t_c for the upper subarea and the travel times through the pipe systems.

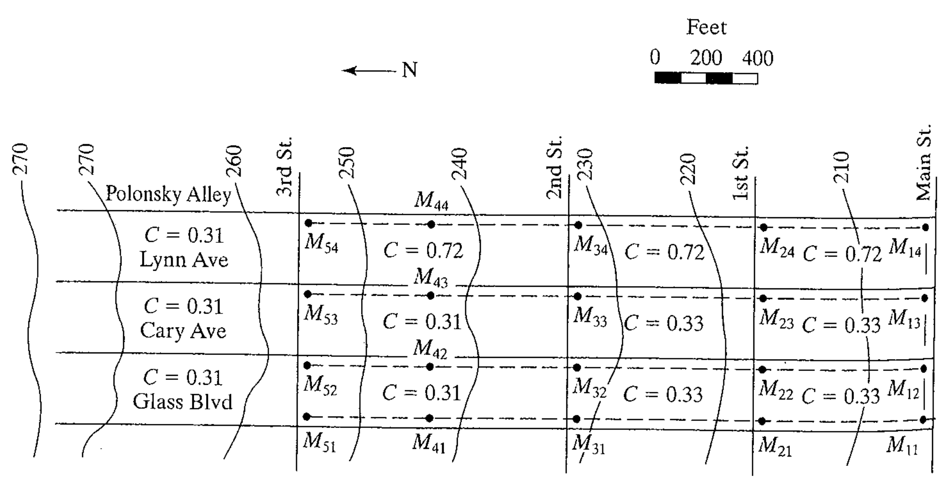

Figure 7.2 Watershed layout for Example 7.12.

TABLE 7.12 Peak-Discharge Computations for the Drainage System Shown in Figure 7.2

	Area (acres)		Runoff Coefficient		Inlet	Travel	Total	Intensity (in./hr)		Peak, q_p (ft^3/sec)	
Manhole	Inlet	Total	Inlet	Weighted	T_c (min)	Time(min)	T_c (min)	Inlet	Total	Inlet	Total
M_{54}	1.8		0.31		15			5.4		3.0	
M_{44}	1.1	2.9	0.72	0.47	15	3	18	5.4	5.0	4.3	6.9
M_{34}	1.1	4.0	0.72	0.54	15	3	21	5.4	4.6	4.3	10.0
M_{24}	1.6	5.6	0.72	0.59	15	4	25	5.4	4.1	6.2	13.6
M_{14}	1.6	7.2	0.72	0.62	15	4	29	5.4	3.9	6.2	17.5
M_{53}	3.7		0.31		15			5.4		6.2	
M_{43}	2.3	6.0	0.31	0.31	15	3	18	5.4	5.0	3.8	9.3
M_{33}	2.3	8.3	0.31	0.31	15	3	21	5.4	4.6	3.8	11.8
M_{23}	3.2	11.5	0.33	0.32	15	4	25	5.4	4.1	5.7	14.9
M_{13}	3.2	21.9	0.33	0.42	15	1	30	5.4	3.8	5.7	34.8
M_{52}	3.7		0.31		15			5.4		6.2	
M_{42}	2.3	6.0	0.31	0.31	15	3	18	5.4	5.0	3.8	9.3
M_{32}	2.3	8.3	0.31	0.31	15	3	21	5.4	4.6	3.8	11.8
M_{22}	3.2	11.5	0.33	0.32	15	4	25	5.4	4.1	5.7	14.9
M_{12}	3.2	36.6	0.33	0.38	15	1	31	5.4	3.6	5.7	49.9
M_{51}	1.8		0.31		15			5.4		3.0	
M_{41}	1.1	2.9	0.31	0.31	15	3	18	5.4	5.0	1.8	4.5
M_{31}	1.1	4.0	0.31	0.31	15	3	21	5.4	4.6	1.8	5.7
M_{21}	1.6	5.6	0.33	0.32	15	4	25	5.4	4.1	2.9	7.2
M_{11}	1.6	43.8	0.33	0.37	15	1	32	5.4	3.5	2.9	56.5

Independent calculations can be made for the pipes located on the four north-south roadways. Rainfall intensities are obtained from Figure 4.4 for the 10-yr return period and for a duration equal to the time of concentration.

The discharges from manholes M_{13}, M_{12}, and M_{11} must be determined using summation of flows from the four feeder lines. The area into manhole M_{13} consists of the 7.2 acres along Polonsky Alley and the 14.7 acres draining into the inlets along Lynn Avenue. The area into manhole M_{12} consists of the 21.9 acres draining into manholes M_{14} and M_{13} and the 14.7 acres draining into inlets along Cary Avenue. The total area draining into a manhole, the weighted runoff coefficient, and an intensity based on a duration equal to the longest time of concentration/travel time combination are used to compute the discharge.

The drainage system shown in Figure 7.2 has a peak discharge from manhole M_{11} of 56.5 ft^3/sec, which is approximately 1.3 ft^3/sec/acre. The sizing of pipes is discussed in Chapter 8.

EXERCISES

7.6.1 A 22-acre watershed in B soil on moderate slopes is planned for commercial development, for which the time of concentration is estimated to be 15 min. Using Figure 4.4, show the peak-discharge frequency curve that allows only the intensity to vary with frequency.

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