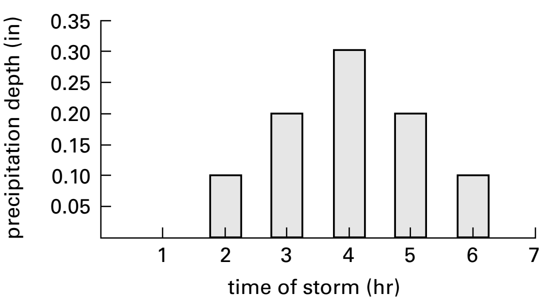
Table 1.5 Precipitation Depth and Intensity Data for a Typical Storm

time interval, t (hr)	incr. depth, d_{incr} (in)	cum. depth, d_{cum} (in)	incr. intensity, i_{incr} (in/hr)	cum. intensity, i_{cum} (in/hr)
0	0.0	0.0	0.0	0.0
1	0.0	0.0	0.0	0.0
2	0.1	0.1	0.1	0.1
3	0.2	0.3	0.2	0.15
4	0.3	0.6	0.3	0.2
5	0.2	0.8	0.2	0.2
6	0.1	0.9	0.1	0.18

(Multiply in by 2.54 to obtain cm.)
(Multiply in/hr by 2.54 to obtain cm/h.)

Figure 1.8 plots a storm hyetograph from the data in Table 1.5. The hyetograph relates incremental precipitation depth on the vertical axis to time on the horizontal axis.

Figure 1.8 *Typical Storm Hyetograph*



Example 1.5

From the storm hyetograph depicted in Fig. 1.8, what is the maximum rainfall intensity? In what time interval does it occur?

Solution

The maximum rainfall intensity is 0.3 in/hr, occurring in hour 4 of the storm.

Theissen Network

Over large areas or watersheds, precipitation can vary depending on elevation, proximity to large waterways, and other factors. If multiple precipitation gauges are available, the precipitation at each station can be weighted in proportion to the area each station represents by using the *Theissen network* (see Fig. 1.9). The Theissen network is constructed according to the following steps.

step 1: Plot the location of each precipitation gauge, or station. Identify each gauge as A, B, C, and so on.

step 2: Connect each station to the two nearest stations with straight lines.

step 3: Delineate perpendicular bisecting lines halfway between adjacent gauges.

step 4: Define a series of polygons with the perpendicular bisecting lines. (For instance, in Fig. 1.9, there are Theissen polygons A, B, C, and D.)

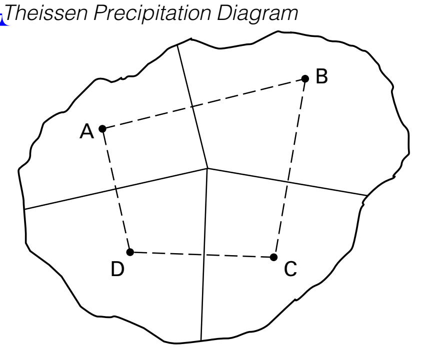
step 5: Calculate the area of each Theissen polygon, then sum their areas.

step 6: Multiply the Theissen area by the precipitation depth at each station to determine the product (the volume of the precipitation).

step 7: Sum the products (precipitation volumes).

step 8: Determine the average precipitation depth per unit area in the watershed by dividing the sum of the products (precipitation volumes) by the sum of the Theissen areas.

Figure ~~1.8~~ *Theissen Precipitation Diagram*



Example 1.6

For the watershed depicted in Fig. 1.9, the precipitation is 4.0 in at gauge A, 5.0 in at gauge B, 3.5 in at gauge C, and 4.5 in at gauge D. The area of Theissen polygon A is 10 mi^2, B is 18 mi^2, C is 15 mi^2, and D is 11 mi^2. Calculate the average precipitation, P, per square mile of the watershed.

Solution

Construct the following table. The weighted precipitation is the Theissen area (column 2) multiplied by the precipitation (column 3).

station	Theissen area (mi^2)	precipitation, P (in)	weighted precipitation (mi^2-in)
A	10	4.0	40.0
B	18	5.0	90.0
C	15	3.5	52.5
D	11	4.5	49.5
sum	54	17.0	232

Referring to the following table, column 3 defines the unit hydrograph for a 0.5 mi^2 watershed area. The unit hydrograph is calculated by dividing each hydrograph runoff point in column 2 by the average precipitation, $P_{\text{ave,excess}}$, which is 1.18 in.

