Chapter 11

Hydrologic Design and Floodplain Analysis

11.1 HYDROLOGIC DESIGN FOR STORMWATER MANAGEMENT: STORM SEWERS DESIGN

Stormwater management is knowledge used to understand, control, and utilize waters in their different forms within the hydrologic cycle (Wanielista and Yousef, 1993). The goal of this chapter is to provide an introduction to the various concepts and design procedures involved in stormwater management. The overall key component of stormwater management is the drainage system. Urbonas and Roesner (1993) point out the following vital functions of a drainage system:

1. It removes stormwater from the streets and permits the transportation arteries to function during bad weather; when this is done efficiently, the life expectancy of street pavement is extended.
2. The drainage system controls the rate and velocity of runoff along gutters and other surfaces in a manner that reduces the hazard to local residents and the potential for damage to pavement.
3. The drainage system conveys runoff to natural or man-made major drainage ways.
4. The system can be designed to control the mass of pollutants arriving at receiving waters.
5. Major open drainage ways and detention facilities offer opportunities for multiple use such as recreation, parks, and wildlife preserves.

Storm drainage criteria are the foundation for developing stormwater control. These criteria should set limits on development, provide guidance and methods of design, provide details of key components of drainage and flood control systems, and ensure longevity, safety, aesthetics, and maintainability of the system served (Urbonas and Roesner, 1993).

11.1.1 Rational Method Design

From an engineering viewpoint the design can be divided into two main aspects: runoff prediction and pipe sizing. The rational method, which can be traced back to the mid-nineteenth century, is still probably the most popular method used for the design of storm sewers (Yen and Akan, 1999). Although criticisms have been raised of its adequacy, and several other, more advanced, methods have been proposed, the rational method, because of its simplicity, is still in continued use for sewer design when high accuracy of runoff rate is not essential.

Table 11.1.1 Runoff Coefficients for Use in the Rational Method

	Return period (years)						
Character of surface	2	5	10	25	50	100	500
Developed							
Asphaltic	0.73	0.77	0.81	0.86	0.90	0.95	1.00
Concrete/roof	0.75	0.80	0.83	0.88	0.92	0.97	1.00
Grass areas (lawns, parks, etc.)							
Poor condition (grass cover less than 50% of the area)							
Flat, 0–2%	0.32	0.34	0.37	0.40	0.44	0.47	0.58
Average, 2–7%	0.37	0.40	0.43	0.46	0.49	0.53	0.61
Steep, over 7%	0.40	0.43	0.45	0.49	0.52	0.55	0.62
Fair condition (grass cover 50% to 75% of the area)							
Flat, 0–2%	0.25	0.28	0.30	0.34	0.37	0.41	0.53
Average, 2–7%	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Steep, over 7%	0.37	0.40	0.42	0.46	0.49	0.53	0.60
Good condition (grass cover larger than 75% of the area)							
Flat, 0–2%	0.21	0.23	0.25	0.29	0.32	0.36	0.49
Average, 2–7%	0.29	0.32	0.35	0.39	0.42	0.46	0.56
Steep, over 7%	0.34	0.37	0.40	0.44	0.47	0.51	0.58
Undeveloped							
Cultivated land							
Flat, 0–2%	0.31	0.34	0.36	0.40	0.43	0.47	0.57
Average, 2–7%	0.35	0.38	0.41	0.44	0.48	0.51	0.60
Steep, over 7%	0.39	0.42	0.44	0.48	0.51	0.54	0.61
Pasture/range							
Flat, 0–2%	0.25	0.28	0.30	0.34	0.37	0.41	0.53
Average, 2–7%	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Steep, over 7%	0.37	0.40	0.42	0.46	0.49	0.53	0.60
Forest/woodlands							
Flat, 0–2%	0.20	0.25	0.28	0.31	0.35	0.39	0.48
Average, 2–7%	0.31	0.34	0.36	0.40	0.43	0.47	0.56
Steep, over 7%	0.35	0.39	0.41	0.45	0.48	0.52	0.58

Note: The values in the table are the standards used by the City of Austin, Texas.
Source: Chow, Maidment, and Mays (1988).

