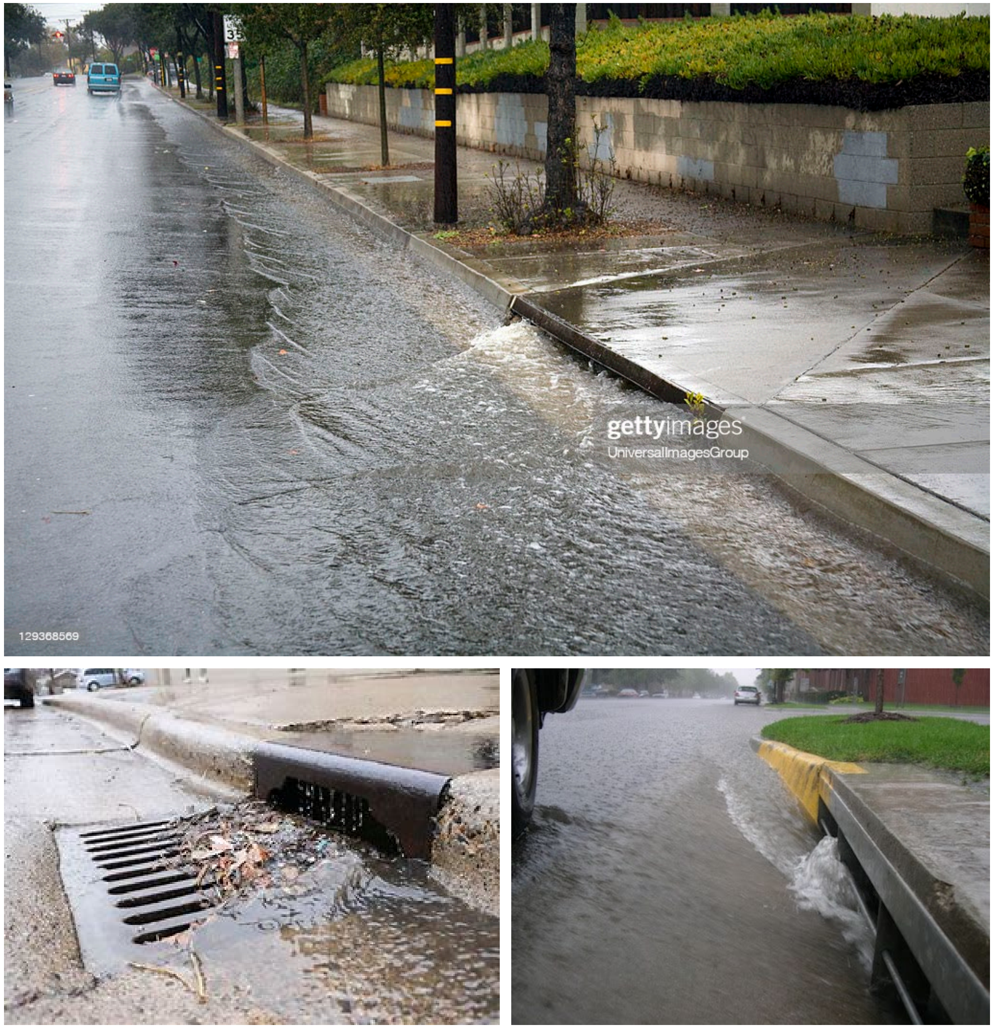
Module 3:
Surface Hydrology

Urban Drainage Systems: Storm Sewer

CWR 3540: Water Resources Engineering
FIU Department of Civil & Environmental Engineering
Professor Fuentes

DIRECT RUNOFF



Types of Drainage Systems

Types

- Urban drainage systems
- Agricultural drainage systems
- Roadway drainage systems
- Airport drainage systems

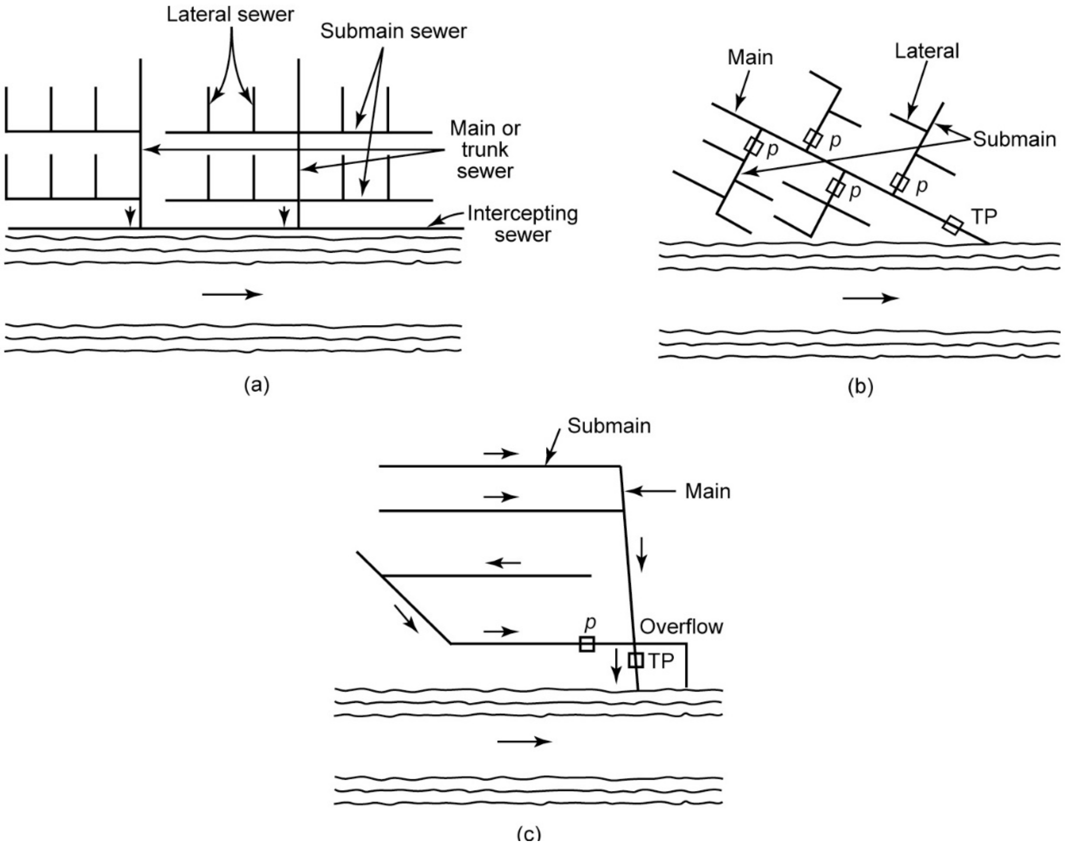
Illustrations

Urban Sewer Systems

- Combined sewer system (USA past practice): stormwater to wastewater ratio > 20-100
- Separate (USA current practice)
  - Stormwater sewer system (for rainfall excess)
  - Sanitary sewer system or "dry-weather flow" (for wastewater from households, commercial establishments, industries, etc.)

Typical of Sewer Layouts
(an illustration for sanitary sewer)

Figure 16.1 Layout of sanitary sewers: (a) perpendicular pattern; (b) fan pattern; (c) zone pattern. *p*, pumping station; TP, treatment plant.



Example of Layout of a Storm Drainage System for a Residential Area

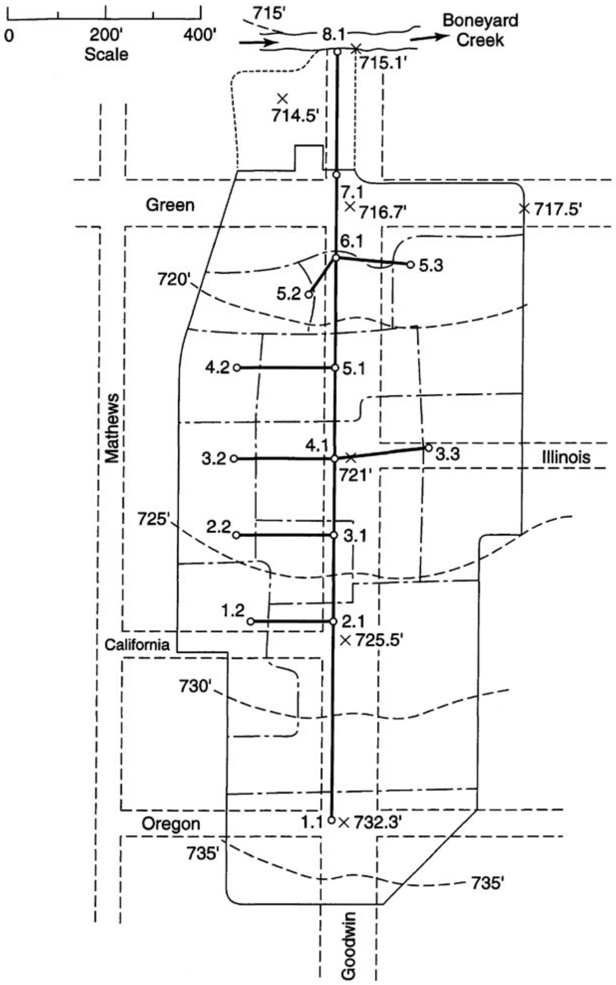


Figure 11.1.1
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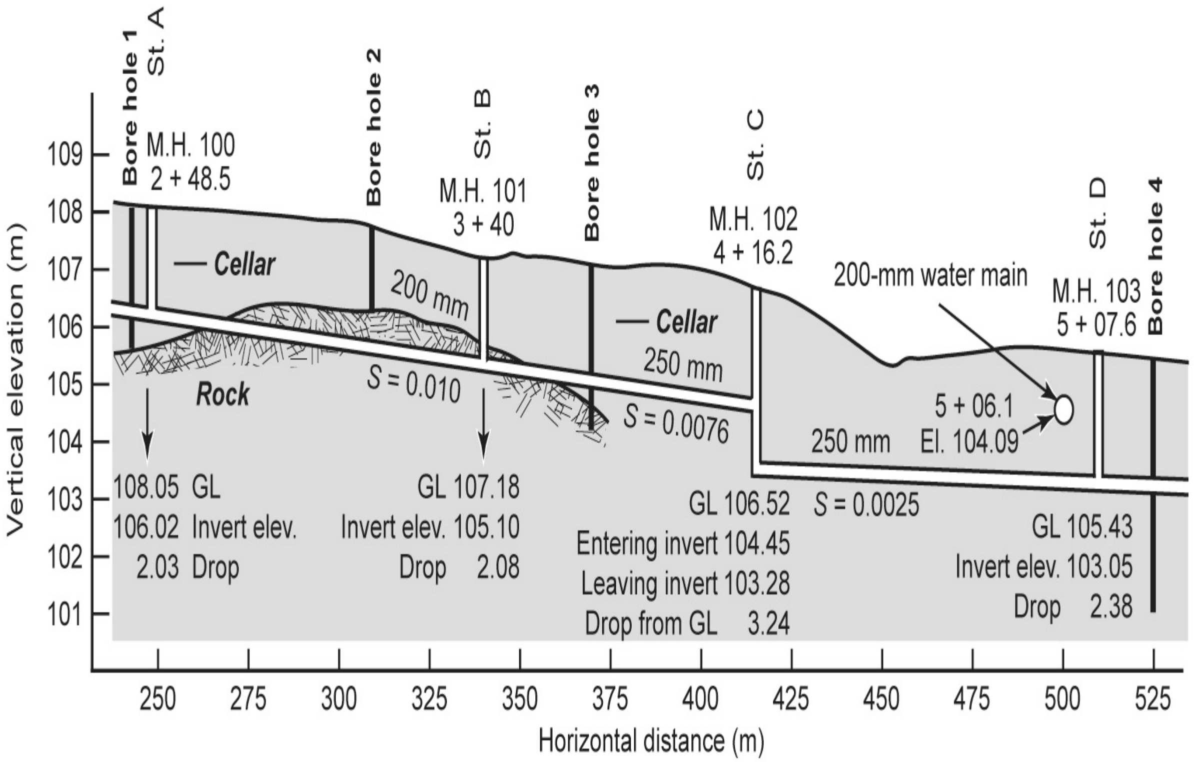
Source: Mays, Wiley (2012)

Elements of a Sewer Section

Figure 16.2 Profile of a sewer section (modified from McGhee, 1991).



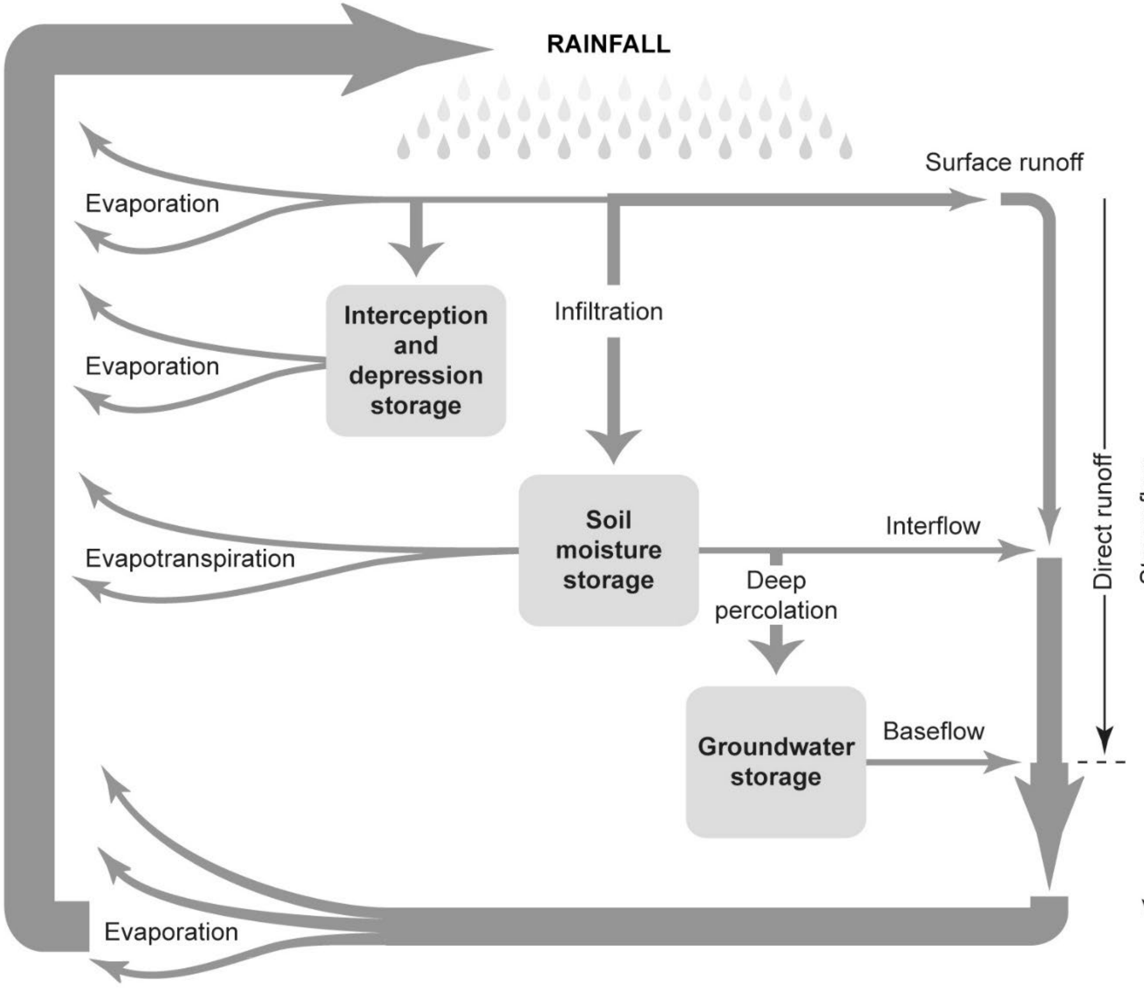
<u>ELEMENTS IN PROFILES</u>

Ground level
Borings
Rock levels
Underground structures
Elevations of foundations and cellars
Cross streets
Manholes
Sewer inverts
Sewer lopes and sizes
Etc.

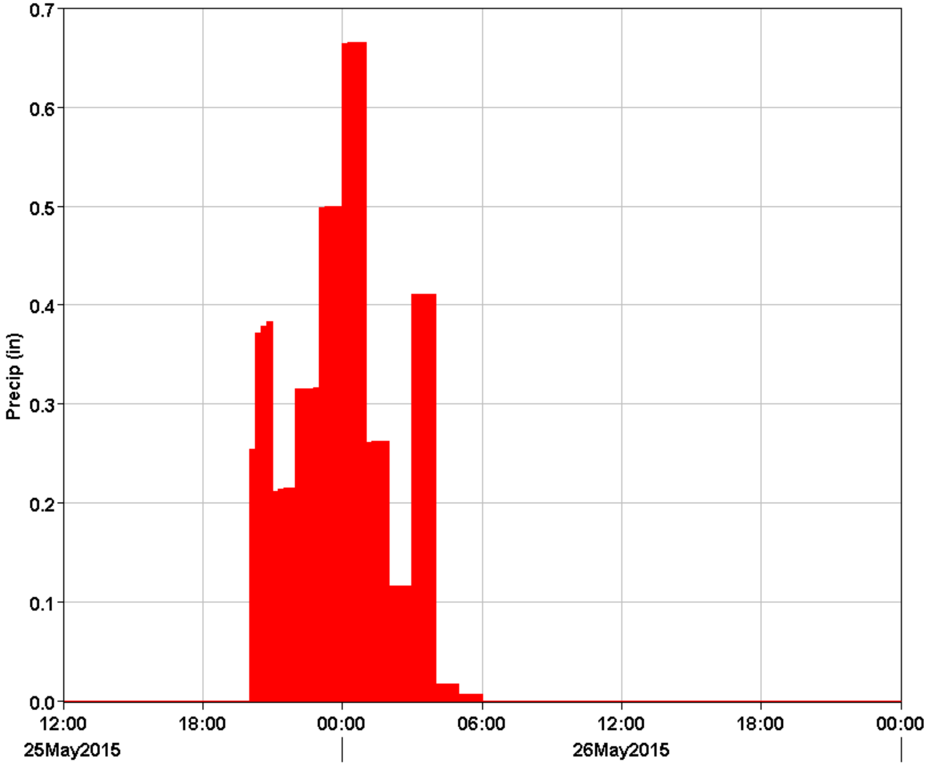
Direct Runoff in Drainage Areas

(or watersheds, basins, catchment areas)

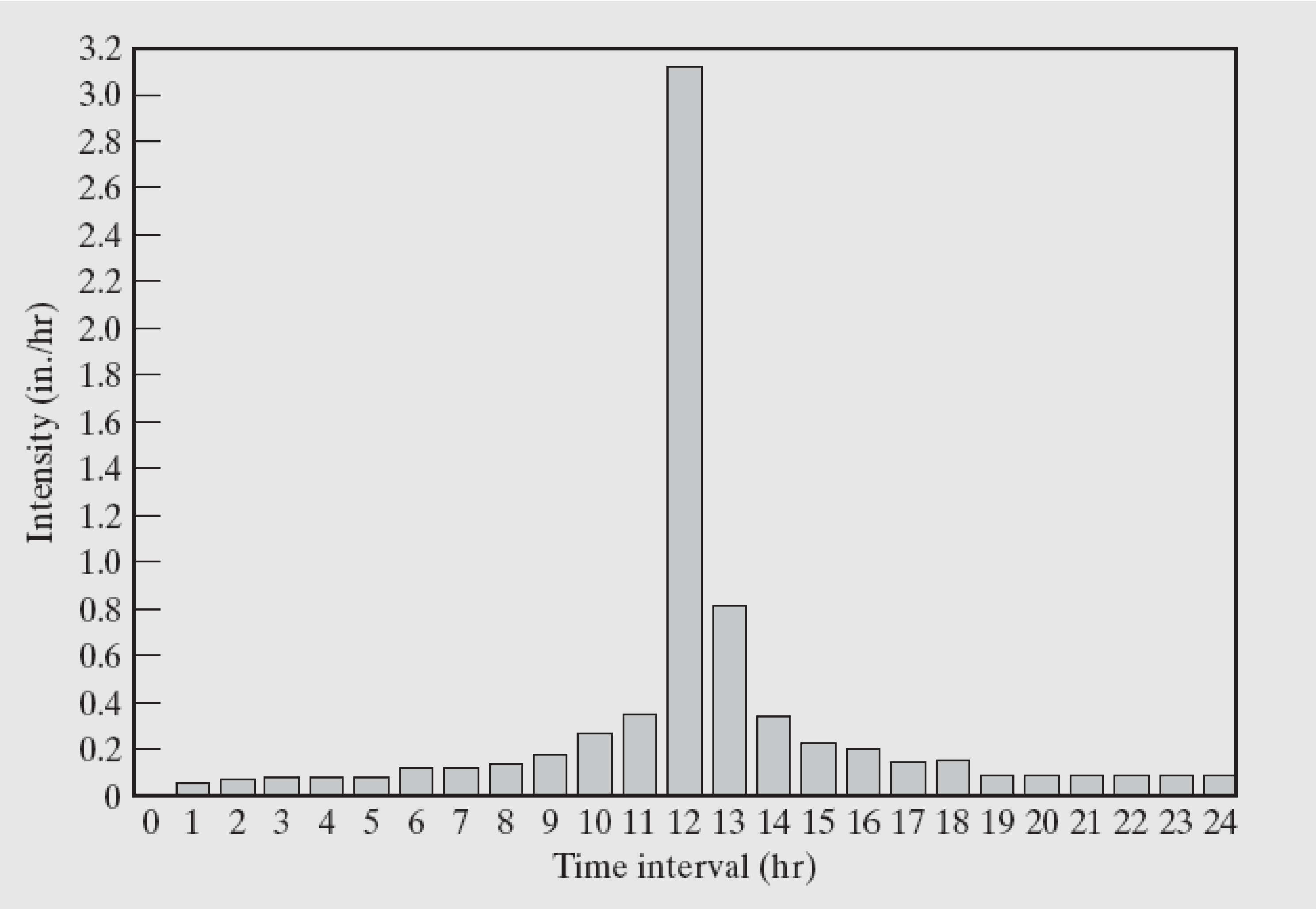
Figure 9.1 Forms of runoff in the hydrologic cycle.