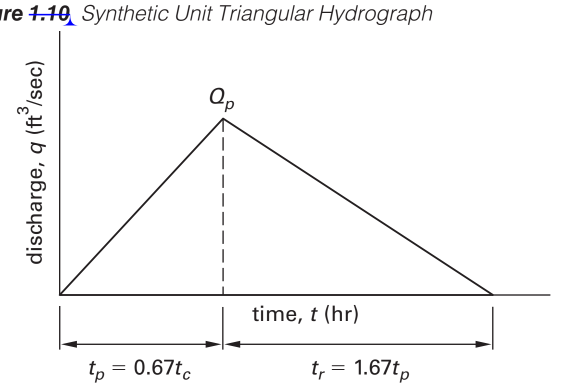
Column 4 defines the hydrograph coordinates for a 2.0 in storm in the watershed, which are calculated by multiplying the unit hydrograph runoff calculated in column 3 by the storm precipitation, 2.0 in.

time (hr)	1.18 in hydrograph runoff (ft^3/sec)	unit hydrograph runoff (ft^3/sec-in)	2.0 in hydrograph runoff (ft^3/sec)
12.7	50	$50/1.18 = 42.4$	$(42.4)(2.0) = 84.7$
14	155	$155/1.18 = 131.4$	$(131.4)(2.0) = 262.7$
15.4	250	$250/1.18 = 211.9$	$(211.9)(2.0) = 423.7$
16	137	$137/1.18 = 116.1$	$(116.2)(2.0) = 232.2$
16.5	50	$50/1.18 = 42.4$	$(42.4)(2.0) = 84.7$

Synthetic Unit Triangular Hydrograph

In ungauged watersheds, *synthetic hydrographs* are used to design reservoirs and detention ponds and can be estimated using the *NRCS synthetic unit triangular hydrograph method* (see Fig. 1.11). The coordinates of the synthetic hydrograph are defined by the peak runoff, Q_p, time to peak runoff, t_p, and time of concentration, t_c.

Figure ~~1.10~~ *Synthetic Unit Triangular Hydrograph*



The *peak runoff* is the maximum rate of runoff flow. (See Sec. 1.11 for how to calculate peak runoff.) The time of concentration is the maximum time for runoff to travel from the farthest and uppermost point in a watershed downstream to the outlet point of the watershed. The storm duration is equal to the time of concentration for small watersheds (1 mi^2). The ascending leg of the hydrograph is the *time to peak runoff*, t_p, as measured from the beginning of overland runoff hydrograph as defined in Fig. 1.11 and Eq. 1.6.

$$t_p = 0.67t_c \qquad 1.6$$

The receding leg of the hydrograph is measured after the time of peak runoff using Eq. 1.7.

$$t_r = 1.67t_p \qquad 1.7$$

Example 1.8

Define a synthetic unit triangular hydrograph where the peak runoff is estimated as 100 ft^3/sec and the time of concentration is 1.5 hr.

Solution

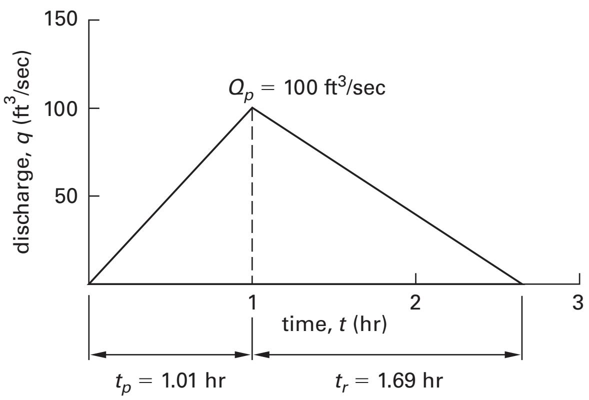
Plot the peak runoff at 100 ft^3/sec. Use Eq. 1.6 to define the time for the ascending limb of the hydrograph as t_p, the time to peak runoff.

$$\begin{aligned} t_p &= 0.67t_c \\ &= (0.67)(1.5\text{ hr}) \\ &= 1.01\text{ hr} \end{aligned}$$

Using Eq. 1.7, define the time of the receding limb of the hydrograph.

$$\begin{aligned} t_r &= 1.67t_p \\ &= (1.67)(1.01\text{ hr}) \\ &= 1.69\text{ hr} \end{aligned}$$

Plot the synthetic unit triangular hydrograph as shown.



