An Invitation to Water Resources Engineering

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About this Book

This book is designed to assist engineers to pass the water resources portion of the Civil Professional Engineer exam. The book can also serve as a text for college undergraduate and graduate level water resources engineering courses. The format of the book is consistent with the PE exam specifications recommended by the National Council of Examiners for Engineering and Surveying. Based on a survey conducted by Professional Publications Inc. (PPI), engineers who have taken the PE exam recommend that study materials (like this book) emphasize the following review topics in water resources engineering:

- Ponds, lakes, lagoons, and groundwater flow
- Water demand, wells, and hydraulic conductivity
- Hydrology and the Rational Method
- Pipe sizing problems
- Darcy-Weisbach and Hazen –Williams equations, and Moody diagram for pipe flow
- Converting units from gallons per minute, acre-feet, million gallons per day, etc.
- Metric units as well as English units.

The field of water resources can be studied in two parts: (1) water quality and (2) water quantity. Water quality involves the purity of water looking at the concentration of pollutants for water treatment and watershed management. Water quantity involves the mass of water on both ends of the water budget ranging from drought (not enough water) to flood (too much water). Water quality is directly related to water quantity as the units of concentration (mg/L) consist of units of volume and mass quantity (mg and L) in the numerator and denominator of the equation.

Water resources can also be grouped into (1) surface water and (2) groundwater flow. Surface water resources involve the movement of water through rivers and streams where runoff is the principle component of flow. Ground water resources includes wells and aquifers where infiltration is the principle component of flow. Many water resources problems include a surface water and groundwater component. For instance, to compute runoff using the Rational formula (Q = ciA), one must estimate the coefficient of runoff (c) which is the ratio of runoff to total precipitation minus infiltration.

Introduction

The water resources component constitutes a significant fraction of the morning and afternoon portions of the principles and practice of engineering exam (PE exam). During the Civil Breadth (AM) portion of the PE exam, the water resources section covers approximately 20 percent of the examination (National Council of Examiners for Engineering and Surveying, 2000). Environmental, geotechnical, structural, and transportation topics cover the remaining 80 percent in the AM.

During the Civil Depth (PM) portion of the PE exam, examinees have the choice to work ***one*** of the following five depth exams: water resources, environmental, geotechnical, structural, or transportation. Should the examinee chose to work the Water Resources P.M. Depth portion of the Civil P.E. Exam, she or he will be required to answer 40 multiple choice questions in this field. Sound knowledge of the water resource discipline is also needed to correctly answer a portion of the PM depth exam in other fields should the examinee chose to work the environmental, geotechnical, structural, or transportation topics.

To pass the water resources portion of the PE exam, one must possess a superior knowledge of hydraulics, hydrology, and water treatment. Hydraulics is the study of the flow of water through natural and engineered systems, usually closed conduit pipe and open channel flow. Hydrology is the study of water quantity dispersed between the earth and the atmosphere usually expressed in terms of the hydrologic cycle or water budget. Water treatment includes the design of water supply treatment and pipeline distribution systems necessary to deliver clean and plentiful drinking water to customers.

The water resources discipline differs from others in civil engineering, such as structural engineering, because it requires understanding natural processes such as precipitation or river systems that respond to the unpredictable forces of nature. For instance, if several structural engineers were asked to design a beam, they all could precisely calculate the same thickness since the beam is an engineered structure manufactured to some common specification using the national code. Whereas, if several water resources engineers were asked to estimate the design flow for a storm sewer, they all might recommend several slightly different but ultimately correct pipe diameters since the design calculation is based on best professional judgment using variable natural processes such as precipitation and assumptions concerning runoff curve number, Manning's roughness value, and time of concentration.

Therefore, it is especially important in the water resources discipline to document assumptions in the calculations based on source tables and references. If the examinees correctly reference the source of assumptions in the calculations, then there is a greater probability in receiving a correct grade on the question. Also, it is important to be conservative in water resources design to account for the unpredictable forces of nature. Design flows, pipe sizes and stormwater basin volumes should always be rounded up to account for the uncertainty of nature and the climate.

Common Unit Conversions and Equations

Know your unit conversions! One of the most fundamental, yet potentially error - prone facets of water resources engineering that can befall the examinee or student is conversion of units. For instance, the United States Geological Survey stream gage network (www.usgs.gov) provides records of stream flow in cubic feet per second (cfs). The designer of a water distribution network requires equations of flow in terms of million gallons per day (mgd) and gallons per minute (gpm). Precipitation is measured by the U. S. National Weather Service in inches yet engineers estimate stormwater runoff in units of cubic feet per second. Since the Federal government continues to provide water resources data such as precipitation and stream flow in English units, it is likely that much of the hydrology and hydraulics problems will be worked in U. S. units. Correct unit conversion is essential if one is to master the water resources portion of the PE exam. Always cancel out the units. Study the Metric (S.I.) and English (U. S.) unit conversions. Some of the more common unit conversions in water resources are listed below:

1 acre = 43,560 square feet

1 square mile = 640 acres

1 cubic foot = 7.48 gallons

1 minute = 60 seconds and 60 minutes = 1 hour

1 day = 24 hours

1 in = 2.54 cm

1 ft = 0.33 m

1 mi = 1.6 km

Physical Properties and Constants

Absolutely know your physical properties and constants that constantly crop up in water resources engineering equations. Table 1 summarizes several of the more important physical properties and constants utilized in water resources.

Table 1. Physical properties and constants in water resources

Symbol	Property	Value (U.S.)	Value (S.I.)
p_{atm}	Atmospheric pressure	14.7 psi at sea level	101.3 kN/m^2 at sea level
g	Acceleration due to gravity	32.2 ft/sec^2	9.8 m/sec^2
γ	Specific weight of water	62.4 lb/ft^3 at 50 deg F	9.81 kN/m^3 at 15 deg C
p	Density of water	1.94 slug/ft^2 at 39 deg F	1000 kg.m2 at 4 deg C
μ	Viscosity of water	2.735 x 10^5 lb-sec/ft^2 at 50 deg F	1.139 x 10^3 N-sec/m^2 at 15 deg C
υ	Kinematic viscosity of water	1.410 x 10^5 lb-sec/ft^2 at 50 deg F	1.139 x 10^3 N-sec/m^2 at 15 deg C

Example 1

U.S. Solution

Calculate the volume (V) in MG from 6 in of rain falling over a 320 ac watershed.

V = 6 in (1 ft /12 in) (320 ac) = 160 ac-ft
= 160 ac-ft (43,560 sf / ac) = 6,969,600 cf
= 6,969,600 cf (7.48 gal/cf) = 52,132,000 gal
V = 52 MG

S.I. Solution

Calculate the volume (V) in ML from 10 cm of rain falling over a 1 ha watershed.

V = 10 cm (1 m/ 100 cm)(1 ha) = 0.1 ha-m
= 0.1 ha-m(10,000 m^2 / ha) = 1,000 m^3
= 1000 m^3 (1000 L / 1 m^3) = 1,000,000 L
V = 1 ML

Example 2

Estimate the flow (Q) in a creek in mgd if the USGS stream gage estimates the flow is 76 cfs.

Q = 76 cfs (60 sec / min) (60 min / hr) (24 hr / day) (7.48 gal / cf) = 4,912,000 gpd
Q = 49 mgd

Example 3

A water purveyor estimates water demand at 2 mgd. What is the design flow (Q) for the water distribution system in gallons per minute?