Limitations
Robertson et al. (1998) < 1 mi²
Wanielista et al. (1997) a) < (0.077–0.156 mi²) or (50 to 100 acres)
b) t_c < 20 min
Note that 1.008 ac-in/hr = 1 ft³/s

Using the rational method, the storm runoff peak is estimated by the rational formula

$$Q = KCiA \tag{11.1.1}$$

where the peak runoff rate Q is in ft^3/s-(m^3/s), K is 1.0 in U.S. customary units (0.28 for SI units), C is the runoff coefficient (Table 11.1.1), i is the average rainfall intensity in in/hr (mm/hr) from intensity-duration-frequency relationships for a specific return period and duration t_c in min, and A is the area of the tributary drainage area in ac (km^2). The duration is taken as the time of concentration t_c of the drainage area.

In urban areas, the drainage area usually consists of subareas or subcatchments of substantially different surface characteristics. As a result, a composite analysis is required that must take into account the various surface characteristics. The areas of the subcatchments are denoted by A_j and the runoff coefficients for each subcatchment are denoted by C_j. Then the peak runoff is computed using the following form of the rational formula:

$$Q = Ki\sum_{j=1}^{m} C_jA_j \tag{11.1.2}$$

where m is the number of subcatchments drained by a sewer.

The *rainfall intensity* i is the average rainfall rate considered for a particular drainage basin or subbasin. The intensity is selected on the basis of design rainfall duration and design frequency of occurrence. The design duration is equal to the time of concentration for the drainage area under consideration. The frequency of occurrence is a statistical variable that is established by design standards or chosen by the engineer as a design parameter.

The *time of concentration* t_c used in the rational method is the time associated with the peak runoff from the watershed to the point of interest. Runoff from a watershed usually reaches a peak at the time when the entire watershed is contributing; in this case, the time of concentration is the time for a drop of water to flow from the remotest point in the watershed to the point of interest. Runoff may reach a peak prior to the time the entire watershed is contributing. A trial-and-error procedure can be used to determine the critical time of concentration. The time of concentration to any point in a storm drainage system is the sum of the inlet time t_0 and the flow time t_f in the upstream sewers connected to the catchment, that is,

$$t_c = t_0 + t_f \tag{11.1.3}$$

where the flow time is

$$t_f = \sum \frac{L_j}{V_j} \tag{11.1.4}$$

where L_j is the length of the jth pipe along the flow path in ft (m) and V_j is the average flow velocity in the pipe in ft/s (m/s). The inlet time t_0 is the longest time of overland flow of water in a catchment to reach the storm sewer inlet draining the catchment.

In the rational method each sewer is designed individually and independently (except for the computation of sewer flow time) and the corresponding rainfall intensity i is computed repeatedly for the area drained by the sewer. For a given sewer, all the different areas drained by this sewer have the same i. Thus, as the design progresses towards the downstream sewers, the drainage area increases and usually the time of concentration increases accordingly. This increasing t_c in turn gives a decreasing i that should be applied to the entire area drained by the sewer.

Inlet times, or times of concentration for the case of no upstream sewers, can be computed using a number of methods, some of which are presented in Table 11.1.2. The longest time of concentration among the times for the various flow routes in the drainage area is the critical time of concentration used.

EXAMPLE 11.1.1

The computational procedure in the rational method is illustrated through an example design of sewers to drain a 20-ac area along Goodwin Avenue in Urbana, Illinois, as shown in Figure 11.1.1. The physical characteristics of the drainage basin are given in Table 11.1.3. (The catchments are identified by the manholes they drain directly into. The sewer pipes are identified by the number of the upstream manhole of each pipe. The Manning's roughness factor n is 0.014 for all the sewers in the example (adapted from Yen (1978)).

SOLUTION

Table 11.1.4 shows the computations for the design of 12 sewer pipes, namely, all the pipes upstream of sewer 6.1. The rainfall intensity-duration relationship is developed using National Weather Service report HYDRO-35 (see Chapter 7 or Frederick et al. (1977)) and plotted in Figure 11.1.2 for the design return period of two years. The entries in Table 11.1.4 are explained as follows:

Columns (1), (2), and (3): The sewer number and its length and slope are predetermined quantities.

