EXAMPLE 5.44

Consider the two inlets and two pipes shown in Figure 5.46. Catchment A has an area of 1 ha and is 50% impervious; catchment B has an area of 2 ha and is 10% impervious.

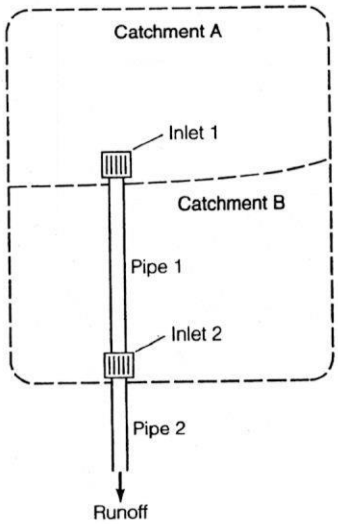

FIGURE 5.46: Computation of peak inlet and pipe flows
Adapted from ASCE (1992)

TABLE 5.34: Catchment Characteristics

Catchment	Surface	C	L (m)	n	S_0
A	Pervious	0.2	80	0.2	0.01
	Impervious	0.9	60	0.1	0.01
B	Pervious	0.2	140	0.2	0.01
	Impervious	0.9	65	0.1	0.01

All impervious areas are directly connected to the sewer inlets. The runoff coefficient C; length of overland flow, L; roughness coefficient, n; and average slope, S_0, of the pervious and impervious surfaces in both catchments are given in Table 5.34. The design storm has a return period of 10 years, and the 10-year IDF curve can be approximated by

$$i = \frac{7620}{t + 36}$$

where i is the average rainfall intensity in mm/h and t is the duration of the storm in minutes. Calculate the peak flows to be handled by the inlets and pipes.

Solution Using the given IDF curve, the effective rainfall rate, i_e, is given by the rational formula as

$$i_e = Ci = C\frac{7620}{t_c + 36} \tag{5.231}$$

where C is the runoff coefficient. The duration, t, of the design storm is taken to be equal to the time of concentration, t_c, given by Equation 5.94 as

$$t_c = 6.99\frac{(nL)^{0.6}}{i_e^{0.4}S_0^{0.3}} \tag{5.232}$$

Simultaneous solution of Equations 5.231 and 5.232 using the catchment characteristics in Table 5.34 leads to the following times of concentration, t_c:

Catchment	Surface	t_c (min)
A	Pervious area	46
	Impervious area	11
B	Pervious	71
	Impervious	12

Consider now the flows at specific locations.

Inlet 1 and Pipe 1: When the entire catchment A is contributing, the time of concentration is 46 min (this is the time for both pervious and impervious areas to be fully contributing), the average rainfall rate, i, from the IDF curve is 92.9 mm/h ($= 2.58 \times 10^{-5}$ m/s), and the weighted-average runoff coefficient, $\overline{C}$, is given by

$$\overline{C} = 0.5(0.9) + 0.5(0.2) = 0.55$$

Since the area of the catchment is 1 ha ($= 10{,}000$ m^2), the peak runoff rate, Q_p, from the catchment is given by the rational formula as

$$Q_p = \overline{C}iA = (0.55)(2.58 \times 10^{-5})(10{,}000) = 0.142 \text{ m}^3/\text{s}$$

Considering only the impervious portion of the catchment, the time of concentration is 11 min, the average rainfall rate, i, from the IDF curve is 162 mm/h ($= 4.50 \times 10^{-5}$ m/s), the runoff coefficient, C, is 0.9, the contributing area is 0.5 ha ($= 5000$ m^2), and the peak runoff rate, Q_p, is given by

$$Q_p = CiA = (0.9)(4.50 \times 10^{-5})(5000) = 0.203 \text{ m}^3/\text{s}$$

The calculated peak runoff from the directly connected impervious area is greater than the calculated runoff from the entire area, and therefore the design flow to be handled by Inlet 1 and Pipe 1 is controlled by the directly connected impervious area and is equal to 0.203 m^3/s.