Q = 2 mgd (1,000,000 gal / mg) (1 day/24 hr) (1 hr / 60 min)
Q = 1,288 gpm

Example 4

U.S. Solution

What is the time (t) in days to drain a 2 billion gallon reservoir with an outlet pipe capacity of 100 cfs?

t = 1 bg (2,000,000,000 gal / bg) (7.48 gal / cf) / 100 cfs
= 2,673,796 s (1 min / 60 s) (1 hr / 60 min)
= 742 hr (1 day / 24 hr)
t = 31 days

S.I. Solution

How many days will it take to drain a 200 million liter reservoir with an outlet pipe capacity of 0.1 m^3/ s?

t = 200 ML(1,000,000 L/ML)(1 m^3/1000 L)/ **0.1** m^3/ s
= 2,000,000 s (1 min/ 60 s) (1 hr/60 min)
= 555 hr (1 day/24 hr)
t = 23 days

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1.0 Hydrology

Hydrology is the study of water quantity and quality dispersed and circulated between the earth and the atmosphere usually expressed in terms of the hydrologic cycle or water budget. Every school child studies the hydrologic cycle, which starts with a form of precipitation - rain, fog, sleet, or snow - falling from the sky. The precipitation falls on to the ground and either runs off to a waterway (runoff), permeates into the ground (infiltration), disperses back into the atmosphere (evaporation) or is absorbed by plants and trees (transpiration). The water vapor in the atmosphere condenses, precipitation occurs, and the hydrologic cycle reoccurs again. The professional engineer is most interested in the runoff and infiltration terms of the hydrologic cycle. Estimates of runoff are needed to design storm sewers, culverts, and stormwater basins. Infiltration is necessary to design groundwater facilities such as septic systems and recharge basins. Key terms in the discipline of hydrology include precipitation (in or in/hr), runoff (cfs), and infiltration (in or in/hr). Figure 1.1 and 1.2 illustrate the key components of the hydrologic cycle.

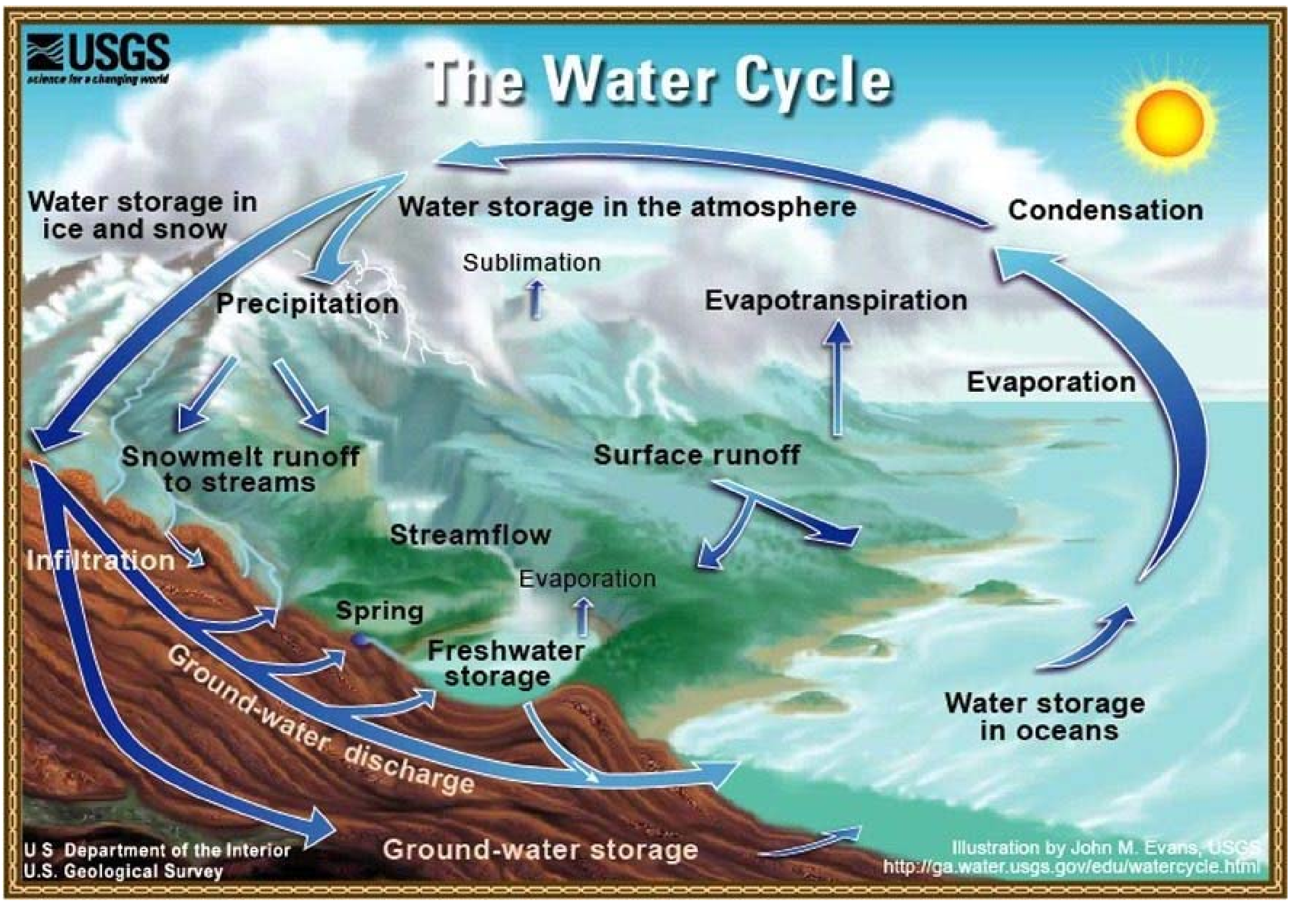


Figure 1.1. The hydrologic cycle (source: United States Geological Survey, 2006)