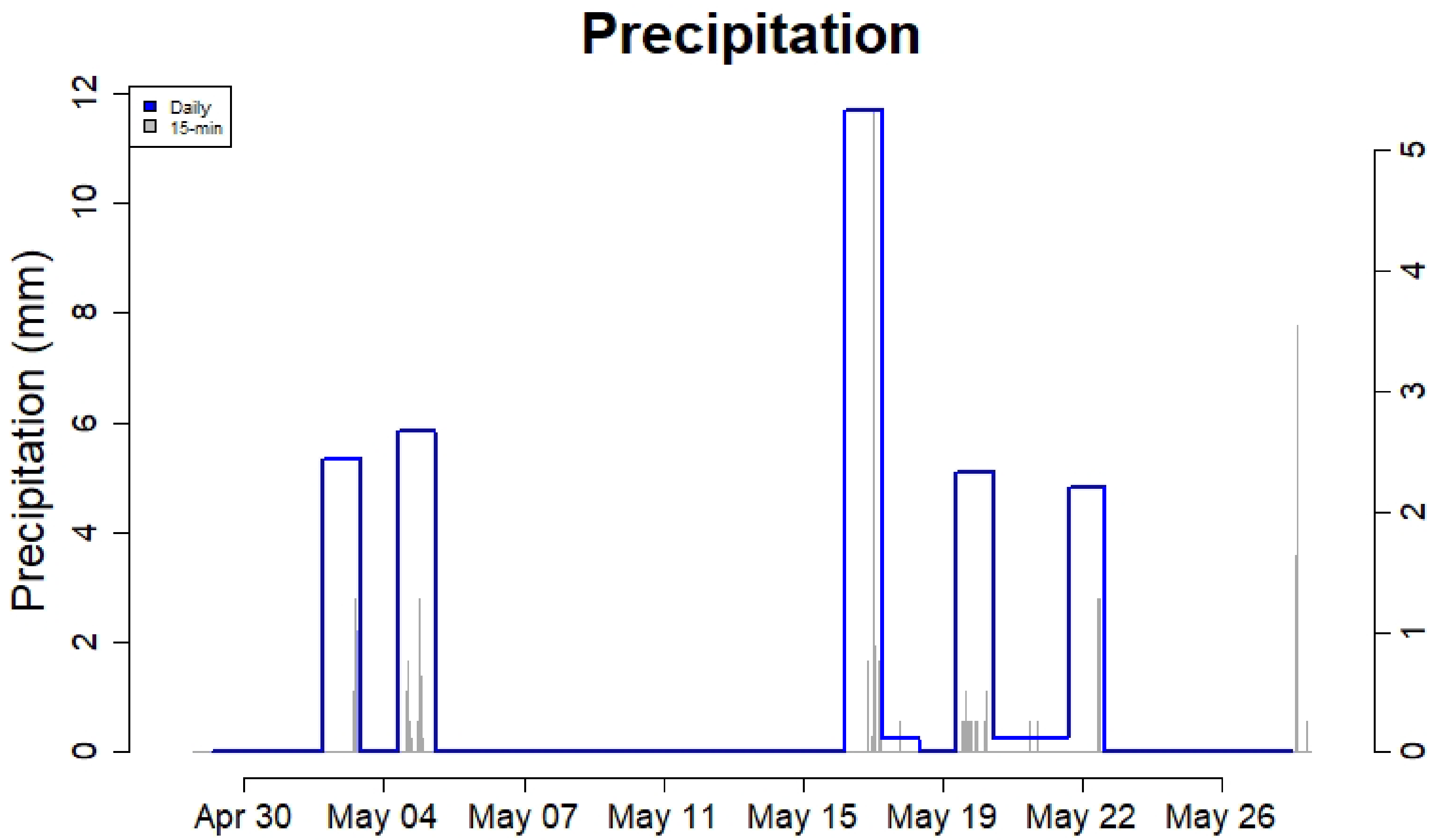
Single-Event Hyetograph



Synthetic Hyetograph

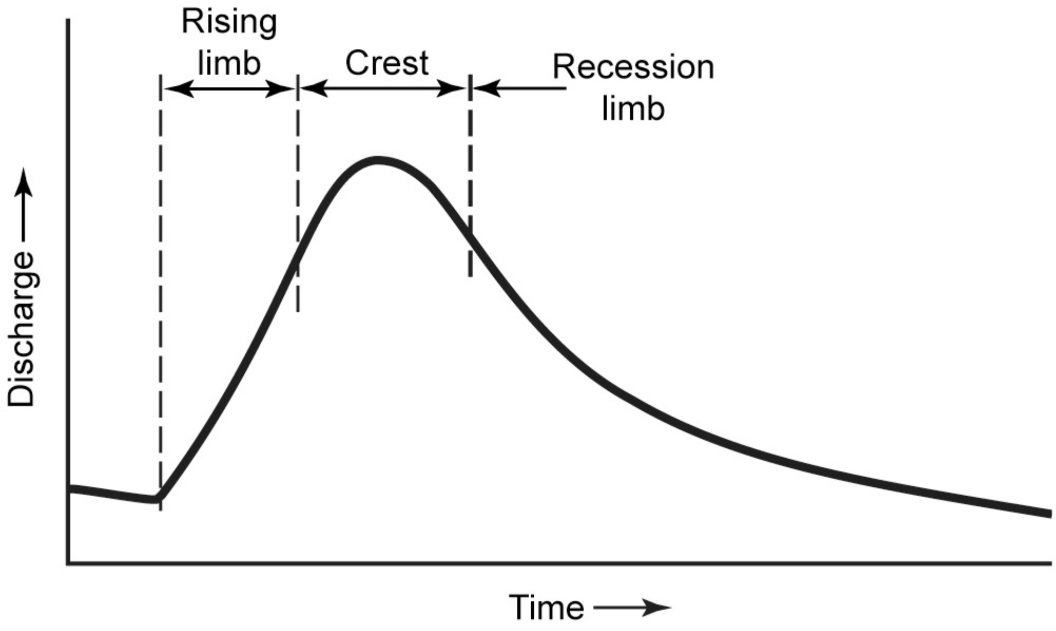


Rainfall Time Series

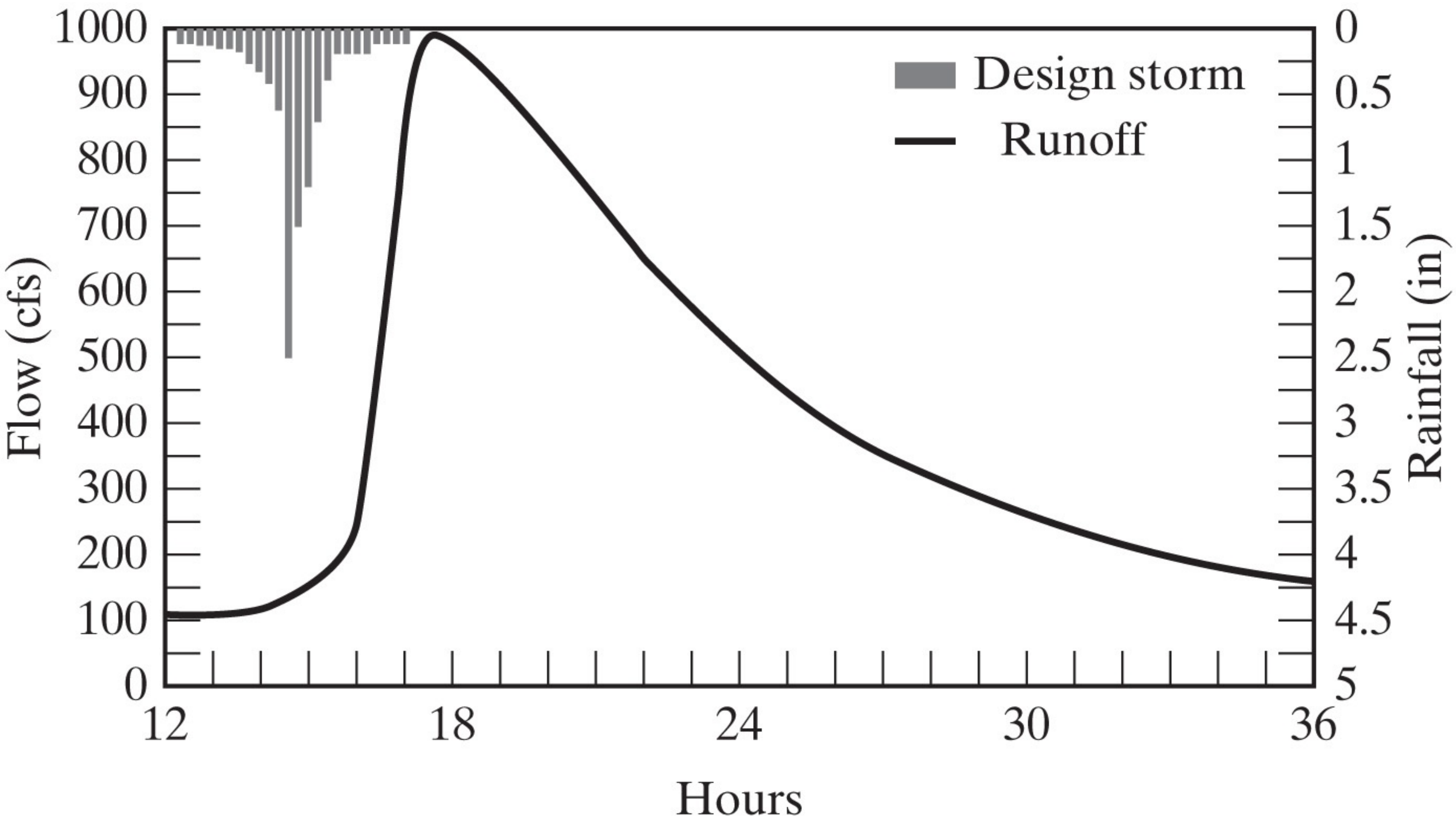


Hydrograph

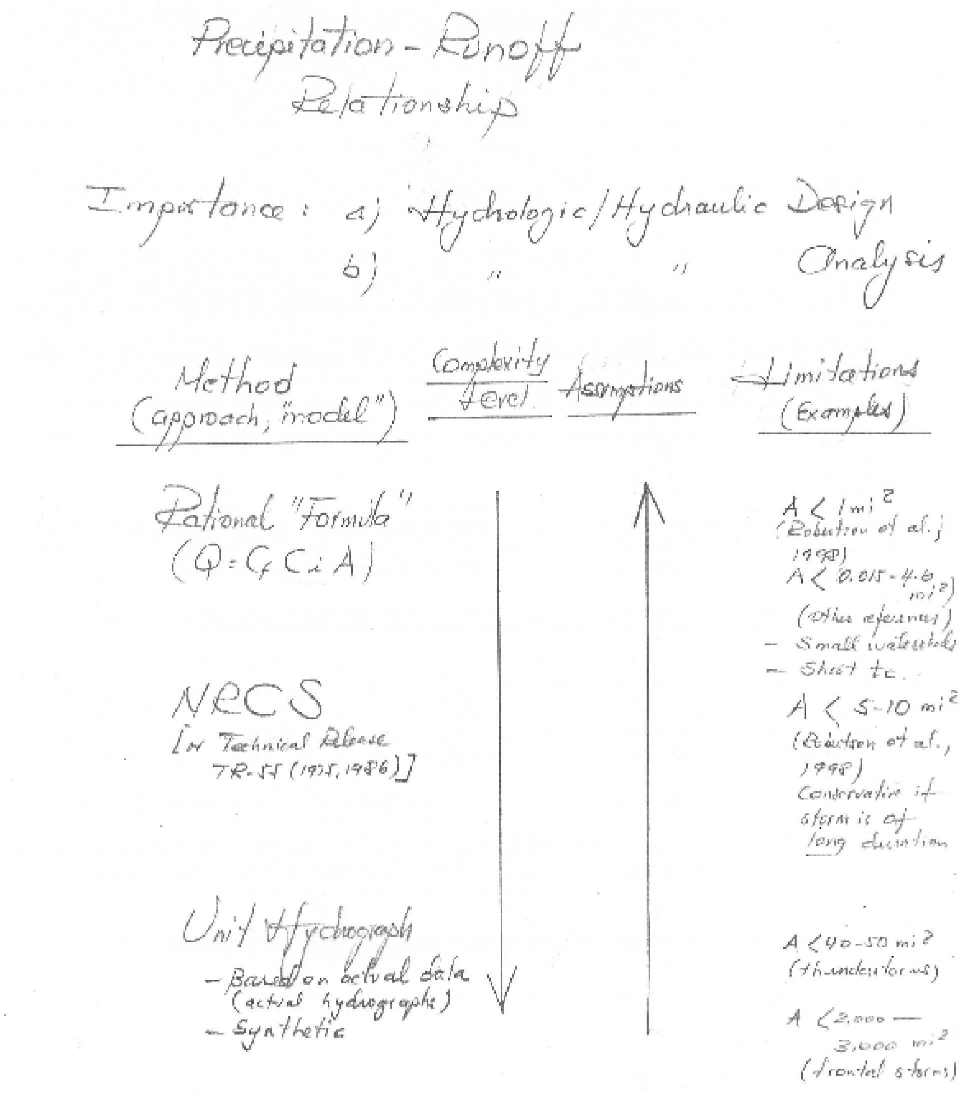
Figure 9.5 Simple storm hydrograph.



Typical Rainfall and Hydrograph.



Precipitation-Runoff Relationships



Precipitation-Runoff
Relationship

Importance: a) Hydrologic/Hydraulic Design
b) " " Analysis

Method (approach, "model")	Complexity Level	Assumptions	Limitations (Examples)
	↓	↑	
Rational "Formula" ($Q=C_f\,C\,i\,A$)			$A < 1\ mi^2$ (Robertson et al., 1998) $A < (0.015-4.6\ mi^2)$ (Other references) – Small watersheds – Short tc.
NRCS [or Technical Release TR-55 (1975, 1986)]			$A < 5-10\ mi^2$ (Robertson et al., 1998) Conservative if storm is of long duration
Unit Hydrograph – Based on actual data (actual hydrographs) – Synthetic			$A < 40-50\ mi^2$ (thunderstorms) $A < 2{,}000 - 3{,}000\ mi^2$ (frontal storms)

RATIONAL METHOD

Rational Method (or “formula)

- $Q = C_f\,C\,i\,A$ (L^3T^{-1}), where:
  - Q = *“design”* storm peak flow rate (in ft^3/s or cfs)
  - C_f = frequency factor, function of return period T
  - C = runoff coefficient (dimensionless, 0 to 1)
  - i = intensity of precipitation for a given duration that equals the “time of concentration”, t_c in ft/s, at a *“design”* return period T (*in years*) and
  - A = drainage area (ft^2)
- Q is *in cfs*, if i is in *in/h* and A in acres

  [i.e., 1.008 acre-in/h = 1 cfs]
- $C_f \cdot C < 1$
- Robertson et al. (1998): application to $A < 1\ mi^2$