Column 4: Total area drained by a sewer is equal to the sum of the areas of the subcatchments drained by the sewer, e.g., for sewer 3.1, the area 8.45 ac is equal to the area drained by sewer 2.1

Table 11.1.2 Summary of Time of Concentration Formulas

Method and date	Formula for t_c (min)	Remarks
Kirpich (1940)	$t_c = 0.0078L^{0.77}S^{-0.385}$ L = length of channel/ditch from headwater to outlet, ft S = average watershed slope, ft/ft	Developed from SCS data for seven rural basins in Tennessee with well-defined channel and steep slopes (3% to 10%); for overland flow on concrete or asphalt surfaces multiply t_c by 0.4; for concrete channels multiply by 0.2; no adjustments for overland flow on bare soil or flow in roadside ditches.
California Culverts Practice (1942)	$t_c = 60(11.9L^3/H)^{0.385}$ L = length of longest watercourse, mi H = elevation difference between divide and outlet, ft	Essentially the Kirpich formula; developed from small mountainous basins in California (U.S. Bureau of Reclamation, 1973 and 1987).
Izzard (1946)	$t_c = \dfrac{41.025(0.0007i + c)L^{0.33}}{S^{0.333}i^{0.667}}$ i = rainfall intensity, in/h c = retardance coefficient L = length of flow path, ft S = slope of flow path, ft/ft	Developed in laboratory experiments by Bureau of Public Roads for overland flow on roadway and turf surfaces; values of the retardance coefficient range from 0.0070 for very smooth pavement to 0.012 for concrete pavement to 0.06 for dense turf; solution requires iteration; product i times L should be < 500.
Federal Aviation Administration (1970)	$t_c = 1.8(1.1 - C)L^{0.50}/S^{0.333}$ C = rational method runoff coefficient L = length of overland flow, ft S = surface slope, %	Developed from airfield drainage data assembled by the Corps of Engineers; method is intended for use on airfield drainage problems, but has been used frequently for overland flow in urban basins.
Kinematic wave formulas (Morgali and Linsley, 1965; Aron and Erborge, 1973)	$t_c = \dfrac{0.94L^{0.6}n^{0.6}}{(i^{0.4}S^{0.3})}$ L = length of overland flow, ft n = Manning roughness coefficient i = rainfall intensity, in/h S = average overland slope, ft/ft	Overland flow equation developed from kinematic wave analysis of surface runoff from developed surfaces; method requires iteration since both i (rainfall intensity) and t_c are unknown; superposition of intensity-duration-frequency curve gives direct graphical solution for t_c.
SCS lag equation (U.S. Soil Conservation Service, 1975)	$t_c = \dfrac{100L^{0.8}[(1000/\mathrm{CN}) - 9]^{0.7}}{1900S^{0.5}}$ L = hydraulic length of watershed (longest flow path), ft CN = SCS runoff curve number S = average watershed slope, %	Equation developed by SCS from agricultural watershed data; it has been adapted to small urban basins under 2000 ac; found generally good where area is completely paved; for mixed areas it tends to overestimate; adjustment factors are applied to correct for channel improvement and impervious area; the equation assumes that $t_c = 1.67 \times$ basin lag.
SCS average velocity charts (U.S. Soil Conservation Service, 1975 and 1986)	$t_c = \dfrac{1}{60}\sum \dfrac{L}{V}$ L = length of flow path, ft V = average velocity in ft/s for various surfaces found using Figure 8.8.2	Overland flow charts in U.S. Soil Conservation Service (1986) show average velocity as function of watercourse slope and surface cover.

Source: Kibler (1982).

(7.30 ac in column 4) plus the area drained by sewer 2.2 (0.45 ac) plus the incremental area given in column (6) (0.70 ac for subcatchment 3.1).

Column (5) The identification number of the incremental subcatchments that drain directly through manhole or junction into the sewer being considered.