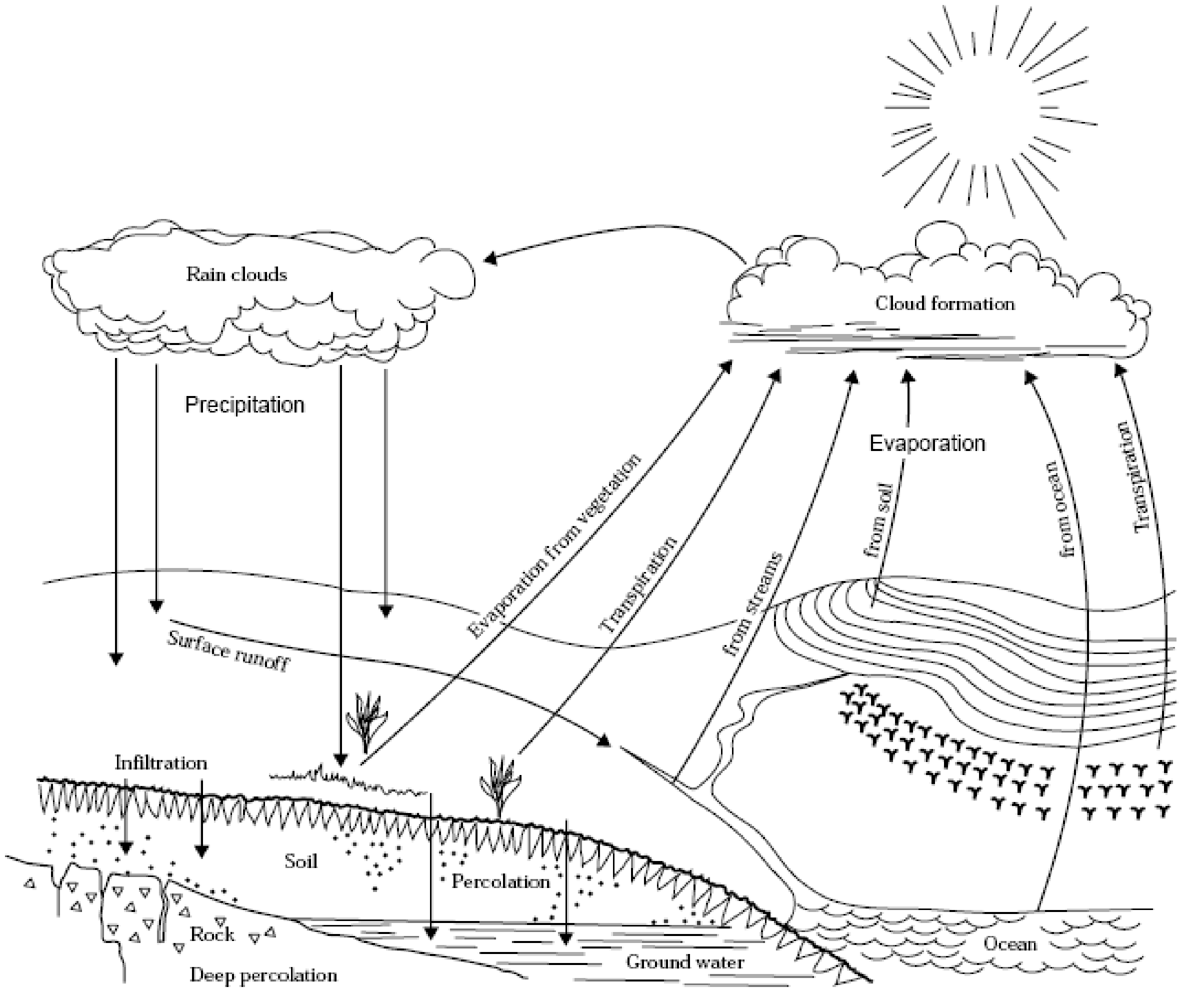


Figure 1.2. The Hydrologic Cycle (source: USDA-NRCS)

The science of hydrology is defined by the water budget equation of the hydrologic cycle:

$P = R + I + ET - \Delta S$

Where:

P = precipitation over a time frame say annually, monthly, daily, or hourly

R = runoff that flows overland to a waterway

I = infiltration into the groundwater table

ET = sum of evaporation (E) to atmosphere plus transpiration (T) by plants

Δ S = change in moisture storage in surface water, groundwater, and/or soil

Units of the water budget are usually in terms of volume (in, cf, or ac-ft) or flow (cfs or mgd).

$$P = R + I + ET$$

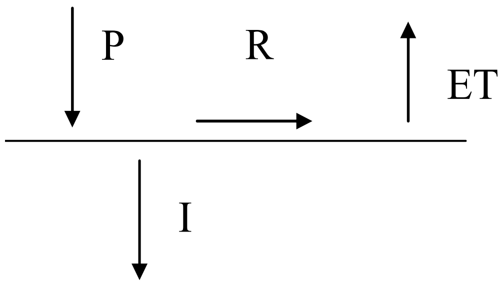


Figure 1.3. Components of the water budget

Example 1.1

In the mid Atlantic region of the United States, a meteorological station records the following data during 2004. The annual precipitation is 42 in, infiltration is 8 in, evapotranspiration is 24 inches, and change in moisture storage is 0. Calculate annual runoff in (a) inches and (b) acre-feet for a watershed area of 1 square mile.

Using the water budget equation, solve for runoff (R):

$$P = R + I + ET - \Delta S$$

$$42 \text{ in} = R + 8 \text{ in} + 24 \text{ in} - 0$$

$$R = 42 - 8 - 24$$

$$R = 10 \text{ in}$$

Compute the runoff in acre-feet

$$R = 10 \text{ in } (1 \text{ ft} / 12 \text{ in})\ (1 \text{ sq mi})\ (640 \text{ ac} / \text{sq mi})$$

$$R = 533 \text{ ac} - \text{ft}$$

Watershed Management

The watershed is the fundamental hydrologic unit for managing water resources. The watershed is defined as the area that flows to a particular waterway such as a lake, stream, river or ocean. Figure 1.5 delineates the major watersheds in the United States. Watersheds come in all sizes ranging from the size of a city block to the largest watershed on earth, the Amazon Basin.

Depending on the project area of interest, watersheds can be nested (Figure 1.5). For instance, in Delaware a 0.5 sq mi urban drainage in Newark is nested within the 7 sq mile Cool Run subwatershed which in turn is nested within the 98 sq mi White Clay Creek watershed. The White Clay Creek watershed is one of the four major streams in the 565 sq mi Christina River subbasin which is part of the 13,000 sq mi Delaware River Basin.

Watersheds can be categorized by the following hierarchy:

Unit	Area (sq mi)	Example
Catchment	0.5 to 1.0	Urban drainage (0.5 sq mi)
Subwatershed	1.0 to 10	Cool Run (7 sq mi)
Watershed	10 to 100	White Clay Creek (98 sq mi)
Subbasin	100 to 1000	Christina Basin (565 sq mi)
Basin	over 1000	Delaware River (13,000 sq mi)

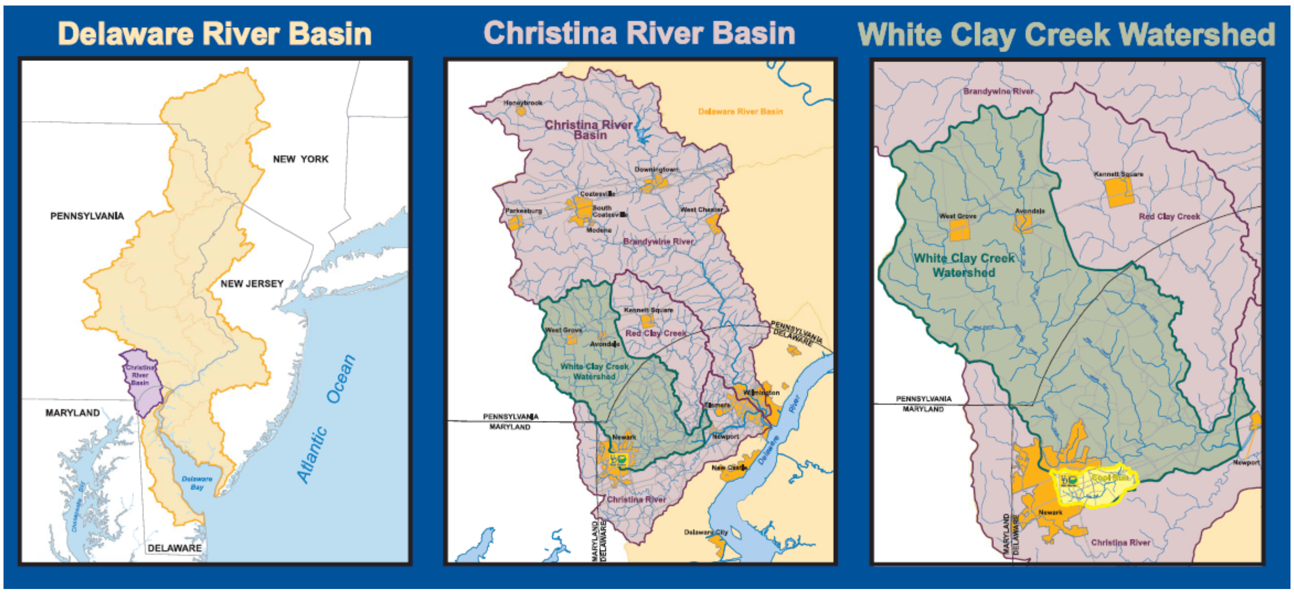


Figure 1.4. The nested watersheds in the Delaware River Basin

The United States Geological Survey delineated the 21 major watersheds in the United States (Figure 1.5). Notice that the watershed boundaries rarely coincide with the state political boundaries. Since the watershed boundaries do not honor the state boundaries, engineers must have knowledge of the scientific <u>and</u> the socio-political aspects of water resources management. Different states in the same watershed may have different stormwater and floodplain regulations. The interstate or intergovernmental nature of our nation's river and stream systems is one of the reasons why watershed management and water resources engineering are so complex and challenging.