Weighted-C

- $C_w = (\sum C_k a_k)/A$, where

  C_w = weighted runoff coefficient

  C_k = runoff coefficient of subdrainage area a_j

  $A = \sum a_k$ = total drainage area or sum of individual subdrainage areas

C_f-Variation

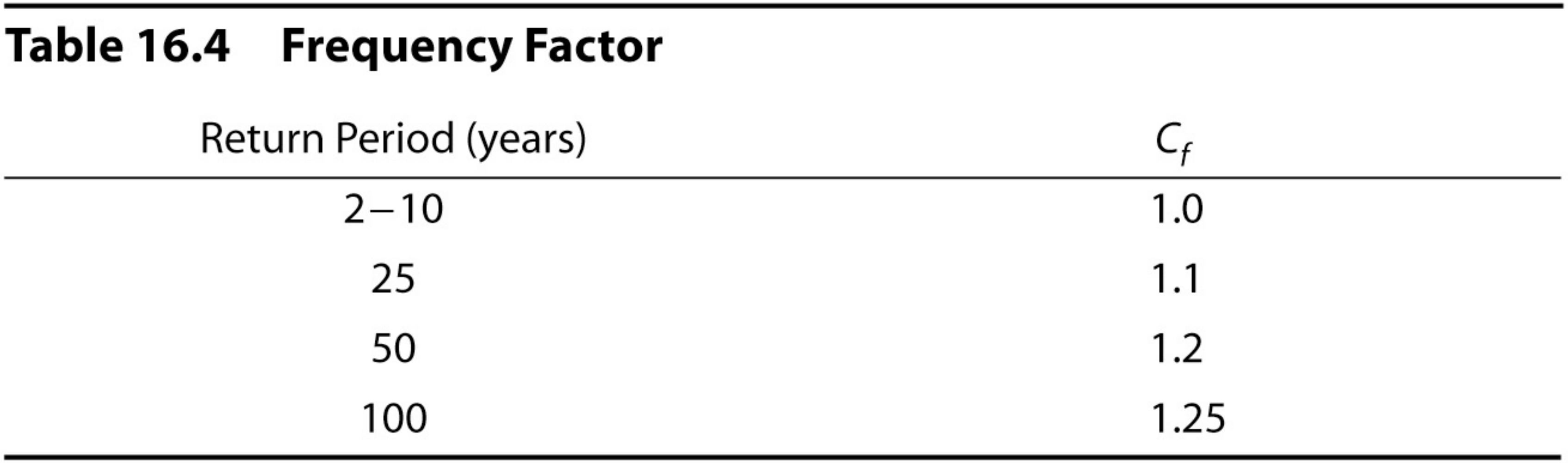
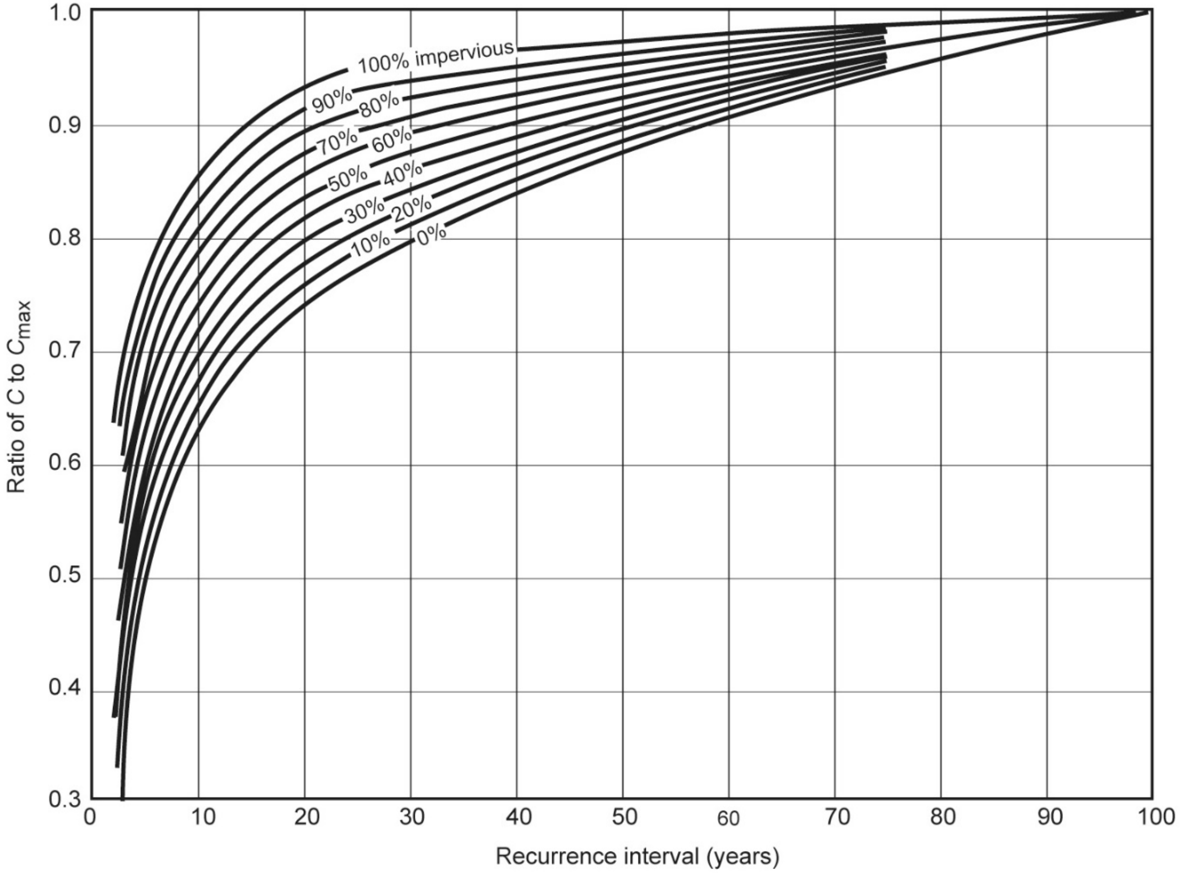


Table 16.4 Frequency Factor

Return Period (years)	C_f
2–10	1.0
25	1.1
50	1.2
100	1.25

C_f-Estimation: US DOT Guideline

Figure 16.5 Correction for design storm frequency (from U.S. Department of Transportation, 1979).



Example of Runoff Coefficient Database

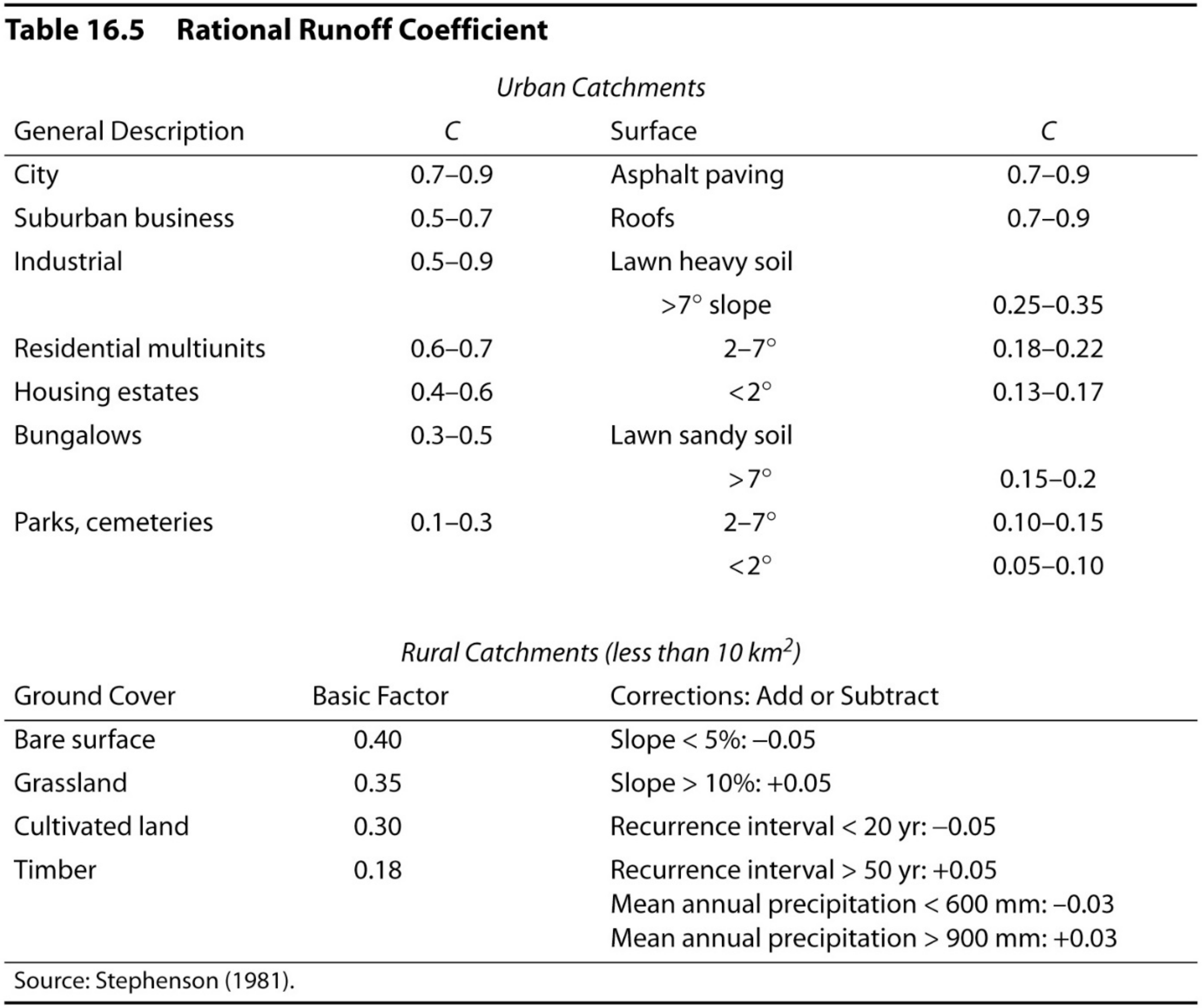


Table 16.5 Rational Runoff Coefficient

Urban Catchments

General Description	C	Surface	C
City	0.7–0.9	Asphalt paving	0.7–0.9
Suburban business	0.5–0.7	Roofs	0.7–0.9
Industrial	0.5–0.9	Lawn heavy soil	
		>7° slope	0.25–0.35
Residential multiunits	0.6–0.7	2–7°	0.18–0.22
Housing estates	0.4–0.6	<2°	0.13–0.17
Bungalows	0.3–0.5	Lawn sandy soil	
		>7°	0.15–0.2
Parks, cemeteries	0.1–0.3	2–7°	0.10–0.15
		<2°	0.05–0.10

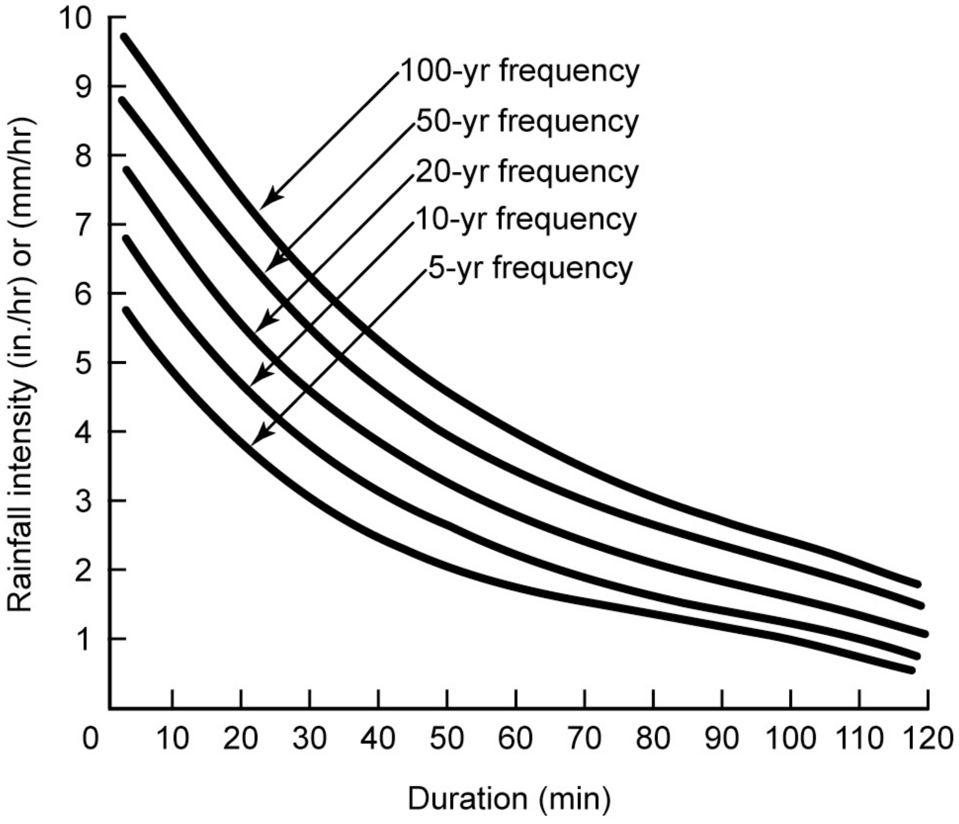
Rural Catchments (less than 10 km^2)

Ground Cover	Basic Factor	Corrections: Add or Subtract
Bare surface	0.40	Slope < 5%: –0.05
Grassland	0.35	Slope > 10%: +0.05
Cultivated land	0.30	Recurrence interval < 20 yr: –0.05
Timber	0.18	Recurrence interval > 50 yr: +0.05
		Mean annual precipitation < 600 mm: –0.03
		Mean annual precipitation > 900 mm: +0.03

Source: Stephenson (1981).

Example of Typical IDFs

Figure 16.6 Intensity-duration-frequency (IDF) curves for Bridgewater, CT.



Time of Concentration, t_c

- t_c = time required for runoff to travel from the hydraulically most remote part of the drainage area to reach the point of interest when calculation the "design" peak flow
- $t_c = t_o + \sum t_f$, *where*
  - t_o = inlet time or commonly overland flow
  - t_f = flow time traveling in all upstream sewers connected after to up to the point of design

NRCS Flow Types in Drainage Areas

(before entering the first inlet)

- 1) Sheet flow – thin layer of flow up to 300 ft:

  $T_{t1} = 0.42(nL)^{0.8}/[(P_2)^{0.5}\ S^{0.4}]$ (Equation 16.9)

- 2) Shallow concentrated flow (> 300 ft):

  $T_{12} = L/V$ (Eq. 16.10, with V from Figure 16.7)

- 3) Open channel flow:

  $V = (K/n)\ R^{2/3}S^{1/2}$ (Eqs. 14.9 a and b, and n selected from Table 14.4)

- 4) Or the combination of the above thereof

Estimation of Overland Flow: Example 16.8

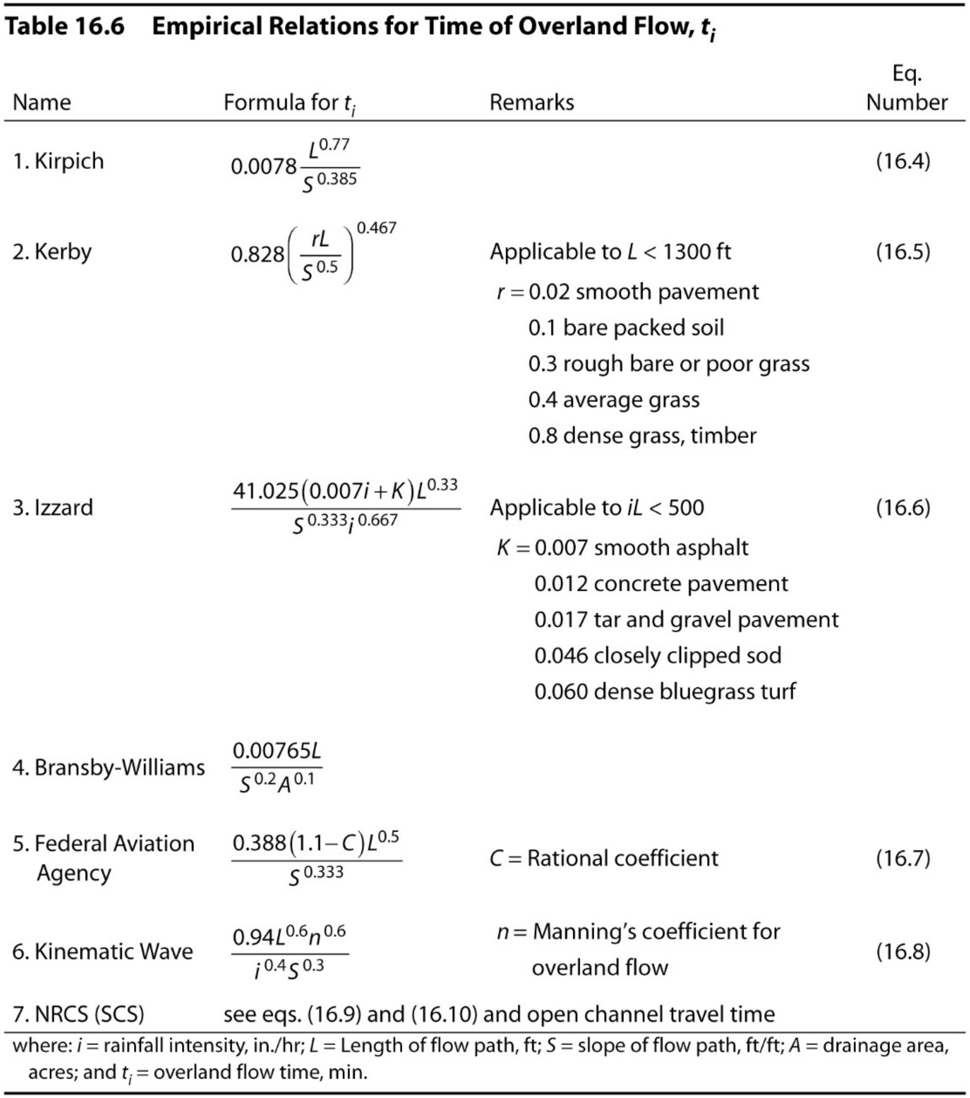


Table 16.6 Empirical Relations for Time of Overland Flow, t_i

Name	Formula for t_i	Remarks	Eq. Number
1. Kirpich	$0.0078\dfrac{L^{0.77}}{S^{0.385}}$		(16.4)
2. Kerby	$0.828\left(\dfrac{rL}{S^{0.5}}\right)^{0.467}$	Applicable to $L < 1300$ ft $r = 0.02$ smooth pavement 0.1 bare packed soil 0.3 rough bare or poor grass 0.4 average grass 0.8 dense grass, timber	(16.5)
3. Izzard	$\dfrac{41.025(0.007i+K)L^{0.33}}{S^{0.333}i^{0.667}}$	Applicable to $iL < 500$ $K = 0.007$ smooth asphalt 0.012 concrete pavement 0.017 tar and gravel pavement 0.046 closely clipped sod 0.060 dense bluegrass turf	(16.6)
4. Bransby-Williams	$\dfrac{0.00765L}{S^{0.2}A^{0.1}}$		
5. Federal Aviation Agency	$\dfrac{0.388(1.1-C)L^{0.5}}{S^{0.333}}$	C = Rational coefficient	(16.7)
6. Kinematic Wave	$\dfrac{0.94L^{0.6}n^{0.6}}{i^{0.4}S^{0.3}}$	n = Manning's coefficient for overland flow	(16.8)
7. NRCS (SCS)	see eqs. (16.9) and (16.10) and open channel travel time		

where: i = rainfall intensity, in./hr; L = Length of flow path, ft; S = slope of flow path, ft/ft; A = drainage area, acres; and t_i = overland flow time, min.