7. RESERVOIR VOLUME (RIPPL DIAGRAM)

The minimum required volume of a water tank or reservoir can be estimated by creating a *Rippl diagram* according to the following steps, and as illustrated in Ex. 1.9.

step 1: Tabulate days of accumulation (column one) and the cumulative volume (column two).

step 2: Plot cumulative volume (y-axis) versus time (x-axis).

step 3: Determine the half-maximum accumulation and mark the point on the curve. The half-maximum accumulation is the intersection of cumulative volume and time at the point where time is one-half of the total duration.

step 4: Draw a straight line through the origin and the half-maximum point and continue it through the top of the plotted cumulative volume curve. This is the average volume line.

step 5: Draw two lines parallel to the average volume line. The first line, tangent 1, runs through times earlier than the cumulative volume curve, except at the point that it touches the curve. Tangent 2 runs through times later than the cumulative volume curve, except at the point that it touches the curve.

step 6: Starting at the point where the lowest parallel line intersects the x-axis, draw a vertical line upward to intersect with the highest parallel line.

step 7: Starting at the intersection of the vertical line with the highest parallel line, draw a horizontal line to intersect the y-axis. The *minimum required volume* is given by that point.

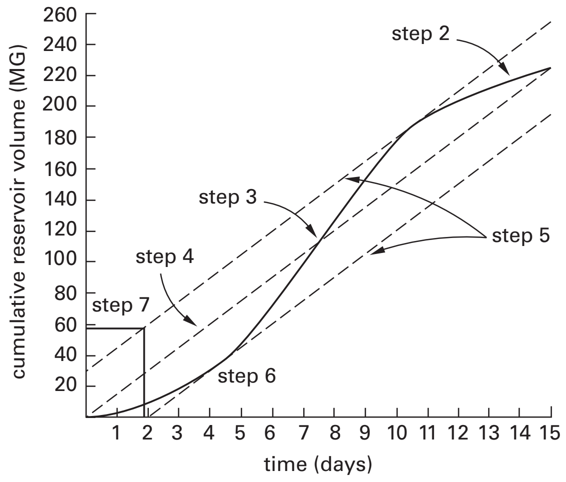
Example 1.9

Estimate the minimum required volume of a reservoir with a cumulative volume relationship as defined in the following table.

time (days)	cumulative volume (MG)	average volume (MG)	tangent 1 (MG)	tangent 2 (MG)
0	0	0	–	30
1	6	15	–	45
2	12	30	0	60
3	18	45	15	75
4	28	60	30	90
5	46	75	45	105
6	68	90	60	120
7	98	105	75	135
8	130	120	90	150
9	155	135	105	165
10	175	150	120	180
11	192	165	135	195
12	205	180	150	210
13	215	195	165	225
14	222	210	180	240
15	225	225	195	255

Solution

The data for step 1 is tabulated in the problem statement and plotted as a Rippl diagram, which is labeled with the associated step numbers. The minimum required volume is found in step 7 to be 60 MG.