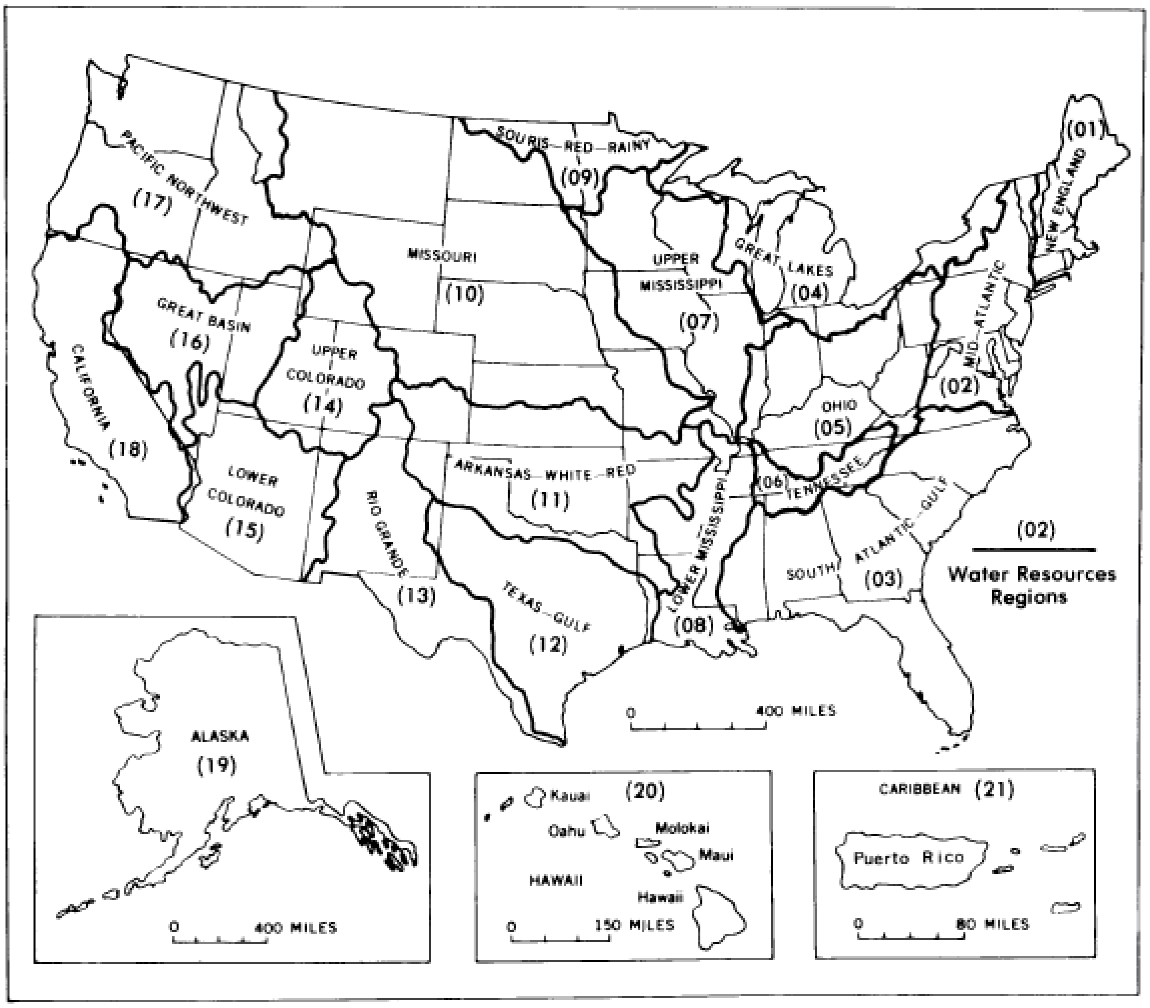


Figure 1.5. Major watersheds in the United States (source: USGS)

Watersheds can be delineated on a topographic map according to the following 3 step process (Figure 1.6).

1. Identify the point of interest (P.I.) at the outlet of the watershed in question.

2. Highlight the streams on the topographic map

3. Starting at the P.I., delineate the watershed with the boundary crossing perpendicular to the contour lines. Look for closed contour lines at the top of the ridgelines which usually indicate the watershed divide.