Travel time in Open Channel Flow: Manning’s Kinematic Equation 16.9

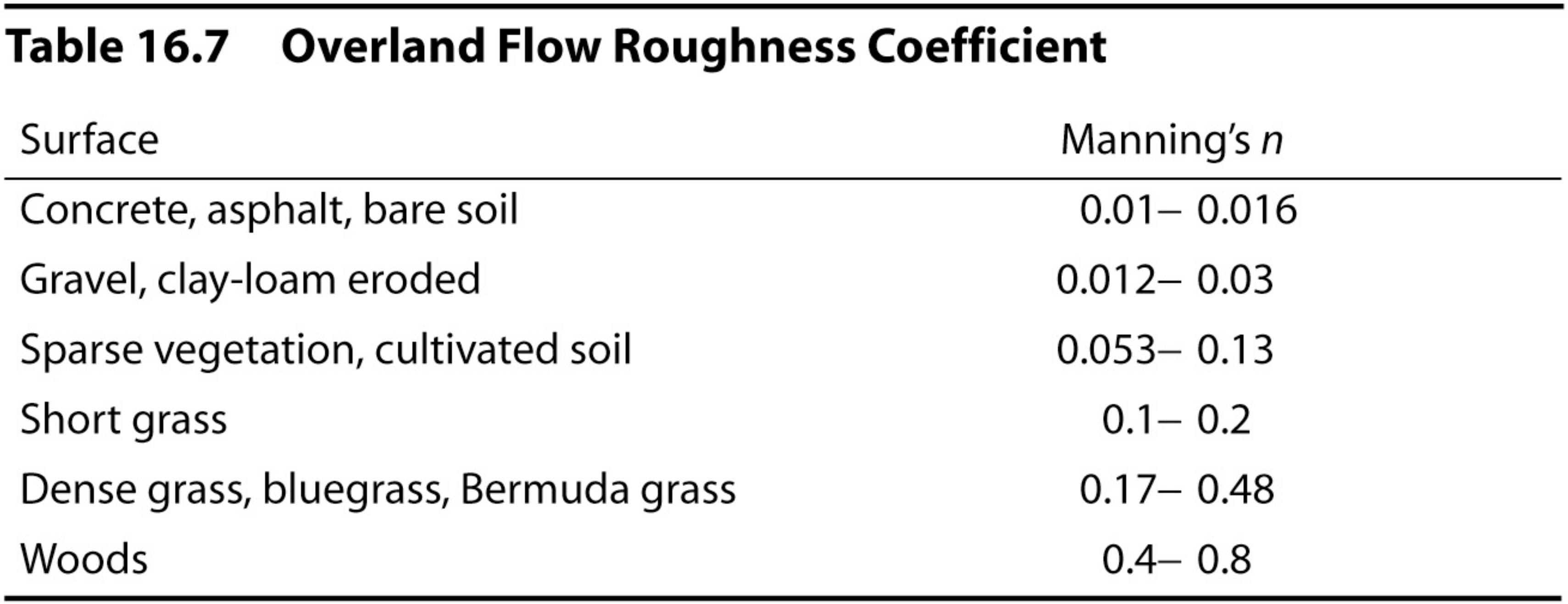
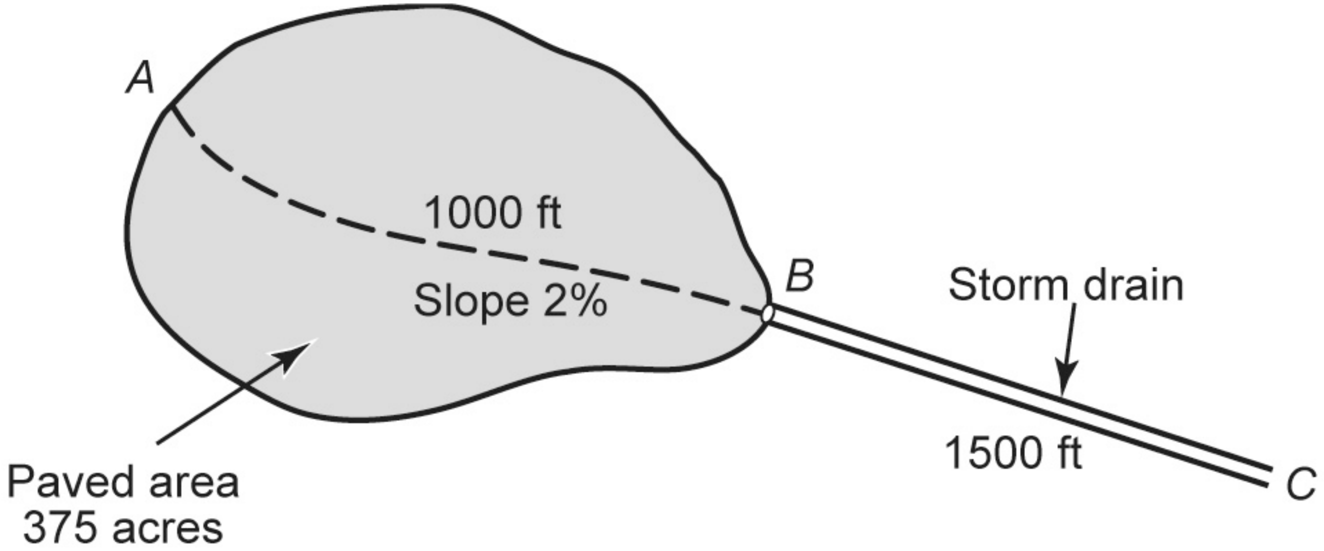


Table 16.7 Overland Flow Roughness Coefficient

Surface	Manning’s *n*
Concrete, asphalt, bare soil	0.01– 0.016
Gravel, clay-loam eroded	0.012– 0.03
Sparse vegetation, cultivated soil	0.053– 0.13
Short grass	0.1– 0.2
Dense grass, bluegrass, Bermuda grass	0.17– 0.48
Woods	0.4– 0.8

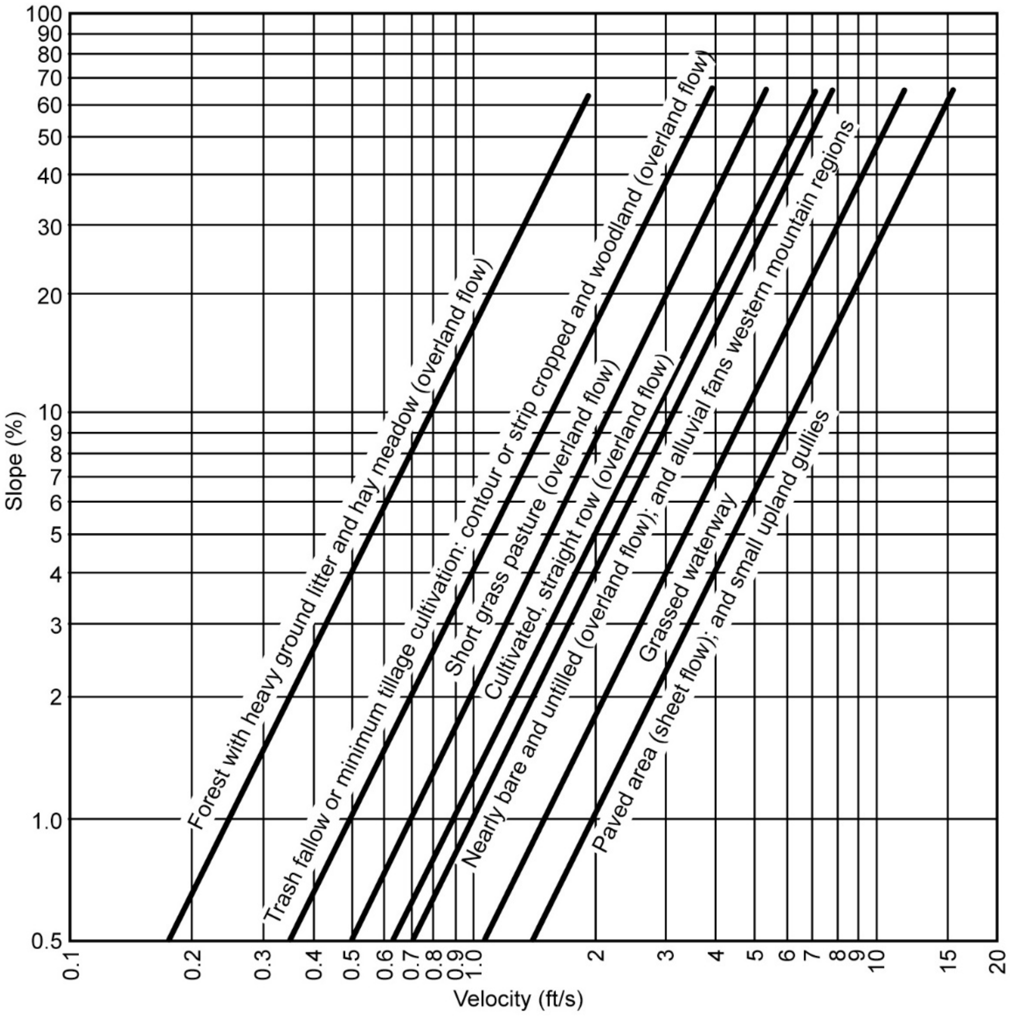
Sketch for Problem 16.8

Figure 16.8 Urbanized watershed for Example 16.8.



Travel Time for Shallow Concentrated Flow: Average Velocity Estimation

Figure 16.7 Average velocity of overland flow (from U.S. SCS 1975b).



Rational Method Application: Examples 16.9 & 16.10

- For drainage areas (i.e., quite often) with different types of surfaces (see Equation 16.11):
  - $Q = i\ C_f \sum C_j a_j$, where

    Cj = runoff coeficient of sub drainage area a_j

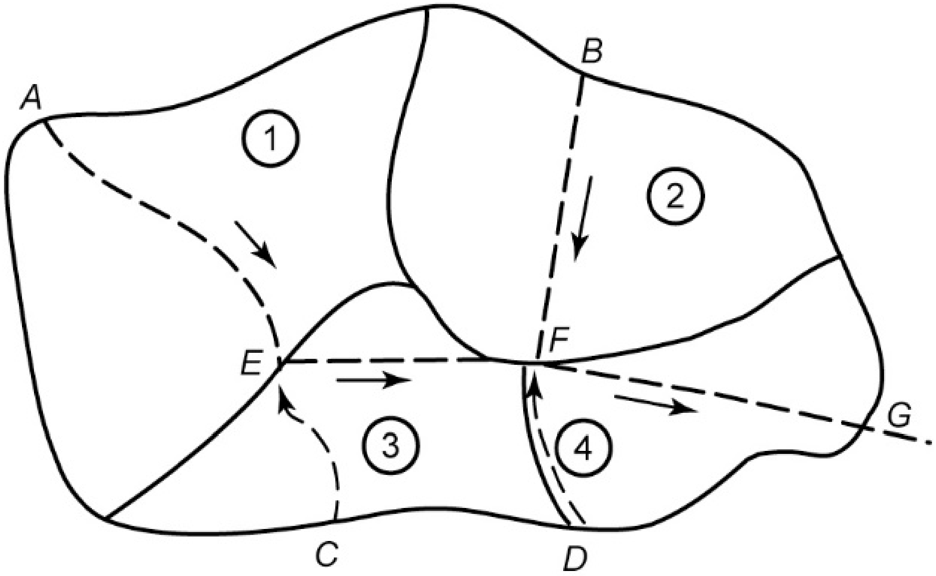
    $A = \sum a_j$

    i = rainfall intensity for the t_c, which is the longest total time to the point where the value of Q is needed

Sketch for Problem 16.8

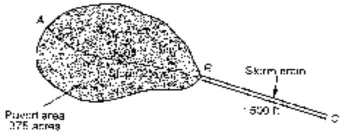
(Peak discharges at Outlet and Interim Points of Entry (e.g., manholes) by *"Lloyd-Davies Method"*

Figure 16.9 Watershed for Example 16.9.



Rational Formula: Example Applications 1

Figure 16.8 Urbanized watershed for Example 16.8.



EXAMPLE 16.8

An urbanized watershed in Providence, Rhode Island, is shown in Figure 16.8. Determine the time of concentration to point C by the various methods. The average velocity of flow in the storm drain = 3 ft/s.

SOLUTION

(a) Time of overland flow:

1. Kirpich method

$$t_1 = \frac{0.0078(1000)^{0.77}}{(0.02)^{0.385}} = 7.13 \text{ min}$$

2. Kerby method

$$r = 0.02$$

$$t_1 = 0.828\left[\frac{(0.02)(1000)}{(0.02)^{0.5}}\right]^{0.467} = 8.36 \text{ min}$$

3. Izzard method. Assume that the time of concentration = 10 min. For Providence, RI (area 3 in Fig. 2.7) and 5 year frequency

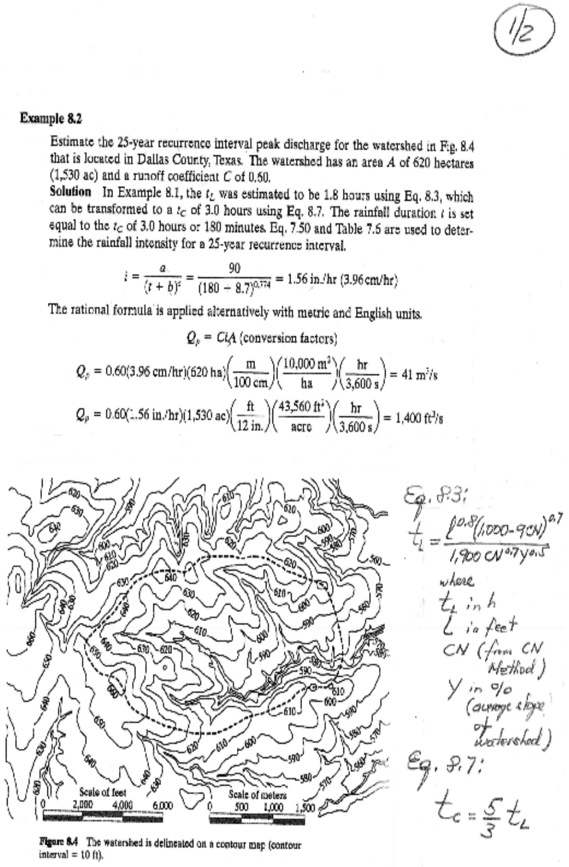
$$i = \frac{131.1}{t+19} = \frac{131.1}{10+19} = 4.52 \text{ in./h}$$

$iL = (4.52)(1000) = 4520 > 500$; thus the formula is not applicable

4. Bransby-Williams method

$$t_1 = \frac{0.00765(1000)}{(0.02)^{0.2}(375)^{0.1}} = 9.25 \text{ min}$$

Rational Formula:
Examples of Applications 2



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Example 8.2

Estimate the 25-year recurrence interval peak discharge for the watershed in Fig. 8.4 that is located in Dallas County, Texas. The watershed has an area A of 620 hectares (1,530 ac) and a runoff coefficient C of 0.60.

Solution In Example 8.1, the t_L was estimated to be 1.8 hours using Eq. 8.3, which can be transformed to a t_C of 3.0 hours using Eq. 8.7. The rainfall duration t is set equal to the t_C of 3.0 hours or 180 minutes. Eq. 7.50 and Table 7.5 are used to determine the rainfall intensity for a 25-year recurrence interval.

$$i = \frac{a}{(t + b)^c} = \frac{90}{(180 - 8.7)^{0.774}} = 1.56 \text{ in./hr } (3.96 \text{ cm/hr})$$

The rational formula is applied alternatively with metric and English units.

$$Q_p = CiA \text{ (conversion factors)}$$

$$Q_p = 0.60(3.96 \text{ cm/hr})(620 \text{ ha})\left(\frac{\text{m}}{100 \text{ cm}}\right)\left(\frac{10{,}000 \text{ m}^2}{\text{ha}}\right)\left(\frac{\text{hr}}{3{,}600 \text{ s}}\right) = 41 \text{ m}^3/\text{s}$$

$$Q_p = 0.60(1.56 \text{ in./hr})(1{,}530 \text{ ac})\left(\frac{\text{ft}}{12 \text{ in.}}\right)\left(\frac{43{,}560 \text{ ft}^2}{\text{acre}}\right)\left(\frac{\text{hr}}{3{,}600 \text{ s}}\right) = 1{,}400 \text{ ft}^3/\text{s}$$

Figure 8.4 The watershed is delineated on a contour map (contour interval = 10 ft).

Eq. 8.3:

$$t_L = \frac{l^{0.8}(1{,}000 - 9CN)^{0.7}}{1{,}900\, CN^{0.7} Y^{0.5}}$$

where

t_L in h

l in feet

CN (from CN Method)

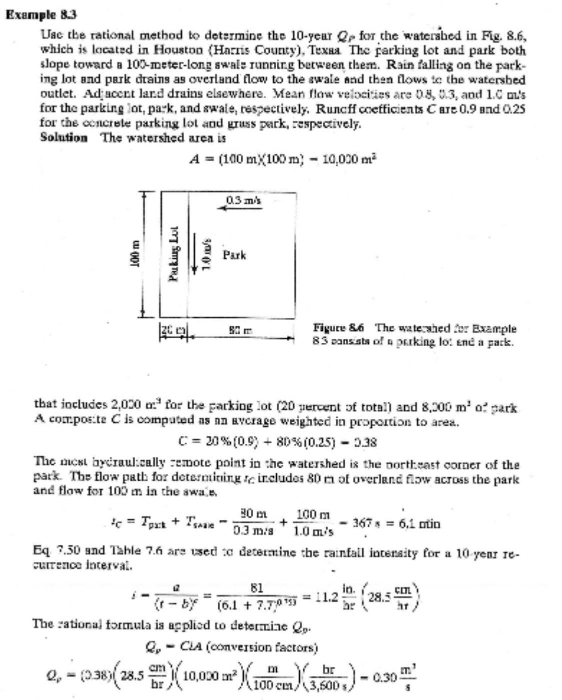
Y in % (average slope of watershed)

Eq. 8.7:

$$t_C = \frac{5}{3} t_L$$

Rational Formula: Examples of Applications 3

(1/2)



Example 8.3

Use the rational method to determine the 10-year Q_P for the watershed in Fig. 8.6, which is located in Houston (Harris County), Texas. The parking lot and park both slope toward a 100-meter-long swale running between them. Rain falling on the parking lot and park drains as overland flow to the swale and then flows to the watershed outlet. Adjacent land drains elsewhere. Mean flow velocities are 0.8, 0.3, and 1.0 m/s for the parking lot, park, and swale, respectively. Runoff coefficients C are 0.9 and 0.25 for the concrete parking lot and grass park, respectively.