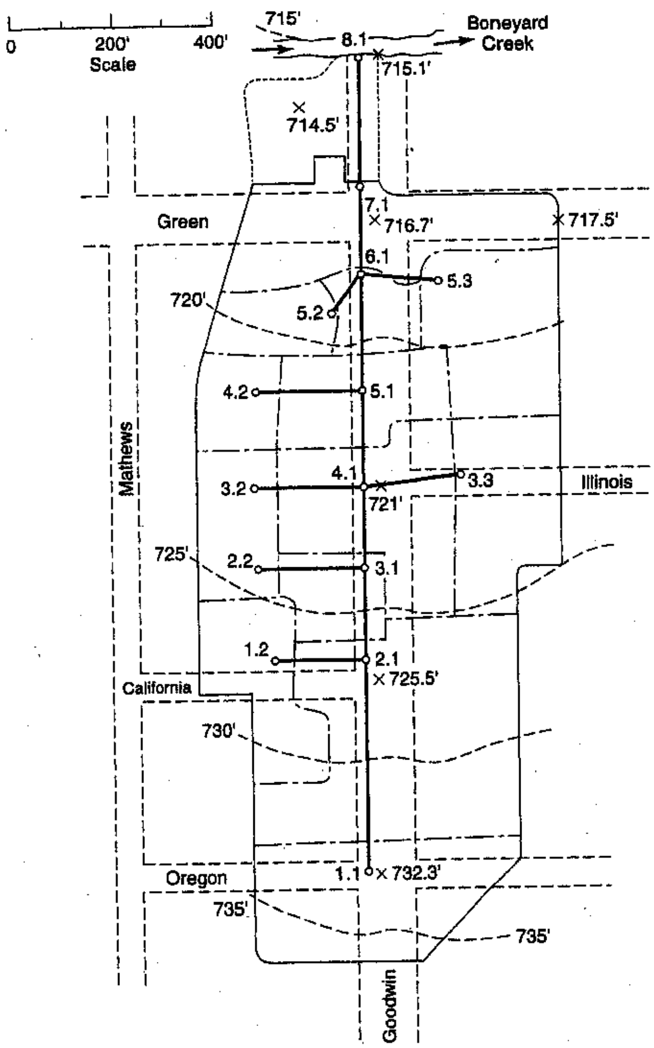

Figure 11.1.1 Goodwin Avenue drainage basin at Urbana, Illinois (from Yen 1978).

Column (6) Size of the incremental subcatchment identified in column 5 (Table 11.1.4).

Column (7) Value of runoff coefficient for each subcatchment (Table 11.1.4).

Column (8) Product of C and the corresponding subcatchment area.

Column (9) Summation of CA for all the areas drained by the sewer, which is equal to the sum of contributing values in column (9) and the values in column (8) for that sewer, e. g., for sewer 3.1, $5.97 = 5.12 + 0.36 + (0.49)$.

Column (10) Values of inlet time (Table 11.1.4) for the subcatchment drained (computed using methods in Table 11.1.2), i.e., the overland flow inlet time if the upstream subcatchment is no more than one sewer away from the sewer being designed (e.g., in designing sewer 3.1, 5.2 min for subcatchment 2.2 and 8.7 min for subcatchment 3.1); otherwise it is the total flow time to the entrance of the immediate upstream sewer (e.g., in designing sewer 3.1, 13.7 min for sewer 2.1).

Table 11.1.3 Characteristics of Catchments of Goodwin Avenue Drainage Basin

(1) Catchment	(2) Ground elevation at manhole (ft)	(3) Area A (ac)	(4) Runoff coefficient C	(5) Inlet time t_0 (min)
1.1	731.08	2.20	0.65	11.0
1.2	725.48	1.20	0.80	9.2
2.1	724.27	3.90	0.70	13.7
2.2	723.10	0.45	0.80	5.2
3.1	722.48	0.70	0.70	8.7
3.2	723.45	0.60	0.85	5.9
3.3	721.89	1.70	0.65	11.8
4.1	720.86	2.00	0.75	9.5
4.2	719.85	0.65	0.85	6.2
5.1	721.19	1.25	0.70	10.3
5.2	719.10	0.70	0.65	11.8
5.3	722.00	1.70	0.55	17.6
6.1	718.14	0.60	0.75	7.3
7.1	715.39	2.30	0.70	14.5

Source: Yen (1978).

Column (11) The sewer flow time of the immediate upstream sewer as given in column (19).

Column (12) The time of concentration t_c for each of the possible critical flow paths; t_c = inlet time (column (10)) + sewer flow time (column (11)) for each flow path.