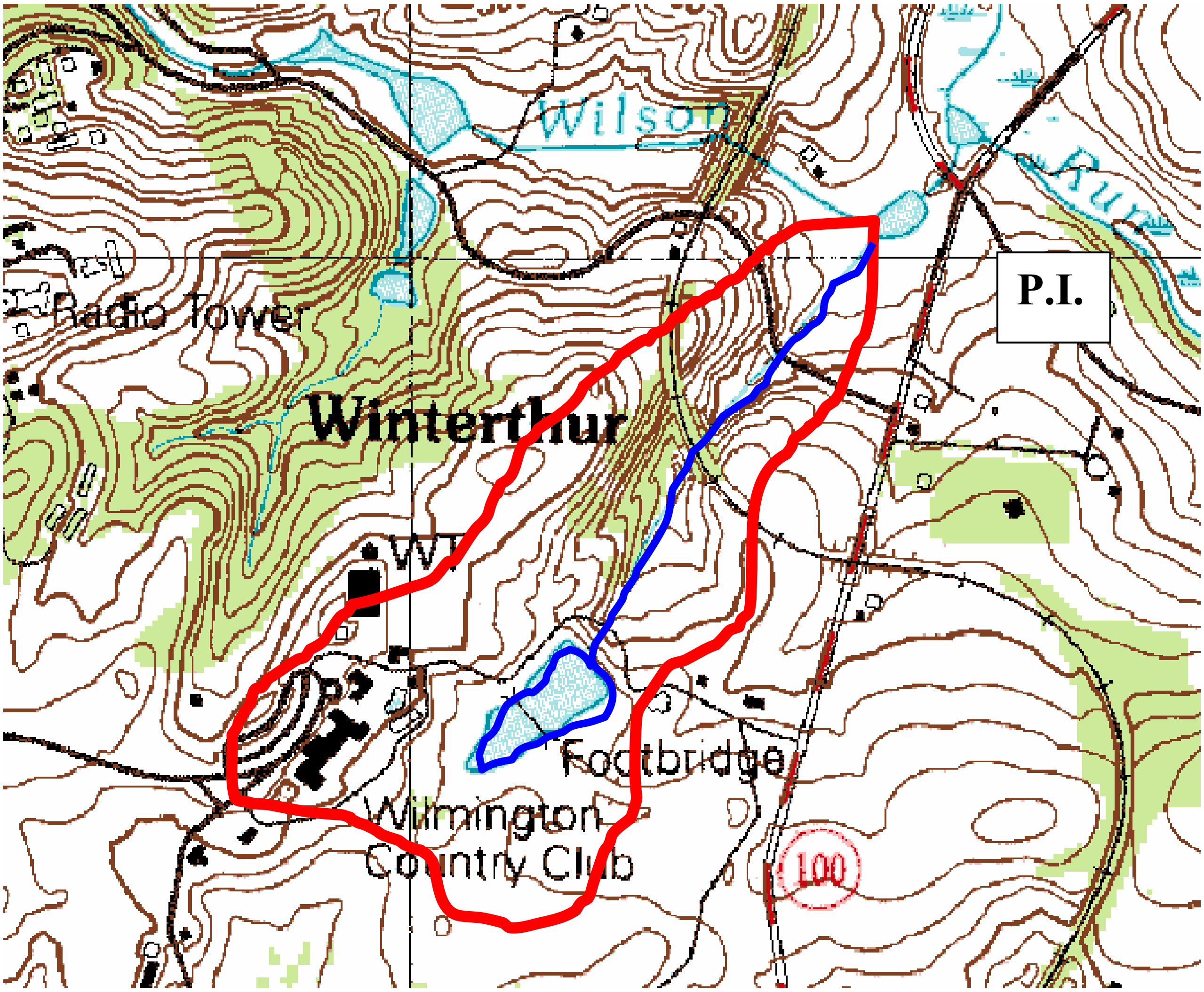


Figure 1.6. Wilson Run watershed boundary

1.1 Storm Characterization

Storm characterization is used to define the precipitation term (P) of the hydrologic cycle. Figure 1.7 depicts the mean annual precipitation in the United States for 1971 – 2000 from the U. S. National Weather Service. Annual precipitation is highest (over 60 in per year) along the gulf coast and the southeastern United States fueled by tropical moisture and in the northwest powered by the moist Pacific Marine currents marine currents. Generally as one preceeds inland, the climate becomes drier in the semiarid (10 to 20 inches of precipitation per year on the Great Plains) and arid climates of the desert southwest (4 to 10 inches per year). Precipitation also increases with elevation in mountainous areas due to the lifting or orographic effect (rising air cools and condenses producing moisture). Table 1.1 summarizes annual precipitation for major U.S. cities.

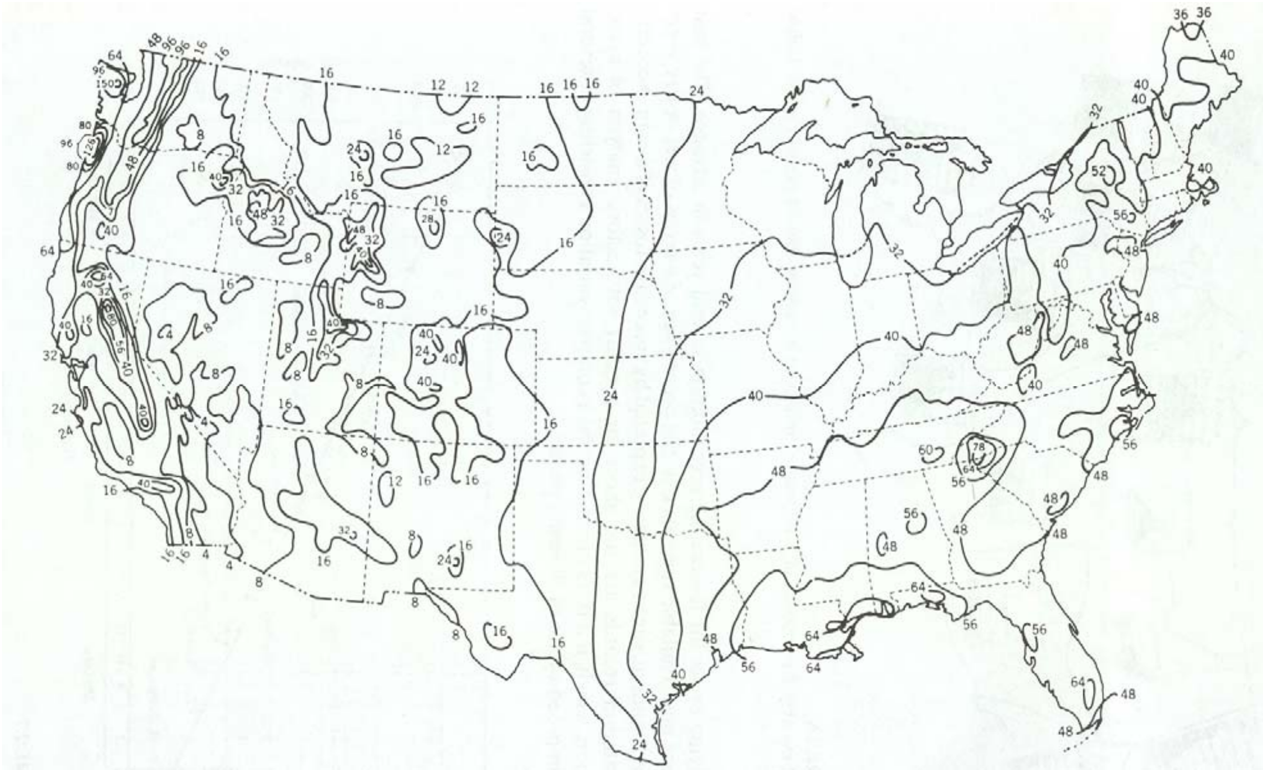


Figure 1.7. Mean annual precipitation in the continental United States in inches.
(source: U. S. National Weather Service)

Table 1.1. Annual precipitation in major U. S. cities.(source: U. S. National Weather Service)

City	Annual Precipitation (in)
Atlanta	50.77
Boston	41.51
Chicago	35.82
Cleveland	36.63
Denver	15.40
Houston	46.07
Kansas City	37.62
Los Angeles	14.77
Miami	55.91
Minneapolis	28.32
New Orleans	61.88
New York	47.25
Philadelphia	41.41
Phoenix	7.66
Salt Lake City	16.18
San Francisco	19.71
Seattle	38.60
St. Louis	37.51
Washington, D. C.	38.63

Precipitation is measured using the following parameters:

- Total depth (in)
- Duration (days or hours)
- Intensity (in/hr)
- Return interval (e.g. 10-yr, 50-yr storm)
- Storm distribution
-

Precipitation depths for the United States can be determined from U. S. NOAA Atlas 2 and Atlas 14 maps and U.S. National Weather Service Technical Papers 40, 43, and 47 as described in Table 1.2 is excerpted from the NOAA web site at http://www.weather.gov/oh/hdsc/currentpf.htm.

Table 1.2. U. S. National Weather Service precipitation publications

States	Duration of storm (hr)
Arizona, Nevada, New Mexico, Utah, and Southeast California	NOAA Atlas 14 (2003)
Remainder of the Western US	NOAA Atlas 2 (1973)
Delaware, Illinois, Indiana, Kentucky, Maryland, New Jersey, North Carolina, Ohio, Pennsylvania, South Carolina, Tennessee, Virginia, West Virginia, and Washington, DC	NOAA Atlas 14, Volume 2 (June 2004)
Remainder of the Eastern US	Tech. Paper 40 (1961)
Hawaii	Tech. Paper 43 (1962)
Alaska	Tech. Paper 47 (1963)
Puerto Rico	Tech. Paper 42 (1961)

Total precipitation depths for the 2-, 5-, 10-, 25-, 50-, and 100- year return intervals for a 24-hour duration storm for the central and eastern United States can be determined from design storm charts (Figure 1.8) contained in Technical Release 55 (TR-55) manual (USDA – NRCS, 1986). For instance, referring to Figure 1.8, the 100 – year, 24-hour precipitation depth for Philadelphia, Pennsylvania is 7.5 inches.