Solution The watershed area is

$$A = (100\text{ m})(100\text{ m}) = 10{,}000\text{ m}^2$$

Figure 8.6 The watershed for Example 8.3 consists of a parking lot and a park.

that includes 2,000 m² for the parking lot (20 percent of total) and 8,000 m² of park. A composite C is computed as an average weighted in proportion to area.

$$C = 20\%(0.9) + 80\%(0.25) = 0.38$$

The most hydraulically remote point in the watershed is the northeast corner of the park. The flow path for determining t_C includes 80 m of overland flow across the park and flow for 100 m in the swale.

$$t_C = T_{\text{park}} + T_{\text{swale}} = \frac{80\text{ m}}{0.3\text{ m/s}} + \frac{100\text{ m}}{1.0\text{ m/s}} = 367\text{ s} = 6.1\text{ min}$$

Eq. 7.50 and Table 7.6 are used to determine the rainfall intensity for a 10-year recurrence interval.

$$i = \frac{a}{(t+b)^c} = \frac{81}{(6.1 + 7.7)^{0.753}} = 11.2\,\frac{\text{in.}}{\text{hr}}\left(28.5\,\frac{\text{cm}}{\text{hr}}\right)$$

The rational formula is applied to determine Q_p.

$$Q_p = CIA\text{ (conversion factors)}$$

$$Q_p = (0.38)\left(28.5\,\frac{\text{cm}}{\text{hr}}\right)\left(10{,}000\text{ m}^2\right)\left(\frac{\text{m}}{100\text{ cm}}\right)\left(\frac{\text{hr}}{3{,}600\text{ s}}\right) = 0.30\,\frac{\text{m}^3}{\text{s}}$$

SOURCE: Wurbs & James, 2002

NRCS TR-55 METHOD

NRCS (SCS) TR-55 Method

- NRCS (SCS): Individual storm event watershed “comprehensive” Hydrologic Model TR-20
  - Developed 1964
  - Updated 2015
- NRCS (SCS): Individual storm event “design peak discharge” hydrologic model TR-55
  - Released 1975
  - Revised in 2013

TR-55 Approaches

- Graphical:
  - $q_p = q_u A_m Q F_p$ where
  - q_p = peak discharge, cfs;
  - q_u = unit peak discharge, cfs/mi^2/in; from graphs in TR-55 for t_c and I_a/P, I_a from Table 16.12 and P is the 24-hr rainfall in TR-55, for the rainfall distribution type I, IA, II or III.
  - A_m = drainage area, mi^2;
  - Q is runoff corresponding to 24-hr rainfall of a desired design frequency or return period (Figures B-1 through B-8 in TR-55; and
  - F_p = pond or swamp adjustment factor (Table 16.11)

TR-55 Approaches

- <u>Tabular:</u>
  - $q_p = q_u A_m Q F_p$ where
  - q_p = peak discharge, cfs;
  - q_u = unit peak discharge, cfs/mi^2/in;
  - A_m = drainage area, mi^2;
  - Q is the runoff corresponding to a 24-hr storm, of a desired design frequency or return period (use Eq. 4.18 for P_{24} from TR-55 App. B); and
  - F_p = pond or swamp adjustment factor

Adjustment Factor F_p

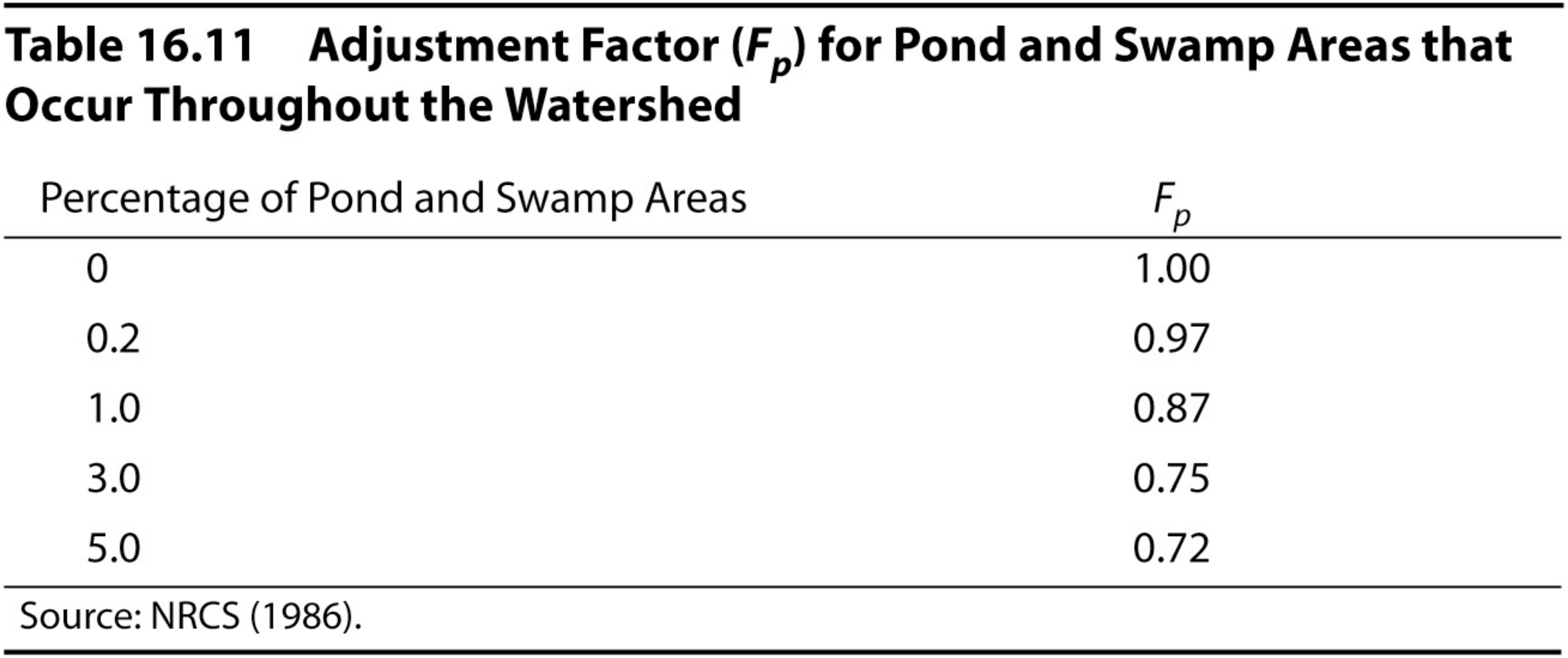


Table 16.11 Adjustment Factor (F_p) for Pond and Swamp Areas that Occur Throughout the Watershed

Percentage of Pond and Swamp Areas	F_p
0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

Source: NRCS (1986).

Unit Peak Discharge, q_u

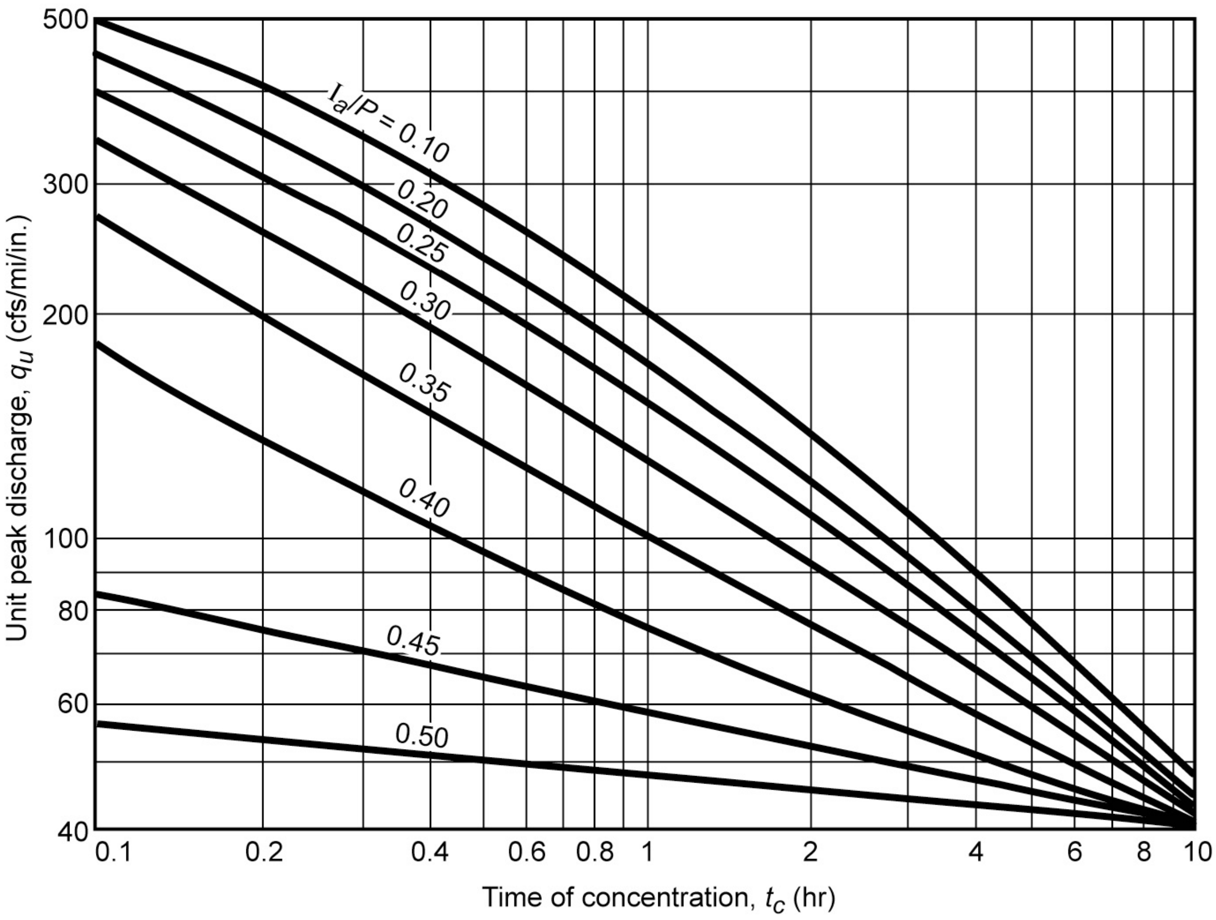


Figure 16.11 Unit peak discharge (q_u) for NRCS (SCS) type I rainfall distribution (from NRCS, 1986).

I_a Values for Runoff CN-values

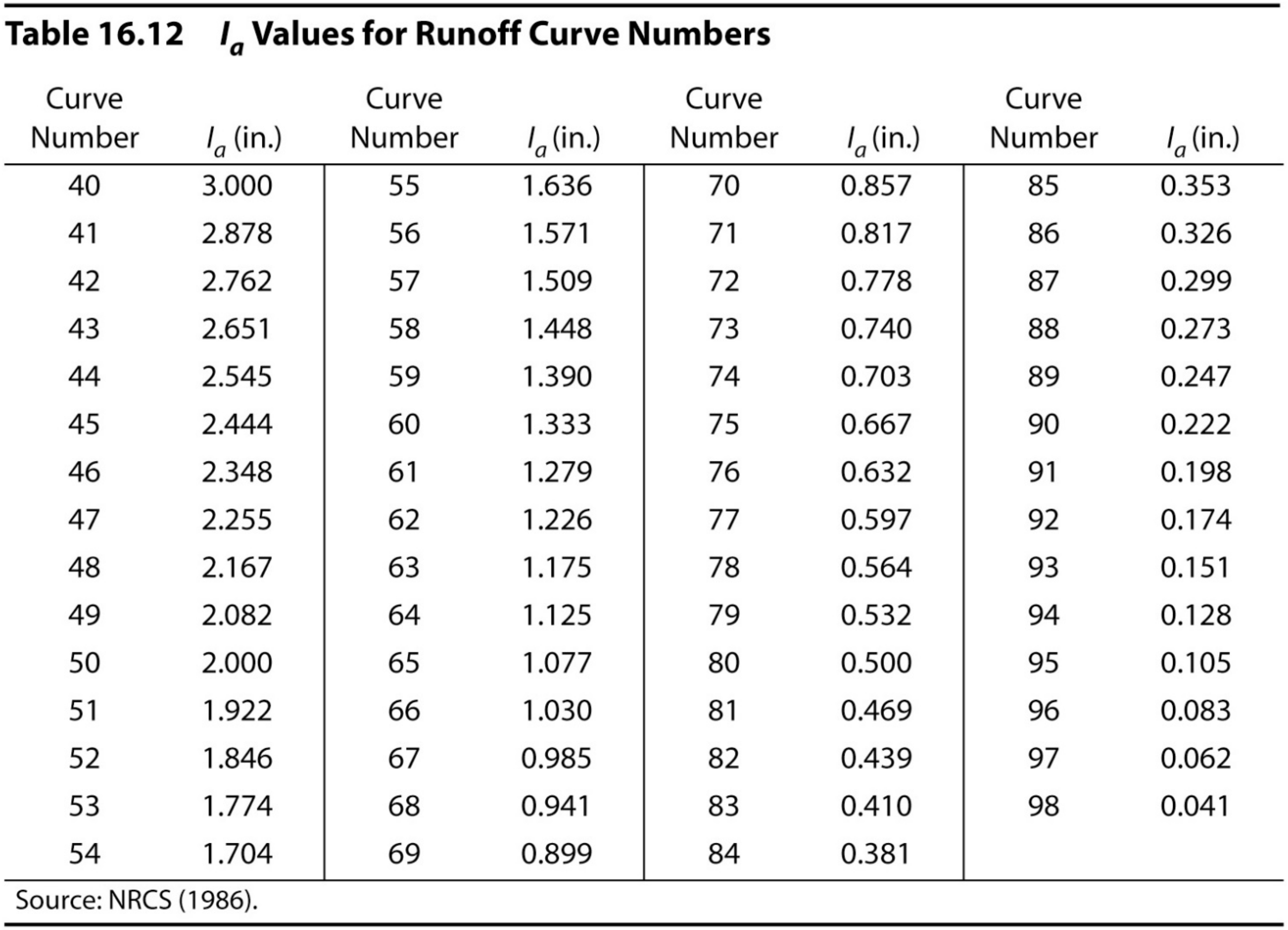


Table 16.12 I_a Values for Runoff Curve Numbers

Curve Number	I_a (in.)	Curve Number	I_a (in.)	Curve Number	I_a (in.)	Curve Number	I_a (in.)
40	3.000	55	1.636	70	0.857	85	0.353
41	2.878	56	1.571	71	0.817	86	0.326
42	2.762	57	1.509	72	0.778	87	0.299
43	2.651	58	1.448	73	0.740	88	0.273
44	2.545	59	1.390	74	0.703	89	0.247
45	2.444	60	1.333	75	0.667	90	0.222
46	2.348	61	1.279	76	0.632	91	0.198
47	2.255	62	1.226	77	0.597	92	0.174
48	2.167	63	1.175	78	0.564	93	0.151
49	2.082	64	1.125	79	0.532	94	0.128
50	2.000	65	1.077	80	0.500	95	0.105
51	1.922	66	1.030	81	0.469	96	0.083
52	1.846	67	0.985	82	0.439	97	0.062
53	1.774	68	0.941	83	0.410	98	0.041
54	1.704	69	0.899	84	0.381		

Source: NRCS (1986).

Example 1 NRCS(SCS) TR-55 Method

- Example 16.11 in pp. 727-728

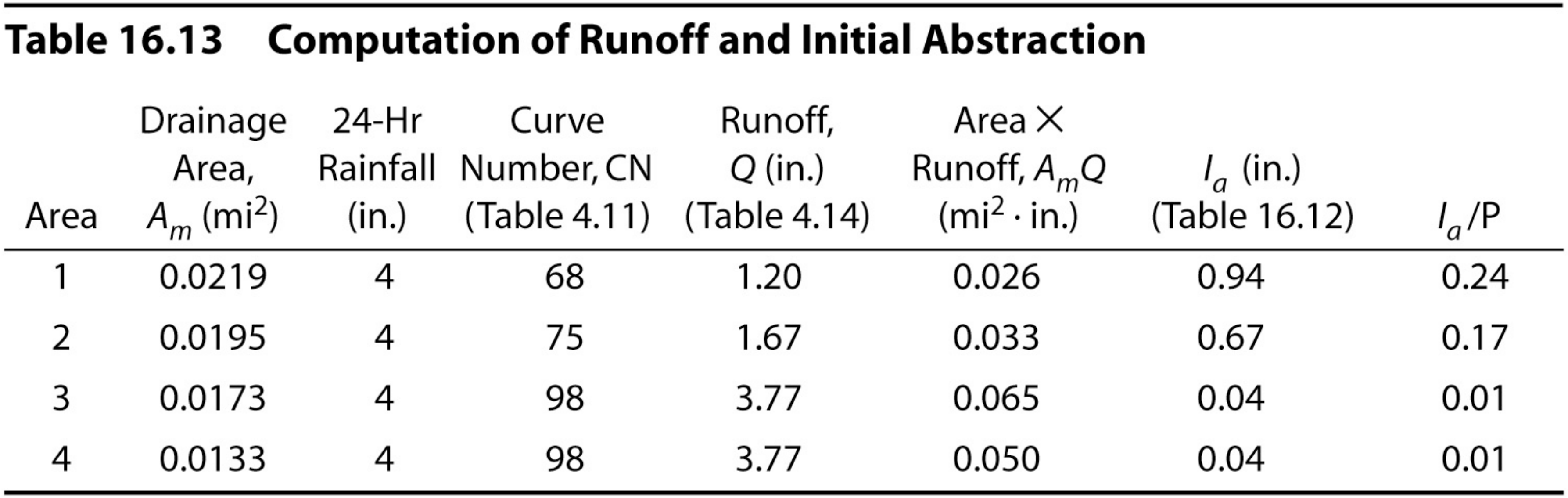


Table 16.13 Computation of Runoff and Initial Abstraction

Area	Drainage Area, A_m (mi^2)	24-Hr Rainfall (in.)	Curve Number, CN (Table 4.11)	Runoff, Q (in.) (Table 4.14)	Area × Runoff, A_mQ (mi^2 · in.)	I_a (in.) (Table 16.12)	I_a/P
1	0.0219	4	68	1.20	0.026	0.94	0.24
2	0.0195	4	75	1.67	0.033	0.67	0.17
3	0.0173	4	98	3.77	0.065	0.04	0.01
4	0.0133	4	98	3.77	0.050	0.04	0.01

Example 1 NRCS (SCS) TR-55 Method (Cont.)

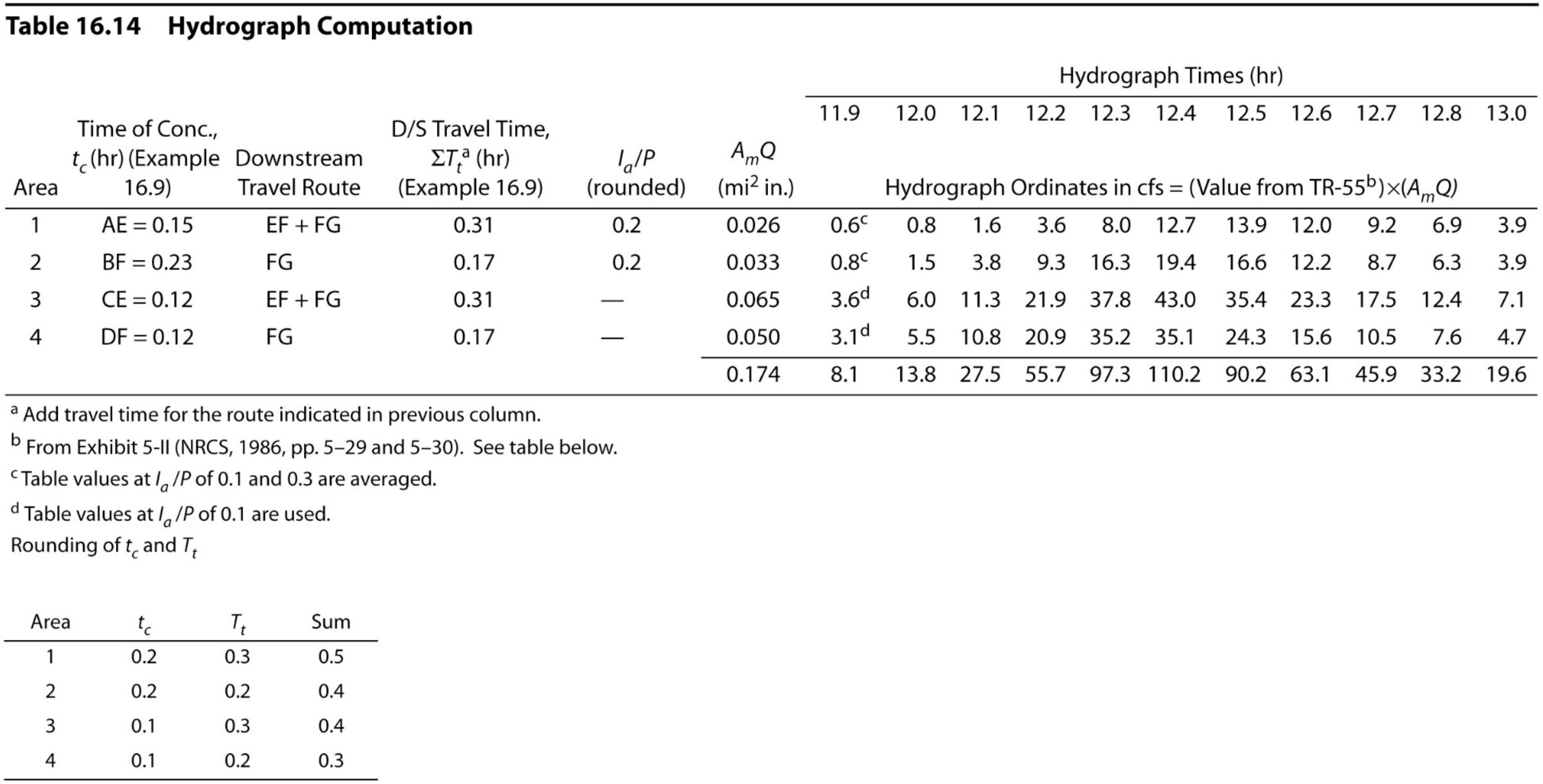


Table 16.14 Hydrograph Computation

Area	Time of Conc., t_c (hr) (Example 16.9)	Downstream Travel Route	D/S Travel Time, ΣT_t[a] (hr) (Example 16.9)	I_a/P (rounded)	A_mQ (mi^2 in.)	Hydrograph Times (hr)										
						11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0
						Hydrograph Ordinates in cfs = (Value from TR-55[b])×(A_mQ)										
1	AE = 0.15	EF + FG	0.31	0.2	0.026	0.6[c]	0.8	1.6	3.6	8.0	12.7	13.9	12.0	9.2	6.9	3.9
2	BF = 0.23	FG	0.17	0.2	0.033	0.8[c]	1.5	3.8	9.3	16.3	19.4	16.6	12.2	8.7	6.3	3.9
3	CE = 0.12	EF + FG	0.31	—	0.065	3.6[d]	6.0	11.3	21.9	37.8	43.0	35.4	23.3	17.5	12.4	7.1
4	DF = 0.12	FG	0.17	—	0.050	3.1[d]	5.5	10.8	20.9	35.2	35.1	24.3	15.6	10.5	7.6	4.7
					0.174	8.1	13.8	27.5	55.7	97.3	110.2	90.2	63.1	45.9	33.2	19.6

[a] Add travel time for the route indicated in previous column.