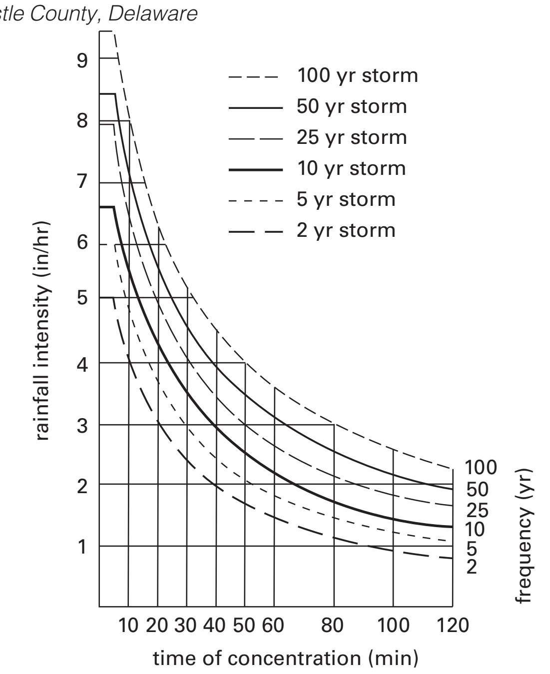


8. RAINFALL INTENSITY-DURATION-FREQUENCY RELATIONS

Intensity-Duration-Frequency Curves

Rainfall intensity, i, and storm duration (given by time of concentration, t_c) are related by *intensity-duration-frequency* (IDF) curves published by the USNWS, the United States Department of Agriculture (USDA), and local or state governments for specific geographic regions of the United States. Most state and local governments as well as the Departments of Transportation publish IDF curves in stormwater manuals. Figure 1.12 provides a typical IDF curve published by the USDA for New Castle County, Delaware.

Figure ~~*1.11*~~ *Rainfall Intensity-Duration-Frequency Curve for New Castle County, Delaware*



Adapted from *Surface Water Management Code*. 2007. USDA and New Castle County.

*Figure ~~1.15~~ Computing Time of Concentration**

Project	By	Date
Location	Checked	Date

Check one: ☐ Present ☐ Developed

Check one: ☐ t_c ☐ T_t through subarea

Notes: Space for as many as two segments per flow type can be used for each worksheet. Include a map, schematic, or description of flow segments.

Sheet flow (Applicable to t_c only)

Segment ID				
1. Surface description (Table 1.12)				
2. Manning's roughness coefficient, n (Table 1.12)				
3. Flow length, L (total $L \leqslant 300$ ft) ... ft				
4. Two year, 24 hour rainfall, P_2 ... in				
5. Land slope, S ... ft/ft				
6. $t_{sf} = \frac{(0.007 \text{ hr})(nL)^{0.8}}{P_2^{0.5}S^{0.4}}$ Compute t_{sf} ... hr		+		=

Shallow concentrated flow

Segment ID				
7. Surface description (paved or unpaved)				
8. Flow length, L ... ft				
9. Watercourse slope, S ... ft/ft				
10. Average velocity, v (Fig. 1.18) ... ft/sec				
11. $t_{sc} = \frac{L}{\text{v}}$ Compute t_{sc} ... hr		+		=

Channel flow

Segment ID				
12. Cross-sectional flow area, A_{ft^2} ... ft^2				
13. Wetted perimeter, WP ... ft				
14. Hydraulic radius, $R = \frac{A_{\text{ft}^2}}{\text{WP}}$, compute R ... ft				
15. Channel slope, S ... ft/ft				
16. Manning's roughness coefficient, n ... ft/ft				
17. $\text{v} = \left(\frac{1.49}{n}\right)R^{2/3}\sqrt{S}$ Compute v ... ft/sec				
18. Flow length, L ... ft				
19. $t_{ch} = \frac{L}{\text{v}}$ Compute t_{ch} ... hr		+		=
20. Watershed or subarea, t_c (add t_{sf}, t_{sc}, and t_{ch} from steps 6, 11, and 19) ... hr				

*TR-55 Worksheet 3 gives each time component as T_t. This book uses subscripts to differentiate between the component travel times. Reprinted from *TR-55: Urban Hydrology for Small Watersheds*. NRCS. Worksheet 3. 1986. USDA.

be calculated by prorating the coefficient by the amount of land cover and then dividing by the total watershed area. For example, in a watershed with two different areas of land use, the composite c_{total} value is

$$c_{\text{total}} = \frac{\sum c_i A_i}{\sum A_i} = \frac{c_1 A_1 + c_2 A_2}{A_1 + A_2} \qquad 1.15$$

Once the time of concentration is calculated, select the rainfall intensity, i, from an IDF curve, such as the one in Fig. 1.12.

Example 1.12

Using Table 1.11, calculate the composite runoff coefficient for a 100 ac watershed occupied by 50 ac of single family residential, 25 ac of light industrial, and 25 ac of park land uses.