Column (13) The rainfall duration t_d is assumed equal to the longest of the different times of concentration of different flow paths to arrive at the entrance of the sewer being considered; e.g., for sewer 3.1, t_d is equal to 14.1 min for sewer 2.1, which is longer than from sewer 2.2 (6.2 min) or directly from subcatchment 3.1 (8.7 min).

Column (14) The rainfall intensity i for the duration given in column (13) is based on HYDRO-35 for the two-year design return period (see Figure 11.1.2).

Column (15) Design discharge is computed by using Equation (11.1.2), i.e., the product of columns (9) and (14).

Column (16) Required sewer diameter in ft, as computed using Manning's formula, Equation (11.1.7), with $n = 0.014$, Q is given in column (15) and S_0 in column (3).

Column (17) The nearest commercial nominal pipe size that is not smaller than the computed size is adopted.

Column (18) Flow velocity computed by using $V = 4Q_p/(\pi D^2)$, i.e., column (15) multiplied by $4/\pi$ and divided by the square of column (17).

Column (19) Sewer flow time is computed as equal to L/V, i.e., column (2) divided by column (18) and converted into min.

This example demonstrates that in the rational method each sewer is designed individually and independently (except for the computation of sewer flow time) and the corresponding rainfall intensity i is computed repeatedly for the area drained by the sewer. For a given sewer, all the different areas drained by this sewer have the same i. Thus, as the design progresses towards downstream sewers, the drainage area increases and usually the time of concentration increases accordingly. This increasing t_c in turn gives a decreasing i, which should be applied to the entire area drained by the sewer. Failure to realize this variation of i is the most common mistake made in using the rational method for sewer design.

Table 11.1.4 Design of Sewers by the Rational Method

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
				Increment														
Sewer	Length L	Slope S (ft)	Total area drained (ac)	Catchment	Area (ac)	C	CA	ΣCA	Inlet time (min)	Upstream sewer flow time (min)	t_c (min)	t_d (min)	i (in/hr)	Design discharge Q_p (cfs)	Computed diameter D_r (ft)	Pipe size used D_n (ft)	Flow velocity (fps)	Sewer flow time (min)
1.1	390	0.0200	2.20	1.1	2.20	0.65	1.43	1.43	11.0	–	11.0	11.0	4.00	5.72	1.08	1.25	4.6	1.42
1.2	183	0.0041	1.20	1.2	1.20	0.80	0.96	0.96	9.2	–	9.2	9.2	4.30	4.13	1.28	1.50	2.3	1.31
2.1	177	0.0245		2.1	3.90	0.70	2.73		13.7	–	13.7							
				1.1					11.0	1.4	12.4							
				1.2					9.2	1.3	10.5							
			7.30					5.12				13.7	3.68	18.8	1.62	1.75	7.8	0.38
2.2	200	0.0180	0.45	2.2	0.45	0.80	0.36	0.36	5.2	–	5.2	5.2	5.30	1.91	0.73	0.83	3.5	0.95
3.1	156	0.0104		3.1	0.70	0.70	0.49		8.7	–	8.7							
									13.7	0.4	14.1							
				2.2					5.2	1.0	6.2							
			8.45					5.97				14.1	3.63	21.6	2.00	2.00	6.9	0.39
3.2	210	0.0175	0.60	3.2	0.60	0.85	0.51	0.51	5.9	–	5.9	5.9	5.07	2.59	0.82	0.83	4.7	0.74
3.3	130	0.0300	1.70	3.3	1.70	0.65	1.11	1.11	11.8	–	11.8	11.8	3.90	4.32	0.90	1.00	5.5	0.39
4.1	181	0.0041		4.1	2.00	0.75	1.50		9.5	–	9.5							
									14.1	0.4	14.5							
				3.3					11.8	0.4	12.2							
			12.75					9.09				14.5	3.60	32.7	2.79	3.00	4.6	0.65
4.2	200	0.0026	0.65	4.2	0.65	0.85	0.55	0.55	6.2	–	6.2	6.2	4.98	2.75	1.20	1.25	2.2	1.49
5.1	230	0.0028		5.1	1.25	0.70	0.88		10.3	–	10.3							
									14.5	0.7	15.2							
			14.65					10.52				15.2	3.50	36.8	3.13	3.50	3.8	1.00
5.2	70	0.0250	0.70	5.2	0.70	0.65	0.46	0.46	11.8	–	11.8	11.8	3.90	1.79	0.67	0.67	5.1	0.23
5.3	130	0.0060	1.70	5.3	1.70	0.55	0.94	0.94	17.6	–	17.6	17.6	3.30	3.10	1.07	1.25	2.5	0.86

Source: Yen (1978).