[b] From Exhibit 5-II (NRCS, 1986, pp. 5–29 and 5–30). See table below.

[c] Table values at I_a/P of 0.1 and 0.3 are averaged.

[d] Table values at I_a/P of 0.1 are used.

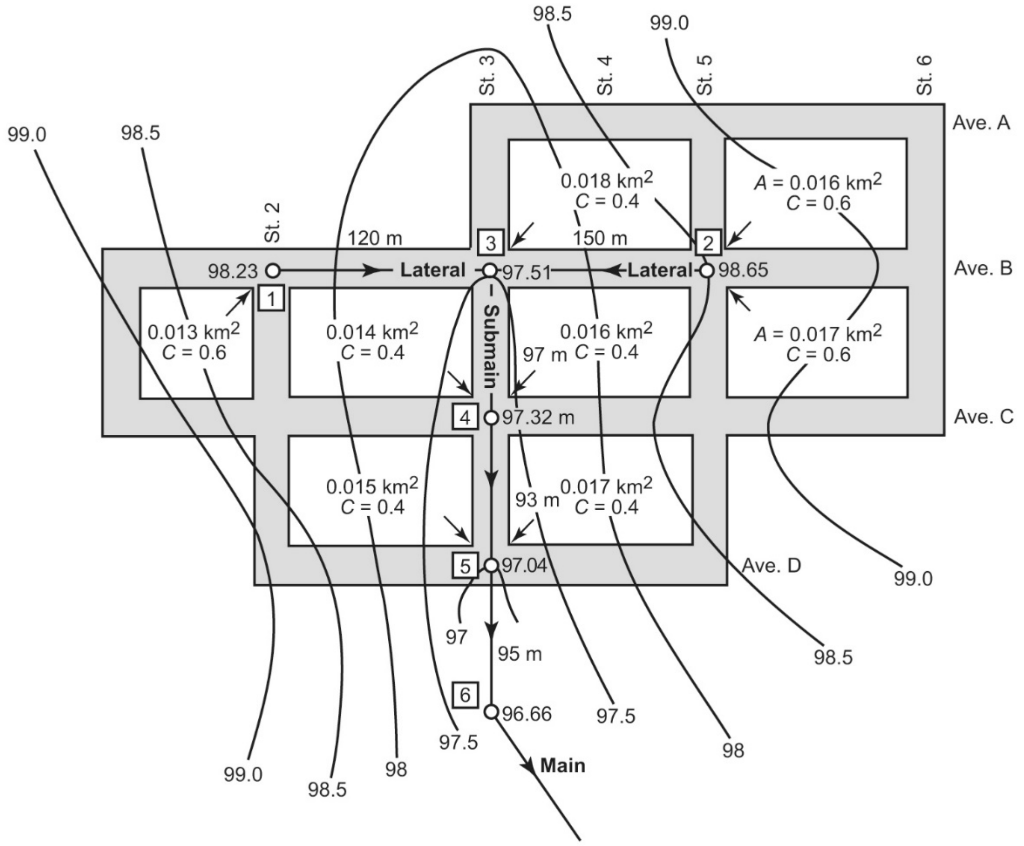
Rounding of t_c and T_t

Area	t_c	T_t	Sum
1	0.2	0.3	0.5
2	0.2	0.2	0.4
3	0.1	0.3	0.4
4	0.1	0.2	0.3

Example 2 NRCS (SCS) TR-55 Method

- Example 16.12 in pp. 729-733

Figure 16.12 Storm drains layout for a section of a city.



Example 2 NRCS (SCS) TR-55 Method (Cont.)

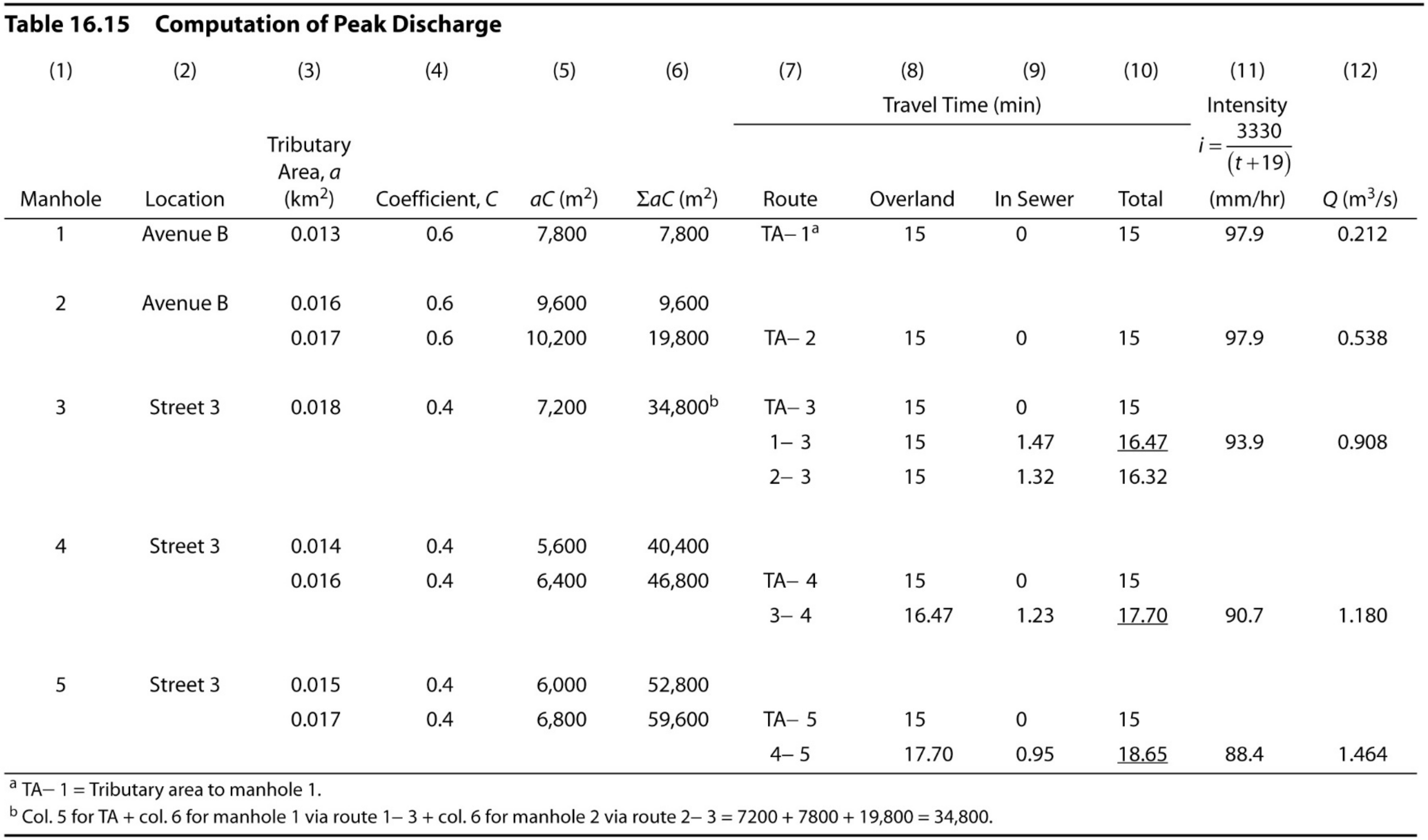


Table 16.15 Computation of Peak Discharge

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
						Travel Time (min)				Intensity $i=\frac{3330}{(t+19)}$	
Manhole	Location	Tributary Area, *a* (km^2)	Coefficient, *C*	*aC* (m^2)	Σ*aC* (m^2)	Route	Overland	In Sewer	Total	(mm/hr)	*Q* (m^3/s)
1	Avenue B	0.013	0.6	7,800	7,800	TA– 1[a]	15	0	15	97.9	0.212
2	Avenue B	0.016	0.6	9,600	9,600						
		0.017	0.6	10,200	19,800	TA– 2	15	0	15	97.9	0.538
3	Street 3	0.018	0.4	7,200	34,800[b]	TA– 3	15	0	15		
						1– 3	15	1.47	16.47	93.9	0.908
						2– 3	15	1.32	16.32		
4	Street 3	0.014	0.4	5,600	40,400						
		0.016	0.4	6,400	46,800	TA– 4	15	0	15		
						3– 4	16.47	1.23	17.70	90.7	1.180
5	Street 3	0.015	0.4	6,000	52,800						
		0.017	0.4	6,800	59,600	TA– 5	15	0	15		
						4– 5	17.70	0.95	18.65	88.4	1.464

[a] TA– 1 = Tributary area to manhole 1.

[b] Col. 5 for TA + col. 6 for manhole 1 via route 1– 3 + col. 6 for manhole 2 via route 2– 3 = 7200 + 7800 + 19,800 = 34,800.

Example 2 NRCS (SCS) TR-55 Method (Cont.)

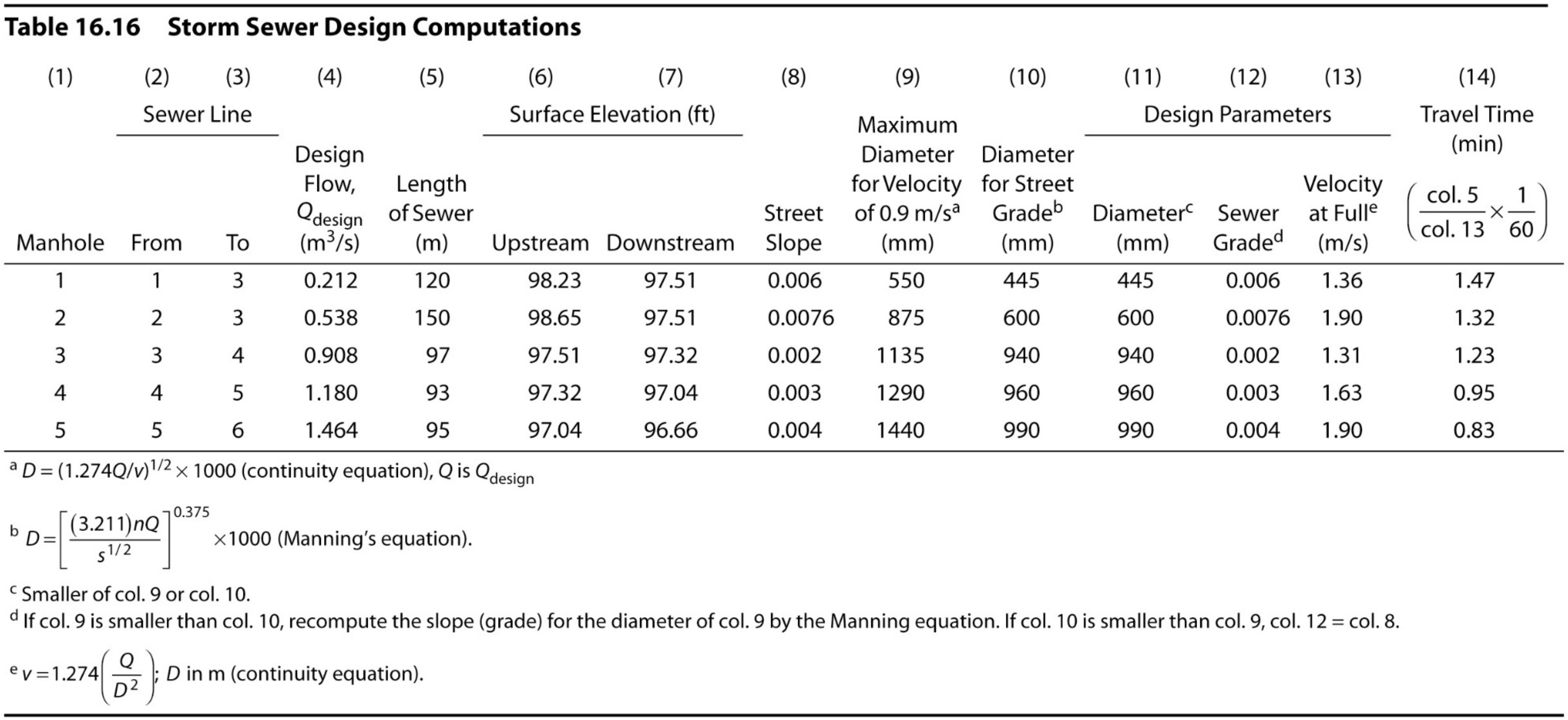


Table 16.16 Storm Sewer Design Computations

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
	Sewer Line				Surface Elevation (ft)					Design Parameters			Travel Time (min)
Manhole	From	To	Design Flow, Q_{design} (m^3/s)	Length of Sewer (m)	Upstream	Downstream	Street Slope	Maximum Diameter for Velocity of 0.9 m/s[a] (mm)	Diameter for Street Grade[b] (mm)	Diameter[c] (mm)	Sewer Grade[d]	Velocity at Full[e] (m/s)	$\left(\frac{\text{col. 5}}{\text{col. 13}} \times \frac{1}{60}\right)$
1	1	3	0.212	120	98.23	97.51	0.006	550	445	445	0.006	1.36	1.47
2	2	3	0.538	150	98.65	97.51	0.0076	875	600	600	0.0076	1.90	1.32
3	3	4	0.908	97	97.51	97.32	0.002	1135	940	940	0.002	1.31	1.23
4	4	5	1.180	93	97.32	97.04	0.003	1290	960	960	0.003	1.63	0.95
5	5	6	1.464	95	97.04	96.66	0.004	1440	990	990	0.004	1.90	0.83

[a] $D = (1.274Q/v)^{1/2} \times 1000$ (continuity equation), Q is Q_{design}

[b] $D = \left[\frac{(3.211)nQ}{s^{1/2}}\right]^{0.375} \times 1000$ (Manning's equation).

[c] Smaller of col. 9 or col. 10.

[d] If col. 9 is smaller than col. 10, recompute the slope (grade) for the diameter of col. 9 by the Manning equation. If col. 10 is smaller than col. 9, col. 12 = col. 8.

[e] $v = 1.274\left(\frac{Q}{D^2}\right)$; D in m (continuity equation).

Detention Basin Storage Capacity

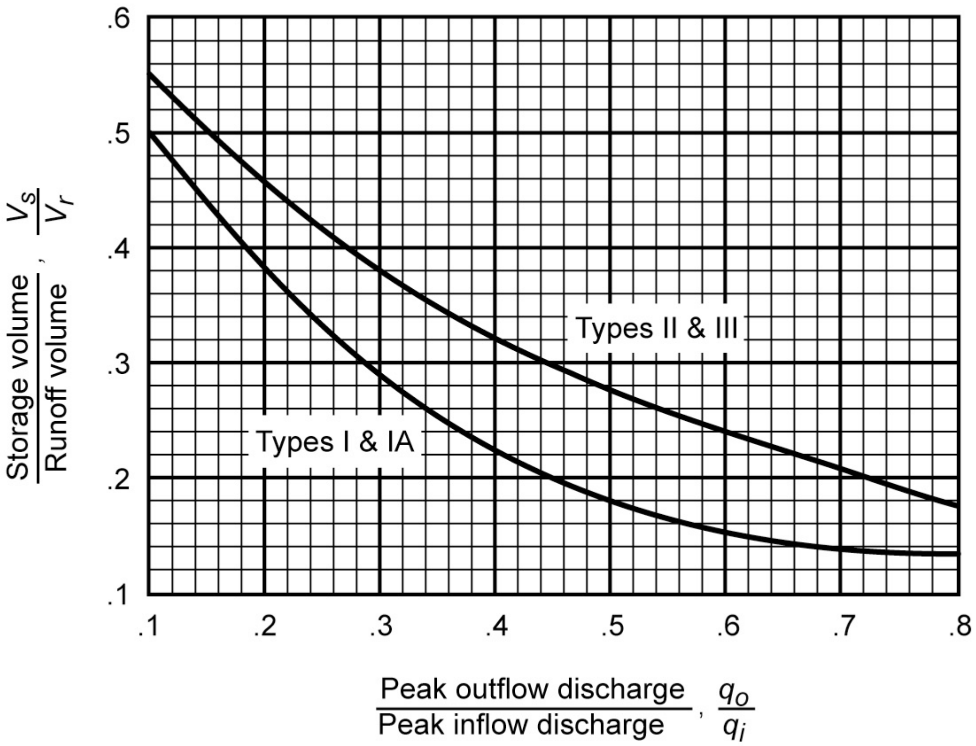
- *Objective:* To reduce the peak flow and volume of a hydrograph.
- Practical: Local governments ordinances requiring that the post development discharge not exceed the predevelopment discharge.
- *Solution (among others):* Detention basin that is designed to reduce the peak flow, store all or part of the volume and then release it at a controlled outflow discharge.

Detention Basin Sizes

- By TR-55 Based Procedure
  - See Figure 16.13: Detention Basin Storage Volume (NRCS, 1986)
- Rational-Method Based Procedure
  - Eq. 16.13: $V_{in} = i \sum a\, C\, T$ (L^3) and
  - Eq. 16.14: $V_{out} = Q_o T$ (L^3)
  - where
    - I = IDF rainfall intensity;
    - T = storm duration and
    - Q_o = maximum outflow rate

NRCS (1986) Detention Basin Sizing

Figure 16.13 Detention basin storage volume (from NRCS, 1986).