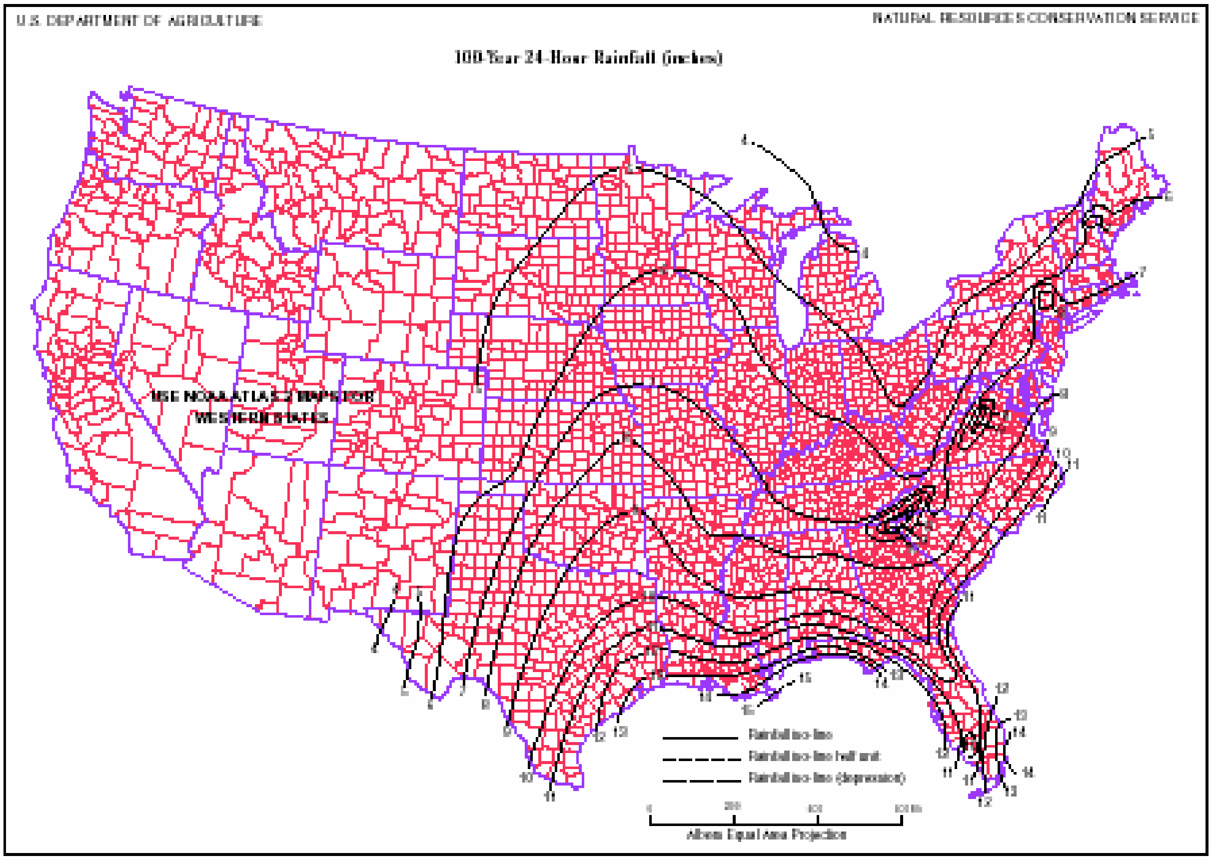


Figure 1.8. Typical precipitation depth and duration curves from the TR-55 manual.
(source: USDA – NRCS, TR-55, 1986.

The TR-55 manual classifies rainfall distribution in the United States according to the map shown in Figure 1.9. Most of the states in the U.S. have a Type II rainfall distribution, which means that over 90 percent of the rainfall is distributed within the middle 12 hours of a 24-hour storm.

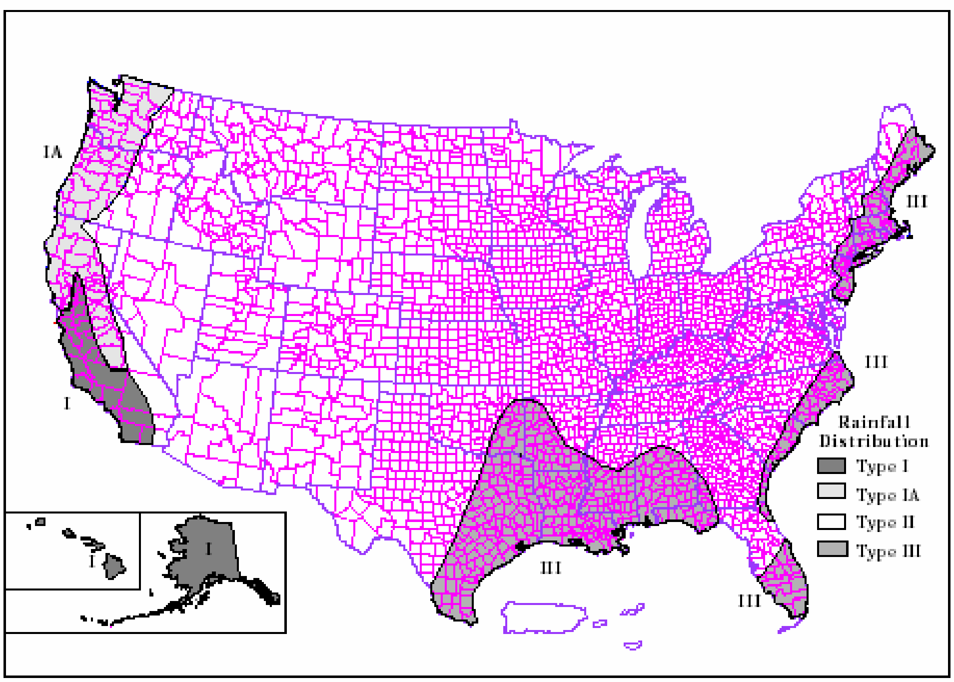


Figure 1.9. Rainfall distribution in the United States (source: USDA-NRCS, TR- 55 manual, 1986)

The U.S. National Weather Service characterizes precipitation using storm hyetographs, which summarize the time interval, depth, and intensity of precipitation events. Table 1.3 records precipitation depth and intensity data for a typical storm.

Table 1.3. Precipitation depth and intensity data for a typical storm.

Hour of Storm	Incremental Depth (in)	Cumulative Depth (in)	Incremental Intensity (in/hr)	Cumulative Intensity (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

Figure 1.10 plots a storm hyetograph from the data in Table 2.2, which relates incremental precipitation depth on the vertical axis with time on the horizontal axis.