Inlet 2: When the entire catchment B is contributing, the time of concentration is 71 min, the average rainfall rate, i, from the IDF curve is 71.2 mm/h ($= 1.98 \times 10^{-5}$ m/s), and the weighted average runoff coefficient, $\overline{C}$, is given by

$$\overline{C} = 0.1(0.9) + 0.9(0.2) = 0.27$$

Since the area of the catchment is 2 ha ($= 20{,}000$ m^2), then the peak runoff rate, Q_p, from the catchment is given by the rational formula as

$$Q_p = \overline{C}iA = (0.27)(1.98 \times 10^{-5})(20{,}000) = 0.107 \text{ m}^3/\text{s}$$

Considering only the impervious portion of the catchment, the time of concentration is 12 min, the average rainfall rate, i, from the IDF curve is 159 mm/h ($= 4.41 \times 10^{-5}$ m/s), the runoff coefficient, C, is 0.9, the contributing area is 0.2 ha ($= 2000$ m^2), and the peak runoff rate, Q_p, is given by

$$Q_p = CiA = (0.9)(4.41 \times 10^{-5})(2000) = 0.079 \text{ m}^3/\text{s}$$

The calculated peak runoff from the directly connected impervious area is less than the calculated runoff from the entire area, and therefore the design flow to be handled by Inlet 2 is controlled by the entire catchment and is equal to 0.107 m^3/s.

Pipe 2: First consider the case where the entire tributary area of 3 ha ($= 30{,}000$ m^2) is contributing runoff to pipe 2. The time of concentration of catchment A is equal to 46 min plus the time of flow in pipe 1, which, in lieu of hydraulic calculations, can be taken as 2 min. Therefore, the time of concentration of catchment A is 48 min. The time of concentration of catchment B is 71 min, and therefore the time of concentration of the entire tributary area to pipe 2 (including both catchments A and B) is equal to 71 min. The average rainfall intensity corresponding to this duration (from the IDF curve) is 71.2 mm/h ($= 1.98 \times 10^{-5}$ m/s); the area-weighted runoff coefficient, $\overline{C}$, is given by

$$\overline{C} = \frac{1}{3}[(0.5 + 0.2)(0.9) + (0.5 + 1.8)(0.2)] = 0.36$$

and the rational formula gives the peak runoff rate, Q_p, as

$$Q_p = \overline{C}iA = (0.36)(1.98 \times 10^{-5})(30{,}000) = 0.214 \text{ m}^3/\text{s}$$

Considering only the impervious portions of catchments A and B, the contributing area is 0.7 ha ($= 7000$ m^2), the time of concentration is 13 min (equal to the time of concentration for Inlet 1 plus travel time of 2 min in pipe), the corresponding average rainfall intensity from the IDF curve is 156 mm/h ($= 4.32 \times 10^{-5}$), the runoff coefficient is 0.9, and the rational formula gives a peak runoff, Q_p, of

$$Q_p = CiA = (0.9)(4.32 \times 10^{-5})(7000) = 0.272 \text{ m}^3/\text{s}$$

Therefore, the peak runoff rate calculated by using the entire catchment is less than the peak runoff rate calculated by considering only the directly connected impervious portion of the tributary area. The design flow to be handled by pipe 2 is therefore controlled by the directly connected impervious area and is equal to 0.272 m^3/s.

Most storm sewers are sized to flow full at the design discharge, although where ground elevations are sufficient, a limited surcharge above the pipe crown may be permitted (ASCE, 1992). To prevent deposition of suspended materials, the minimum slope of the sewer should produce a velocity of at least 60 to 90 cm/s (2 to 3 ft/s) when the sewer is flowing full; to prevent scouring, the velocity should be less than 3 to 4.5 m/s (10 to 15 ft/s). The appropriate flow equation for sizing storm sewers is the Darcy–Weisbach equation, but it is also common practice to use the Manning equation. Caution should be exercised when using the Manning equation, which is valid only for hydraulically rough (fully turbulent) flow and is appropriate only when the following condition is satisfied (French, 1985):

$$n^6\sqrt{RS_0} \geq 9.6 \times 10^{-14} \tag{5.233}$$

where n is the Manning roughness coefficient, R is the hydraulic radius of the pipe (in meters), and S_0 is the slope of the pipe. Manning roughness coefficients recommended for closed-conduit flow are given in Table 5.35. In cases where the Darcy–Weisbach equation is used, the pipe roughness is commonly assumed to be independent of the pipe material (due to the accumulation of slime) and a value of 0.6 mm is typically

TABLE 5.35: Manning Coefficient in Closed Conduits

Material	n
Asbestos-cement pipe	0.011–0.015
Brick	0.013–0.017
Cast-iron pipe (cement lined and seal coated)	0.011–0.015
Concrete (monolithic):	
Smooth forms	0.012–0.014
Rough forms	0.015–0.017
Concrete pipe	0.011–0.015
Corrugated metal pipe (1.3-cm × 6.4-cm corrugations):	
Plain	0.022–0.026
Paved invert	0.018–0.022
Spun asphalt lined	0.011–0.015
Plastic pipe (smooth)	0.011–0.015
Vitrified clay:	
Pipes	0.011–0.015
Liner plates	0.013–0.017

Source: ASCE (1982).