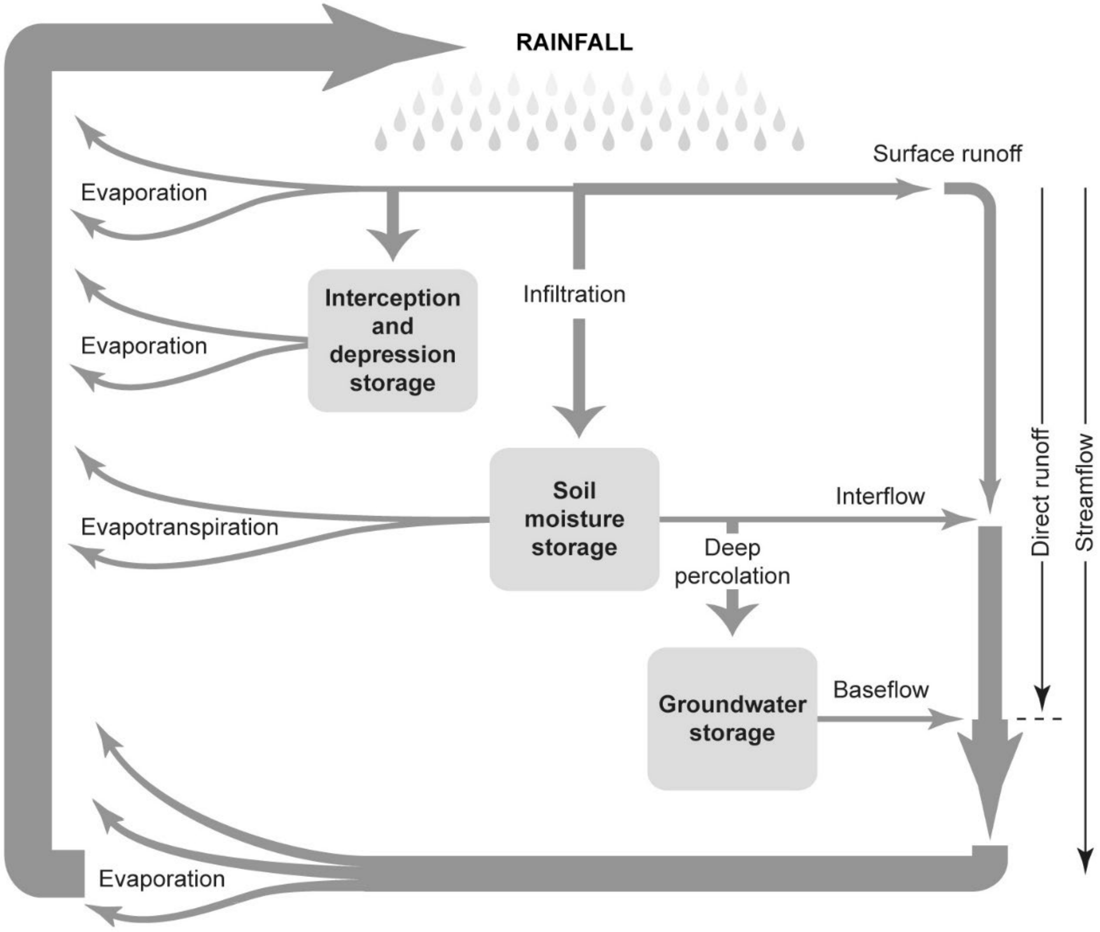
UNIT HYDROGRAPH METHOD

Unit Hydrograph Method

- *Objective:* to construct storm or streamflow hydrographs
- *Definition:* Hydrograph that results from 1 unit (e.g., 1 in, 1 cm, 1 foot, etc.) of precipitation excess applied instantly over a basin
- *Derivation Approaches:*
  - Directly from a storm hydrograph recorded in the basin for a particular duration of a precipitation event
  - Use of a synthetic unit hydrograph

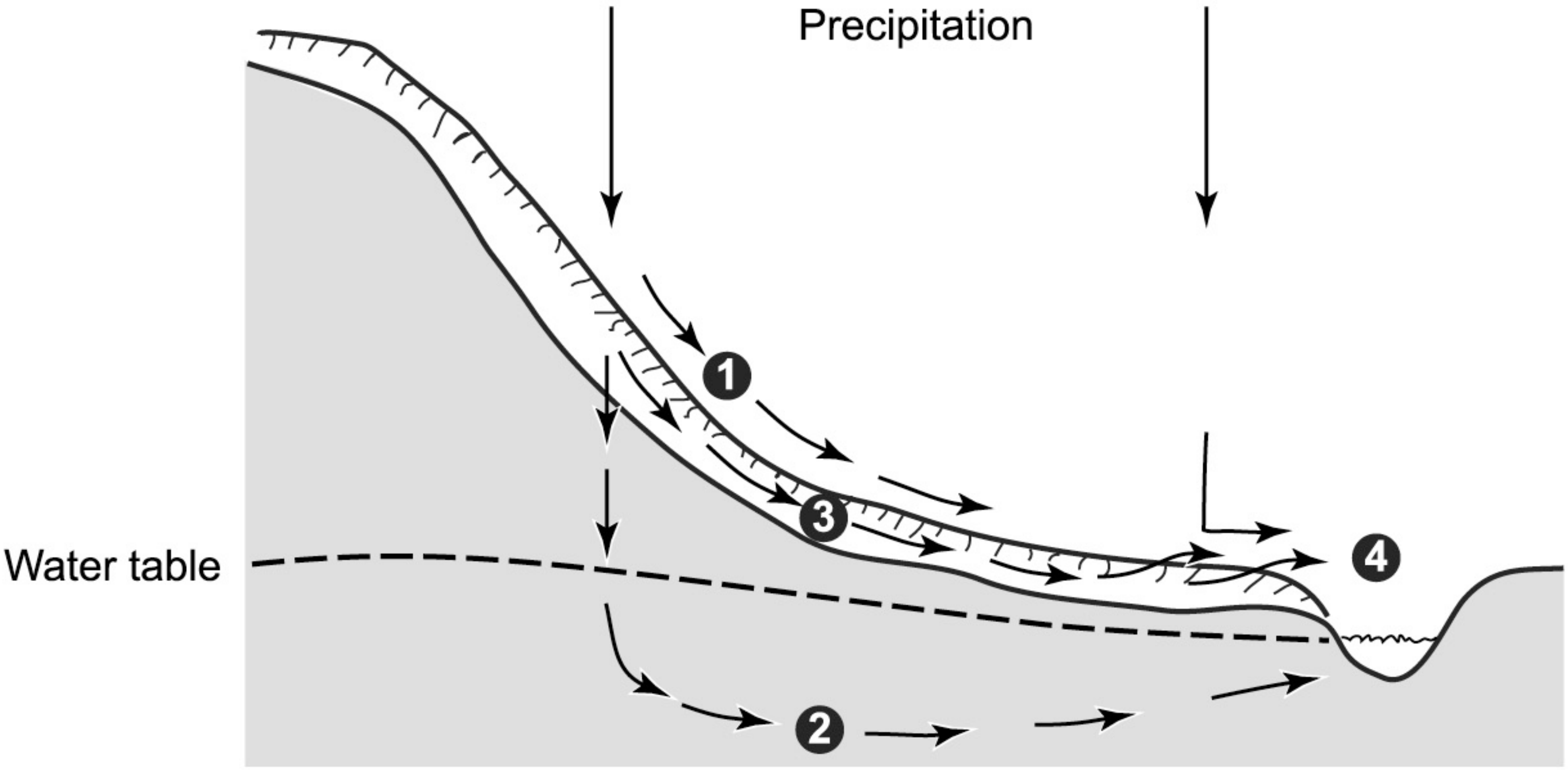
Runoff in the Hydrologic Cycle

Figure 9.1 Forms of runoff in the hydrologic cycle.



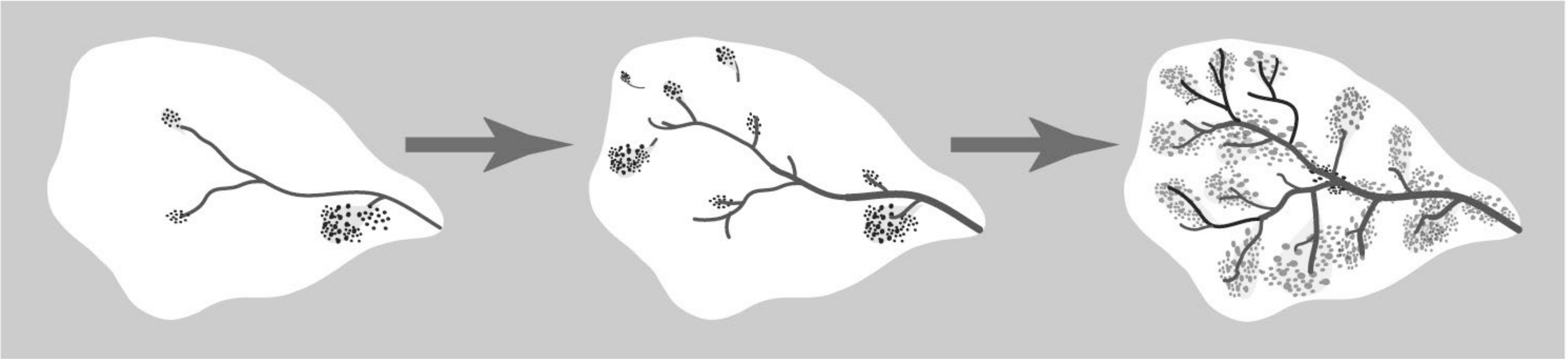
Runoff Paths

Figure 9.2 Paths of runoff (after Dunne, 1982).



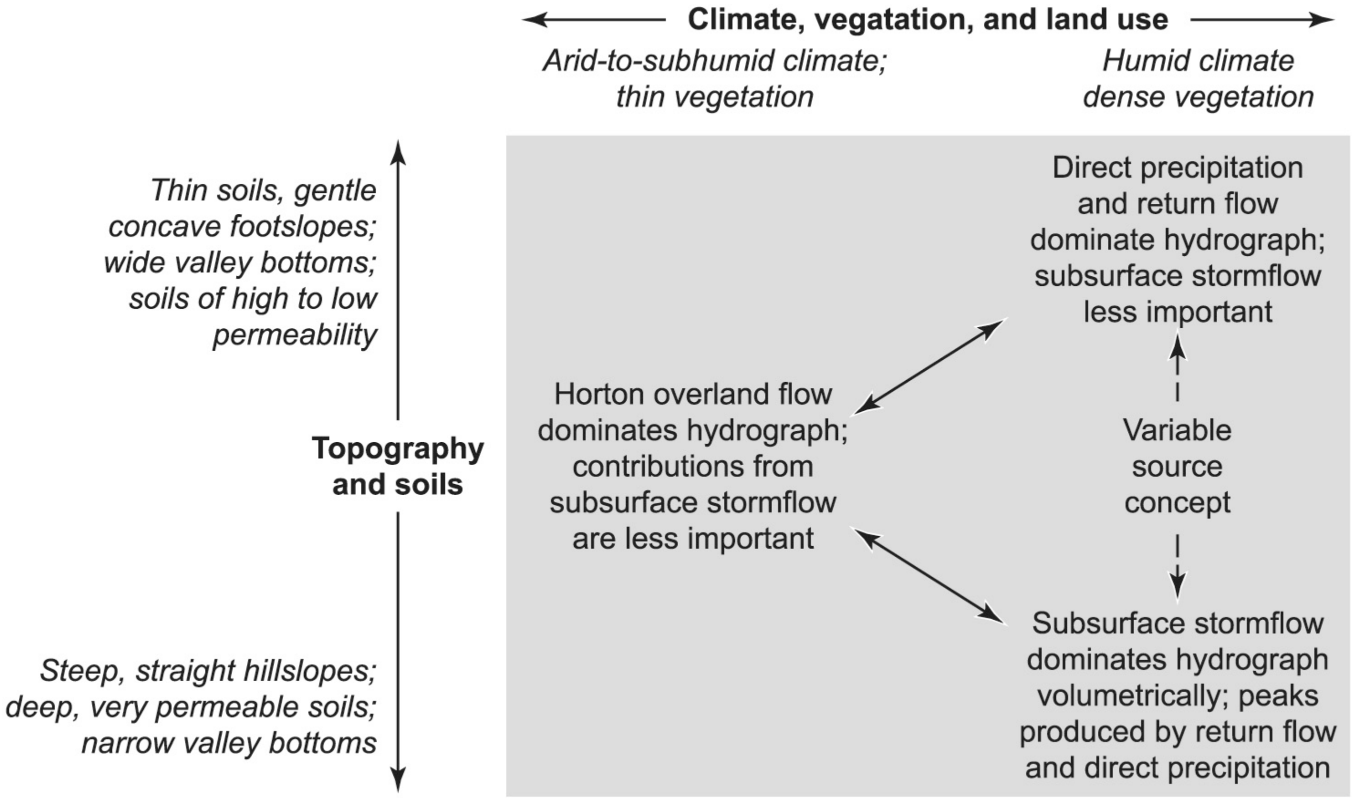
Expansion of Source Area

Figure 9.3 Expansion of source area.



Conditions Controlling Runoff

Figure 9.4 Conditions controlling the runoff mechanism (after Dunne, 1982).



Techniques to Estimate Streamflow

- Hydrograph Analysis: Rainfall-Runoff model
- (e.g., Unit Hydrograph)
- Correlation with Meteorological Data (e.g., statistical techniques and probability theory)
- Correlation with Hydrological Data at Another Site (i.e., correlating data from one site to a neighboring one)
- Sequential Data Generation (i.e., stochastic process)
- Ungaged Sites (e.g., regional regression data)

Data Situation and Estimation Techniques

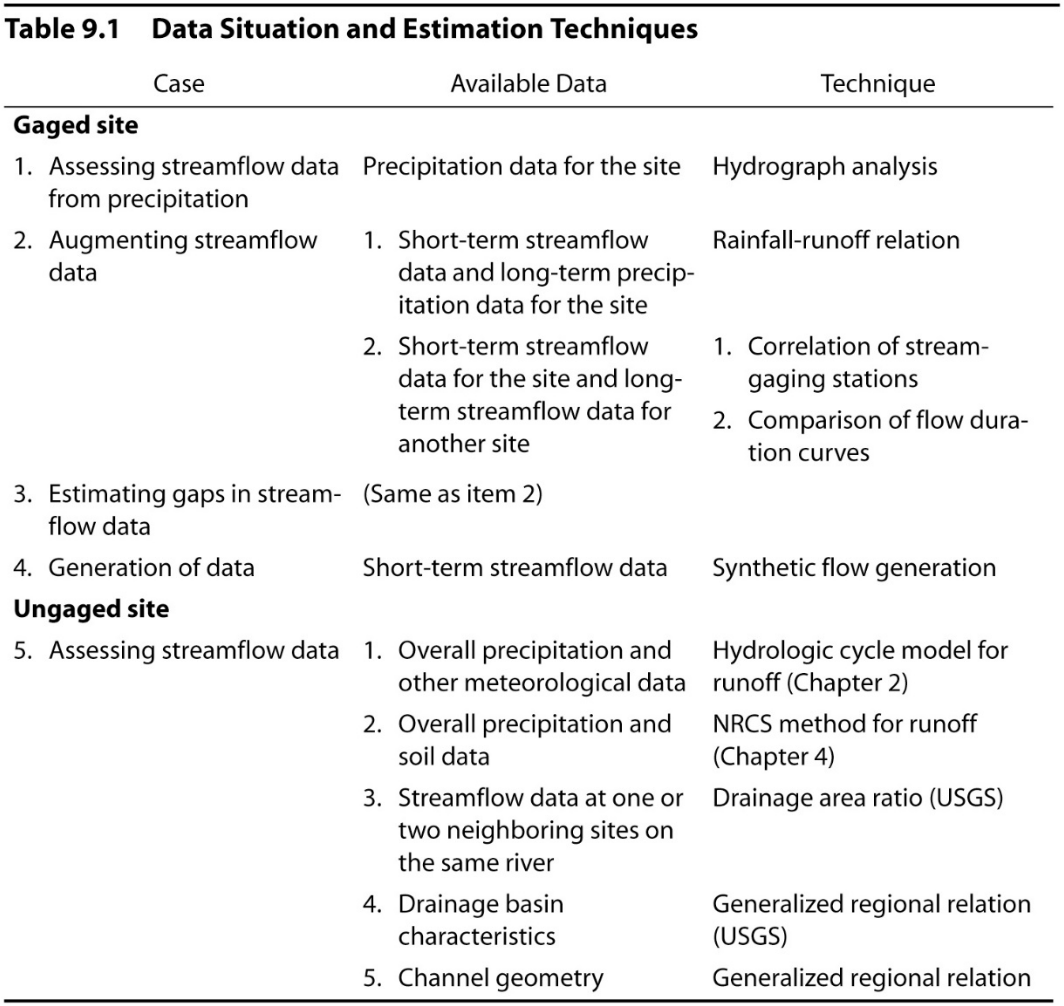
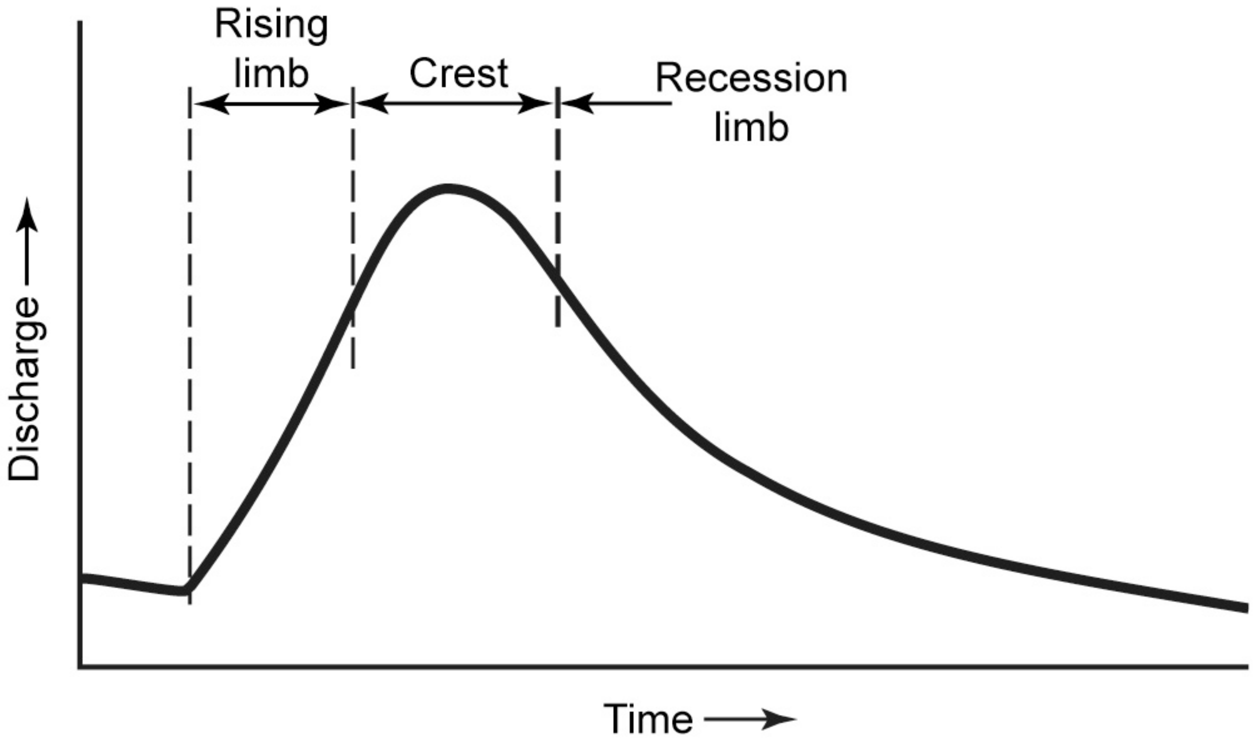