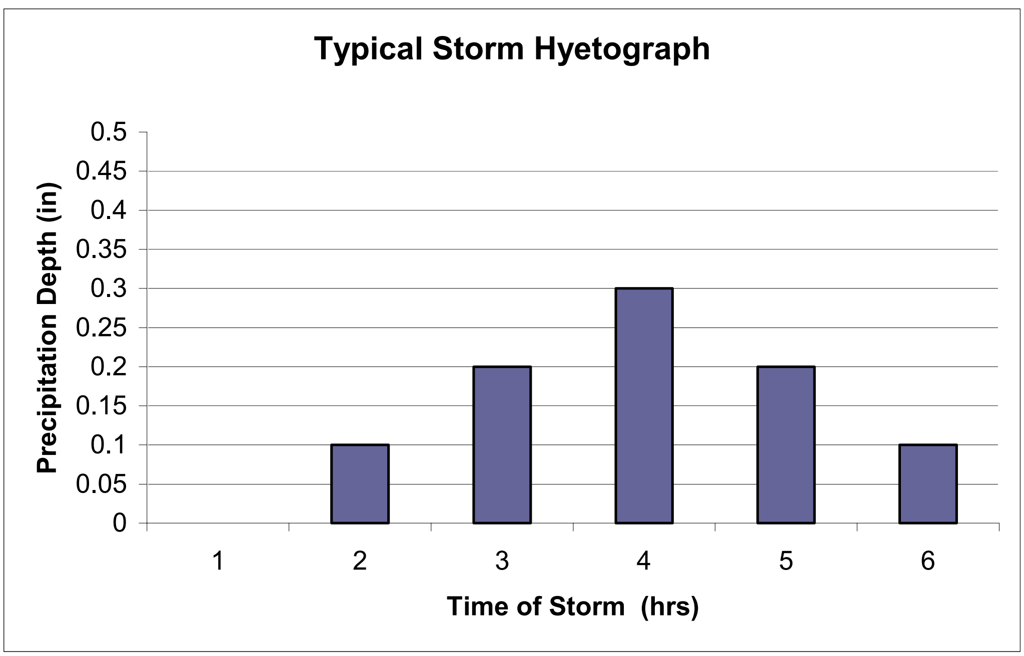


Figure 1.10. Typical storm hyetograph.

Over large areas or watersheds, precipitation can vary markedly depending on elevation, proximity to large waterways and other factors. If multiple precipitation gages are available, the precipitation at each station can be weighted in proportion to the area that each gage represents using the Theissen network (Figure 1.11). The Theissen network is constructed according to the following steps:

1. Plot the location of each precipitation gage.

2. Connect each adjacent gage by straight lines.

3. Delineate perpendicular bisecting lines halfway between each adjacent gage.

4. Define a series of polygons with the perpendicular bisecting lines.

5. Calculate the Theissen area of each polygon. Sum the Theissen areas.

6. Multiply the Theissen area by the precipitation depth at each station to determine the product.

7. Sum the products

8. Determine the average precipitation in the watershed as the sum of the products divided by the sum of the Theissen areas.

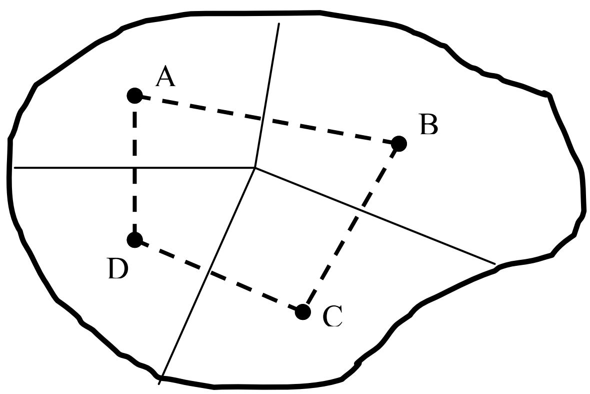


Figure 1.11. Theissen precipitation diagram

Example 1.2

Compute the average precipitation for the watershed depicted in Figure 1.11 if the precipitation at gage A is 4.0 in, at gage B is 5.0 in, at gage C is 3.5 in, and at gage D is 4.5 in. Construct a table as follows:

Station	(1) Theissen Area (sq mi)	(2) Precipitation (in)	(3) = (1) x (2) Product (sq mi – 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

The average precipitation is then the sum of the product (column 3) divided by the sum of the Theissen area (column 1):

$$P = (232 \text{ sq mi} - \text{in}) / (54 \text{ sq mi})$$

$$P = 4.3 \text{ in}$$

1.2 Storm Frequency

Engineers employ the concept of storm frequency to account for probability and risk in designing water resources structures. One commonly hears the concept of the 100-year storm or the 25 –year flood. A 100-year storm is commonly incorrectly defined as the storm likely to occur once every 100 years. While there is a low probability for a 100 –year storm to occur twice within a short period say of 4 years, it is not uncommon. It is more appropriate to describe the severity of a 100 –year storm by its probability as an event with a 1 % chance of occurring within a given year.

The frequency of a storm is related to the probability by the following equation:

$$T = 1 / p$$

Where:

T = return interval or recurrence interval of a storm in years.

p = probability of a storm occurring once in any given year

A 100-year storm has a probability (p) of 1/T or 1 / 100 which is a 0.01 or a 1 percent chance of occurring in any given year. Table 1.4 relates storm return interval with probability.

Table 1.4. Storm return interval and probability

T	p	Chance of occurring once in any given year
2 – yr	0.50	50 %
5 – yr	0.20	20 %
10 – yr	0.10	10 %
25 - yr	0.04	4 %
50 –yr	0.02	2 %
100-yr	0.01	1 %
500-yr	0.005	0.5 %

The probability of a storm or flood occurring one or more times in a given time period is given by the following equation:

$$f(n) = \frac{(N!)\,(P)^n\,(1-p)^{N-n}}{n!\,(N-n)!}$$

Where:

$f(n)$ = probability of a storm or flood of a certain return interval occurring within N years.

N = number of years in time frame.

n = number of times a storm or flood occurs in a certain time frame.

p = probability of a storm or flood.

! = factorial or the product of all the whole numbers, except zero, that are less than or equal to that number. For instance, 4! = 4 x 3 x 2 x 1.

Example 1.3

What is the probability f(n) that a 100 – yr storm occurs twice in 4 years.

N = 4 yr

n = 2

p = 1/T = 1/100 yr = 0.01

Employing the equation:

$$f(n) = \frac{(N!)\,(P)^n\,(1-p)^{N-n}}{n!\,(N-n)!}$$

$$f(2) = \frac{(4\text{ yr}!)\,(0.01)^2\,(1-0.01)^{4-2}}{2!(4\text{ yr}-2)!}$$
$$= \frac{4(3)(2)(1)\,(0.0001)\,(0.9801)}{(2)(1)(2)(1)}$$

$$f(2) = 0.0005 = 0.05\%$$

There is a 0.05 % probability that a 100-year storm can occur twice in 4 years, or a 1 in 2000 chance.

To minimize risk to life and property, engineers employ the following return interval criteria to conservatively design water resources engineering structures (Table 1.5).

Table 1.5. Water resources design return interval criteria.

Return Interval (T)	Design Criteria
10 –year	Storm sewer design
50 –year	Small culvert and bridge design (low traffic)
100 –year	Large culvert and bridge design (high traffic) Floodplain delineation Stormwater detention basins
500 –year	Dam Safety

1.3 Hydrographs

Hydrographs depict the change in flow or runoff over time. Water resources engineers utilize hydrographs whenever the volume *and* rate of runoff are necessary for design. The volume of runoff is necessary to estimate the required storage volume of a storm water pond and can be calculated as the area under the hydrograph curve.

Hydrographs are composed of two flow components: (1) base flow, which is the movement of groundwater to a stream or waterway, and (2) overland flow, which is the surface runoff to a waterway during and after a precipitation event.