Solution

Using Eq. 3.22 and Eq. 3.23, calculate the critical depth for a rectangular channel.

$$\begin{aligned} q &= \frac{Q}{b} = \frac{50\ \frac{\text{ft}^3}{\text{sec}}}{10\ \text{ft}} \\ &= 5\ \text{ft}^3/\text{sec-ft} \end{aligned}$$

$$\begin{aligned} d_c &= \frac{q^{2/3}}{g^{1/3}} \\ &= \frac{\left(5\ \frac{\text{ft}^3}{\text{sec-ft}}\right)^{2/3}}{\left(32.2\ \frac{\text{ft}}{\text{sec}^2}\right)^{1/3}} \\ &= 0.9\ \text{ft} \end{aligned}$$

7. GRADUALLY VARIED FLOW

Gradually varied flow occurs in open channels in which the flow depth and velocity vary slowly from section to section. In contrast, rapidly varied flow undergoes swift changes in depth and velocity, such as the flow at a hydraulic jump over a spillway. Water surface depth profiles can be estimated for gradually varied flow using the normal depth, d_n, as calculated by the Manning equation; using the critical depth, d_c, from Eq. 3.23; using Eq. ~~3.30~~ 3.27 for the Froude number, Fr; or using the standard step method discussed in Sec. 3.10.

The *Froude number* in a rectangular channel can be estimated by

$$\text{Fr} = \frac{q}{\sqrt{g d_1^3}} \qquad 3.27$$

The *specific discharge* (flow rate), q, in a rectangular channel is calculated using Eq. 3.22. The depth of flow is d, and g is the gravitational acceleration.

Figure 3.9 describes typical water surface profiles for mild and steep channel slopes with gradually varied flow. Mild channel slope profiles M1, M2, and M3 are controlled by downstream water surface elevations. Steep channel slopes S1, S2, and S3 are controlled by upstream water surface elevations.

Example 3.18

The flow in a 10 ft wide, concrete, rectangular channel is 1900 ft^3/sec. The flow depth upstream is 12 ft, and the downstream flow is critical. The channel slope is 0.002 ft/ft. Determine the normal and critical depths, and estimate the type of water surface profile.

Solution

Determine the normal depth using trial and error and the Manning equation. Assume a normal depth, d_n, of 15 ft. Table 3.1 gives the Manning roughness coefficient of concrete as 0.013. From Eq. 3.7,

$$\begin{aligned} Q &= \left(\frac{1.49}{n}\right) A R^{2/3} \sqrt{S} \\ &= \left(\frac{1.49}{0.013}\right)(15\ \text{ft})(10\ \text{ft})\left(\frac{(15\ \text{ft})(10\ \text{ft})}{15\ \text{ft} + 10\ \text{ft} + 15\ \text{ft}}\right)^{2/3} \\ &\quad \times \sqrt{0.002\ \frac{\text{ft}}{\text{ft}}} \\ &= 1855\ \text{ft}^3/\text{sec} \end{aligned}$$

This is close enough to the known flow rate, so use a normal depth, d_n, of 15 ft.

Determine the critical depth from Eq. 3.22 and Eq. 3.23.

$$\begin{aligned} d_c &= \frac{q^{2/3}}{g^{1/3}} \\ &= \frac{\left(\frac{Q}{b}\right)^{2/3}}{g^{1/3}} \\ &= \frac{\left(\frac{1900\ \frac{\text{ft}^3}{\text{sec}}}{10\ \text{ft}}\right)^{2/3}}{\left(32.2\ \frac{\text{ft}}{\text{sec}^2}\right)^{1/3}} \\ &= 10.4\ \text{ft} \end{aligned}$$

The normal depth of 15 ft is greater than the critical depth of 10.4 ft. Therefore, the flow is subcritical and mild. The depth at the upstream end, d_1, is 12 ft, which is greater than critical depth and less than normal depth. Therefore, $d_n > d_1 > d_c$, and the water surface profile type is mild at M2, as shown in Fig. 3.9.

8. HYDRAULIC JUMP

Hydraulic jump occurs when supercritical flow at high velocity abruptly transitions to subcritical flow at low velocity. This occurs when a steeply-sloped stream channel abruptly changes to a mild slope. The high velocity flow at lower depth transitions quickly to a low velocity flow at higher depth. Hydraulic jump commonly occurs at the base of dams or spillways, or at rapids or waterfalls along rivers and streams. Figure 3.10 describes the characteristics of hydraulic jump.