Table 9.1 Data Situation and Estimation Techniques

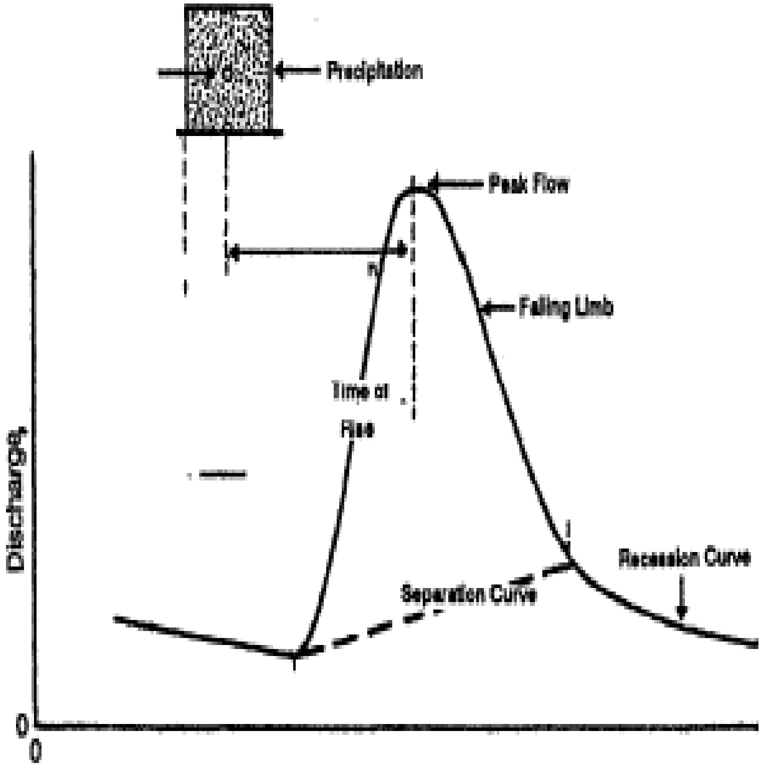
Case	Available Data	Technique
Gaged site		
1. Assessing streamflow data from precipitation	Precipitation data for the site	Hydrograph analysis
2. Augmenting streamflow data	1. Short-term streamflow data and long-term precipitation data for the site	Rainfall-runoff relation
	2. Short-term streamflow data for the site and long-term streamflow data for another site	1. Correlation of stream-gaging stations 2. Comparison of flow duration curves
3. Estimating gaps in streamflow data	(Same as item 2)	
4. Generation of data	Short-term streamflow data	Synthetic flow generation
Ungaged site		
5. Assessing streamflow data	1. Overall precipitation and other meteorological data	Hydrologic cycle model for runoff (Chapter 2)
	2. Overall precipitation and soil data	NRCS method for runoff (Chapter 4)
	3. Streamflow data at one or two neighboring sites on the same river	Drainage area ratio (USGS)
	4. Drainage basin characteristics	Generalized regional relation (USGS)
	5. Channel geometry	Generalized regional relation

Sketch of Storm Hydrograh

Figure 9.5 Simple storm hydrograph.

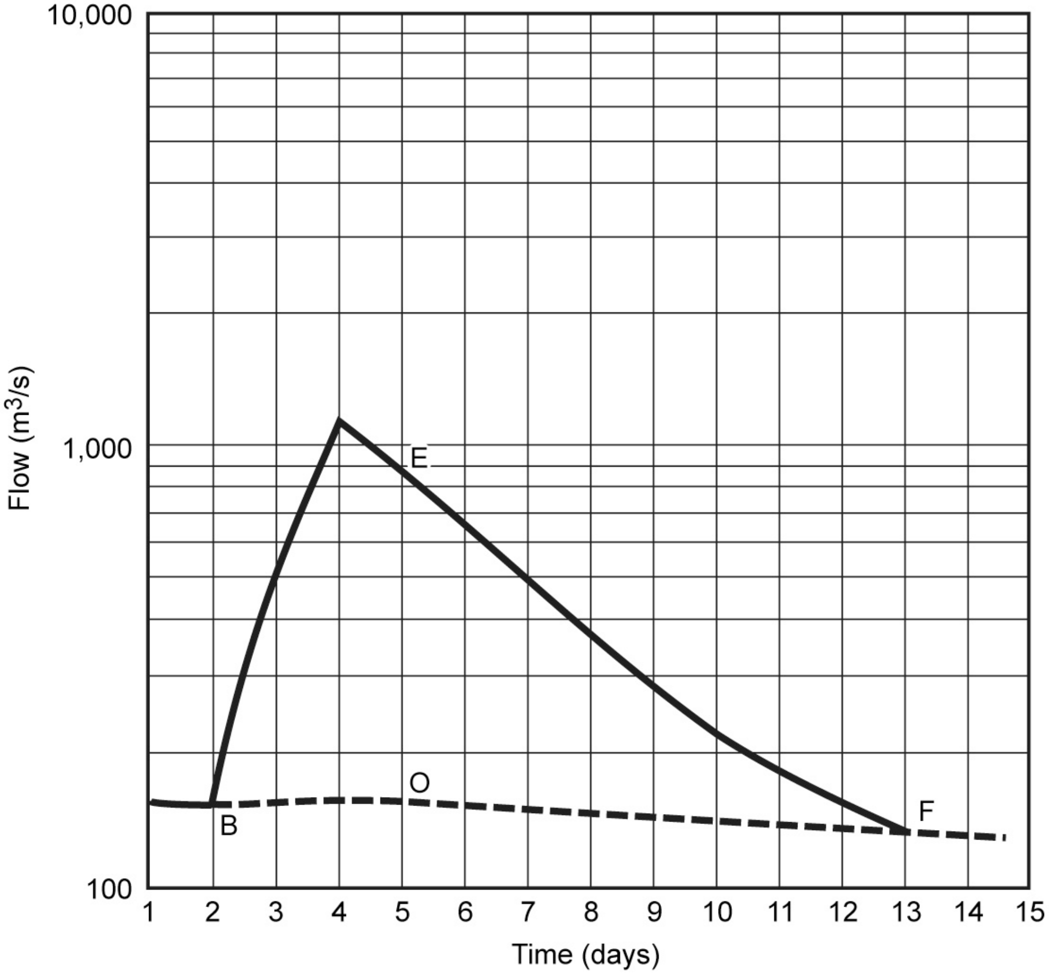


Typical Storm Hydrograph



Baseflow Separation (by Recession Curve)

Figure 9.6 Baseflow separation by the recession curve approach.



Methods of Baseflow Separation

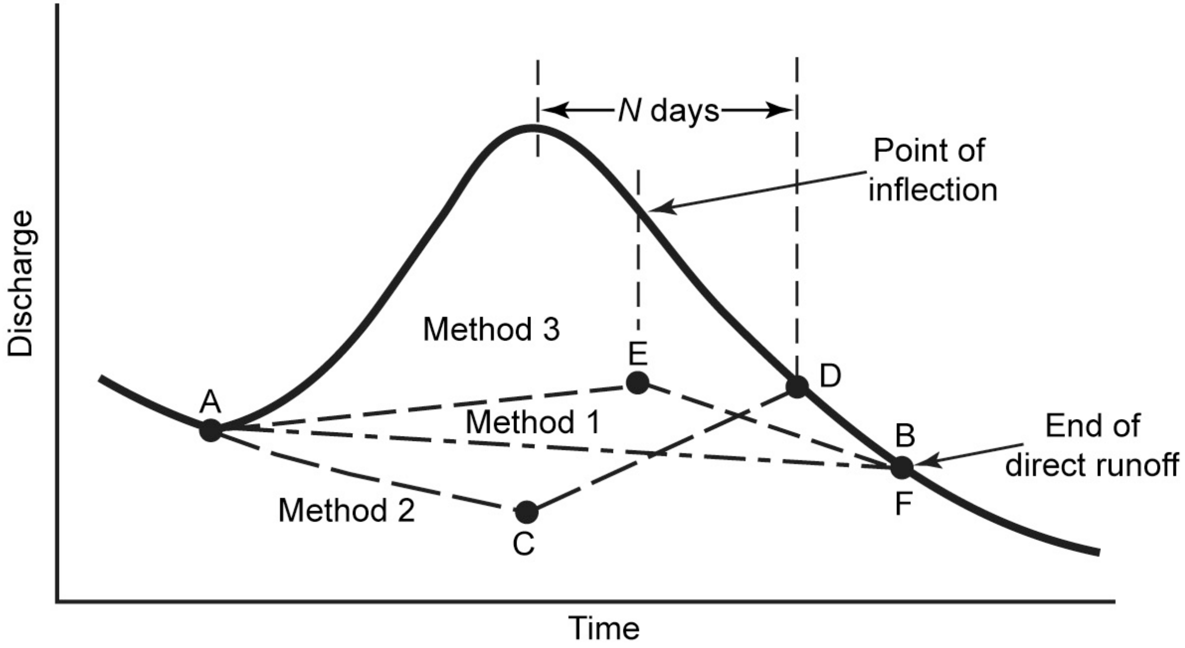


Figure 9.7 Methods of baseflow separation.

Deconvolution

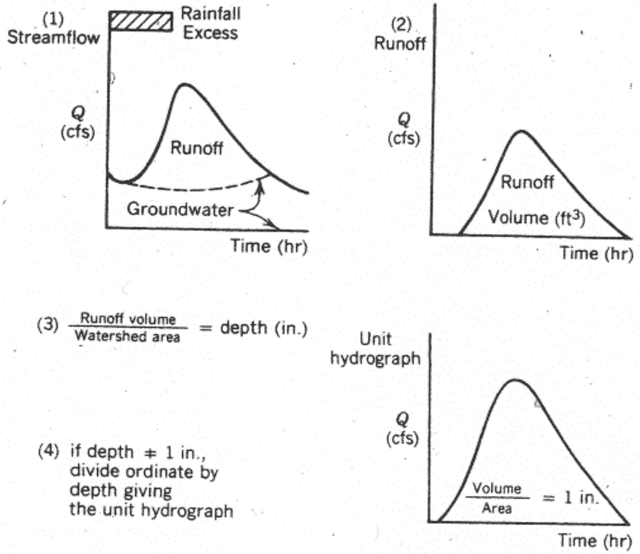


FIGURE 6.11 The steps for calculating a unit hydrograph.

SOURCE : WANIELISTA, M. (1997)

Convolution

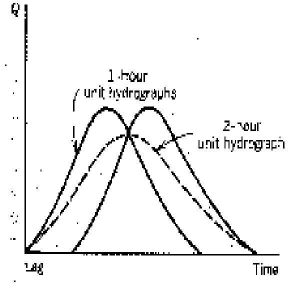


FIGURE 6.12 The example lagging procedure.

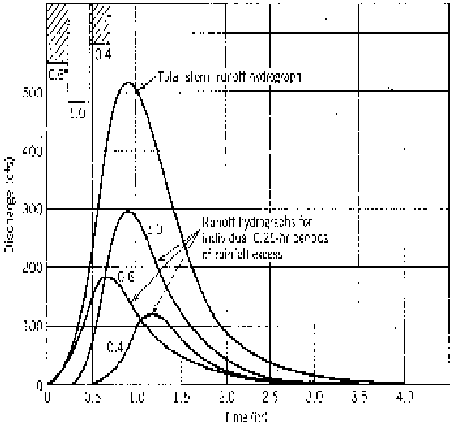
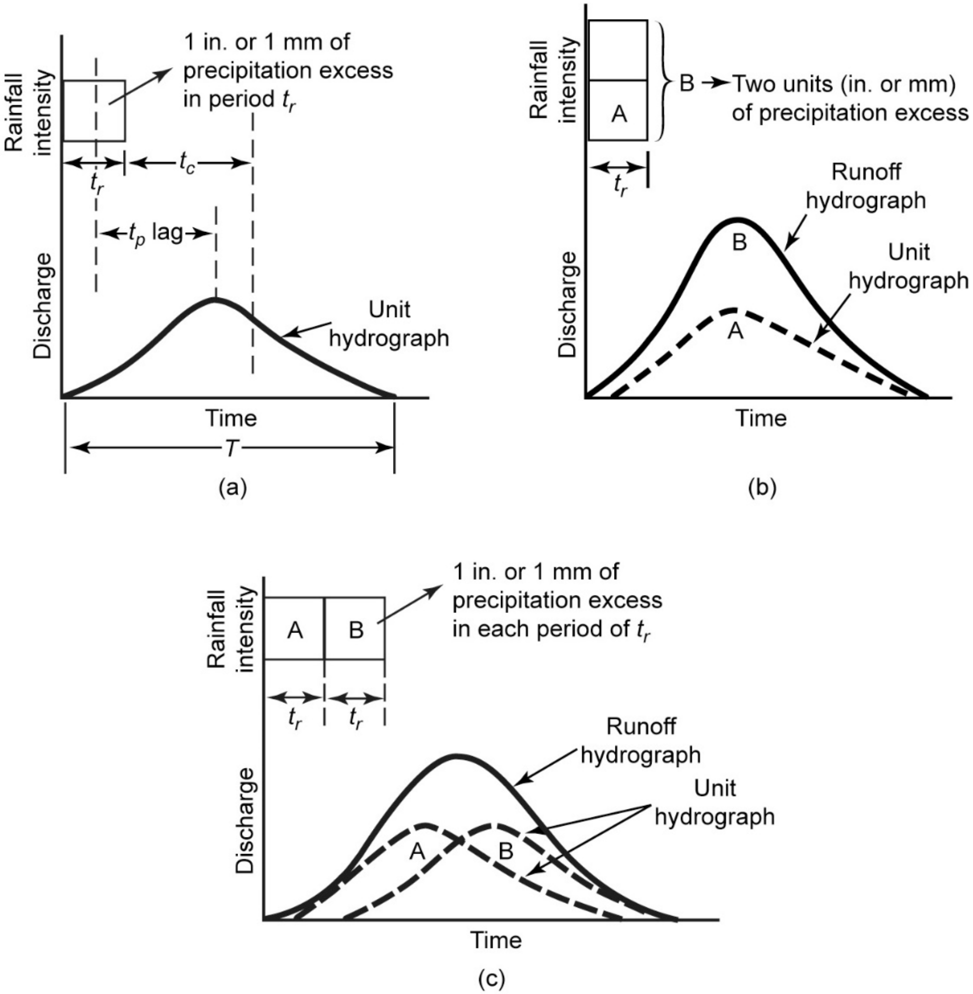


FIGURE 6.13 Combining hydrographs.

SOURCE: WANIELISTA, M. (1997)

UH Basic Principles

Figure 9.9 Principles of the unit hydrograph: (a) unit hydrograph; (b) runoff hydrograph for two units of precipitation of duration t_r; (c) runoff hydrograph from unit precipitation for two consecutive periods of duration t_r.

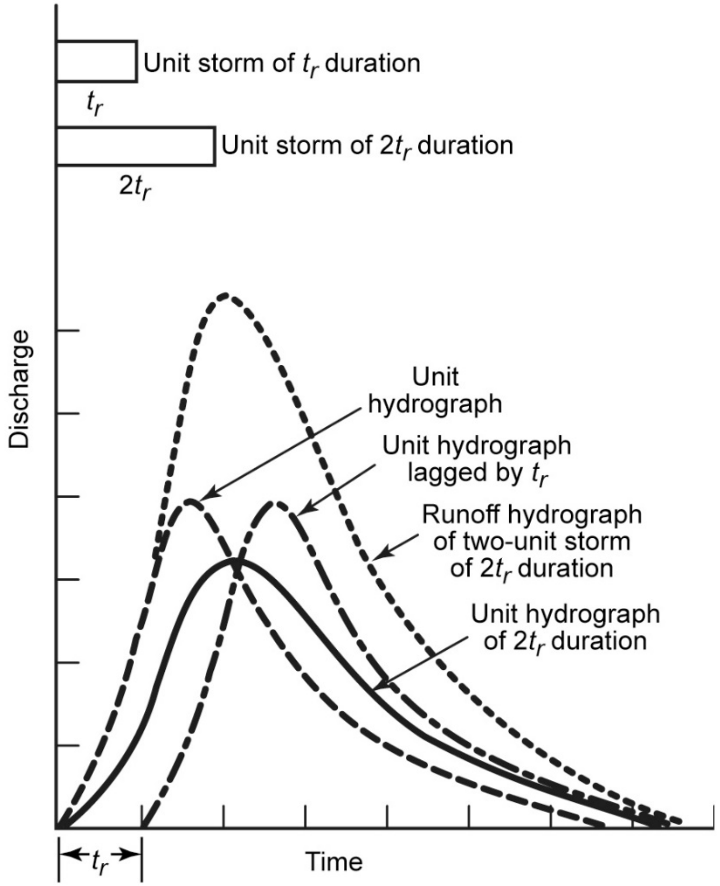


Changing the UH Duration

- Lagging method
- S-curve method

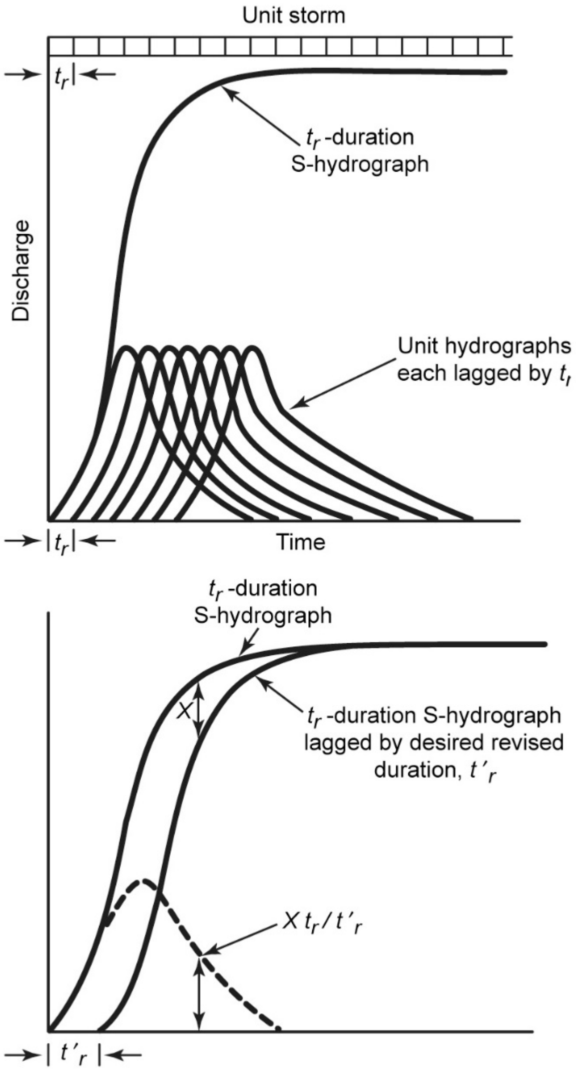
Lagging Method

Figure 9.15 Lagging procedure to convert unit hydrograph duration.



S-Curve method

Figure 9.16 Illustration of the S-curve.



Synthetic UHs

- Snyder’s Method
- NRCS Method
- Others