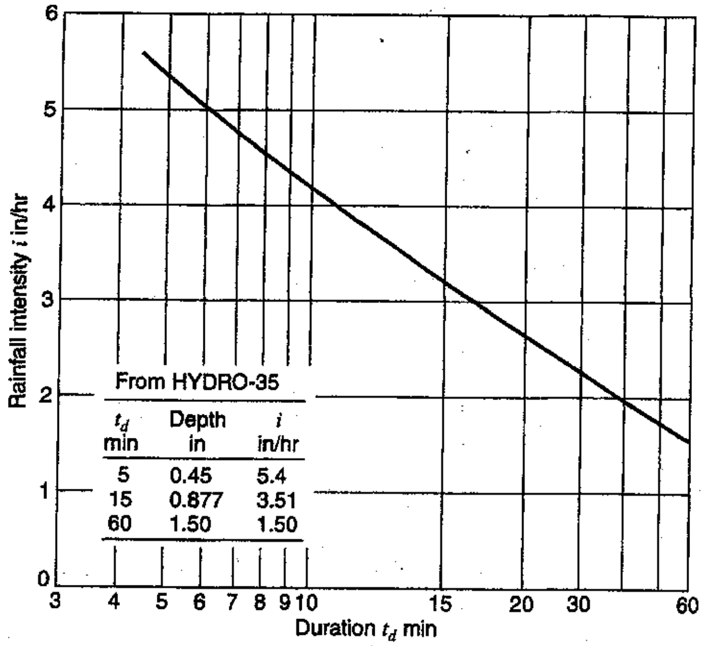

Figure 11.1.2 Variation of rainfall intensity with duration at Urbana, Illinois (from Yen (1978)).

The size of a particular pipe is based upon computing the smallest available commercial pipe that can handle the peak flow rate determined using the rational formula (11.1.2). Manning's equation has been popular in the United States for sizing pipes:

$$Q = \frac{m}{n} S_f^{1/2} A R^{2/3} \tag{11.1.5}$$

where m is 1.486 for U.S. customary units (1 for SI units), S_f is the friction slope, A is the inside cross-sectional area of the pipe $\pi D^2/4$ in ft^2 (m^2), R is the hydraulic radius, $R = A/P = D/4$ in ft (m), P is the wetted perimeter (πD) in ft (m), and K is the inside pipe diameter in ft (m). By substituting in the bed slope S_0 for the friction slope (assuming uniform flow) and $A = \pi D^2/4$ and $R = D/4$ (assuming that the pipe is flowing full under gravity, not pressurized), Manning's equation becomes

$$Q = \frac{m}{n} S_0 \left(\frac{\pi D^2}{4}\right)\left(\frac{D}{4}\right)^{2/3} = m\left(\frac{0.311}{n}\right) S_0^{1/2} D^{8/3} \tag{11.1.6}$$

Equation (11.1.6) can be solved for the diameter

$$D = \left(\frac{m_D Q n}{\sqrt{S_0}}\right)^{3/8} \tag{11.1.7}$$

where m_D is 2.16 for U.S. customary units (3.21 for SI units). Q is determined using the rational formula, and D is rounded up to the next commercial size pipe. The Darcy–Weisbach equation can also be used to size pipes,

$$Q = A\left(\frac{8g}{f} R S_f\right)^{1/2} \tag{11.1.8a}$$

Equation (11.1.8a) can be solved for D using $S_f = S_0$ as

$$D = \left(\frac{0.811 f Q^2}{g S_0}\right)^{1/5} \tag{11.1.8b}$$

which is valid for any dimensionally consistent set of units. ■

SOURCE : Mays, L. W.
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