Commonly, it is necessary to separate base flow from the overland flow component using the straight line method. A horizontal line is drawn starting from the point where the hydrograph slope starts to rise and ends at the point where the hydrograph is falling as depicted in Figure 1.12. The portion of the hydrograph below the horizontal line is the base flow component and the portion above the line is the overland flow component.

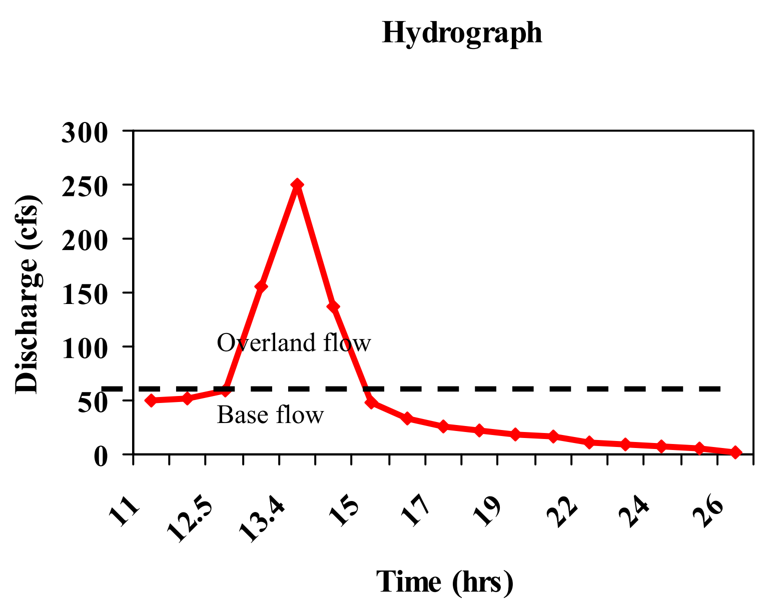


Figure 1.12. Hydrograph depicting baseflow and overland flow requirements.

Once overland flow is separated from the hydrograph, the volume of overland runoff can be calculated as the area under the curve. The volume can be estimated by several techniques such as estimating the total area of triangles and rectangles under the curve. For example, the volume of overland flow from the overland flow hydrograph in Figure 1.12 can be calculated as the area of the triangle as follows.

$$V = \frac{(b)(h)}{2}$$

Where:

V = volume of overland runoff (cf)

b = base of the triangle (hr)

h = height of the triangle (cfs)

$$V = \frac{(16.5 \text{ hr} - 12.7 \text{ hr})(250 \text{ cfs} - 50 \text{ cfs})(60 \text{ min/hr})(60 \text{ sec/min})}{2}$$

$$= \frac{(3.8 \text{ hr})(200 \text{ cgs})(3600 \text{ sec/hr})}{2}$$

$$V = 1{,}368{,}000 \text{ cf}$$

Unit Hydrograph

A unit hydrograph can be developed where 1 in of precipitation falls evenly over the entire area of a watershed,

The volume of overland runoff volume for a unit hydrograph relates to the watershed area and precipitation depth according to the following equation:

$$V = A(P)$$

Where:

V = volume of overland runoff (cf)

A = area of the watershed (sq mi or ac)

P = precipitation depth (in)

The unit hydrograph of a storm can be defined by multiplying the flow coordinates on the vertical axis by the precipitation depth of the storm.

Example 1.4

Define the unit hydrograph for the hydrograph coordinates in Table 2.5 assuming the watershed area is 0.5 sq mi. Then, define the hydrograph for this watershed for a 2 inch storm.

Find the average precipitation for the watershed.

$$V = A/P$$

Or:

$$P = V/A$$

$$= \frac{1,368,000 \text{ cf } (12 \text{ in/ft})}{0.5 \text{ sq mi } (640 \text{ ac/sq mi})(43,560 \text{ sf/ac})}$$

$$= 1.2 \text{ in}$$

Referring to Table 1.6, coordinates for the 1.0 inch unit hydrograph (column 3) are computed by dividing each runoff point in column 2 by P = 1.2 in..

The hydrograph coordinates for a 2 inch storm in the watershed (column 4) are computed by multiplying the unit hydrograph runoff (column 3) by 2 inches.

Table 1.6. Unit Hydrograph Tabulation

(1) Time (hrs)	(2) 1.2 inch Hydrograph Runoff (cfs)	(3) Unit Hydrograph Runoff (cfs/in)	(4) 2.0 in Hydrograph Runoff (cfs)
12.5	60	60/1.2 = 50	50(2) = 100
13.0	155	155/1.2 = 129	129(2)= 258
13.4	250	208	416
14	137	114	228
15	49	41	82

Synthetic Unit Triangular Hydrograph

In ungaged watersheds, synthetic hydrographs can be estimated using the USDA - NRCS synthetic unit triangular hydrograph method (Figure 1.13). The coordinates of the synthetic hydrograph are defined by the peak runoff (Q_p), time to peak (T_p), and time of concentration (T_c) where:

Q_p = peak runoff as defined by the Rational method or the USDA NRCS TR55 Curve Number method (cfs).

T_c = time of concentration (hr or min).

T_p = time to peak runoff (hr or min)as measured from the beginning of overland runoff hydrograph = $0.67(T_c)$

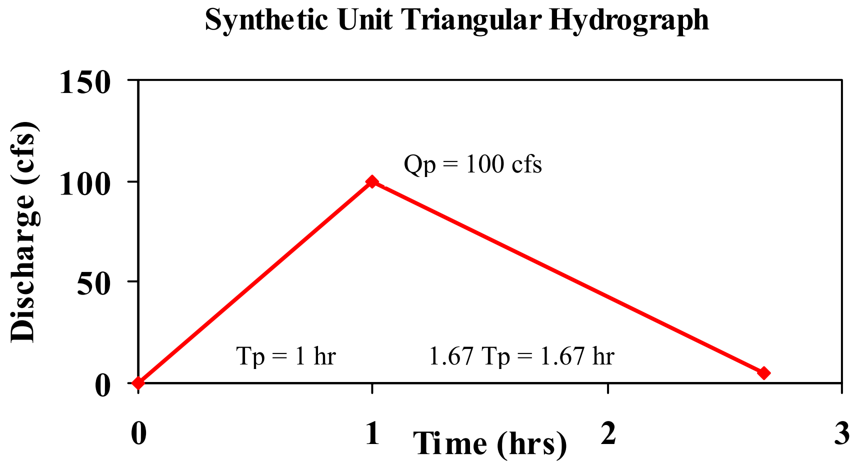


Figure 1.13. Synthetic unit triangular hydrograph.

Example 1.5

Define a synthetic unit triangular hydrograph where the peak runoff is estimated as 100 cfs and the time of concentration is 1.5 hrs.

Plot the peak runoff at 100 cfs.

Define the ascending limb of the hydrograph = time to peak (Tp) = 0.67(Tc) = 0.67(1.5) = 1 hr.

Define the receding limb of the hydrograph = 1.67 (Tp) = 1.67 (1) = 1.67 hr.

Plot the synthetic unit triangular hydrograph as in Figure 2.7.

1.4 Rainfall Intensity and Duration

Rainfall intensity and storm duration are related by intensity – duration- frequency (IDF) curves published by the U. S. National Weather Service, the U.S. Department of Agriculture, and local or state governments for specific geographic regions of the United States. The storm duration is equal to the time of concentration for small watersheds. Figure 1.14 provides a typical IDF curve published by the U.S. Department of Agriculture for New Castle County, Delaware. Most state and local governments publish IDF curves in storm water manuals for particular areas of the country.

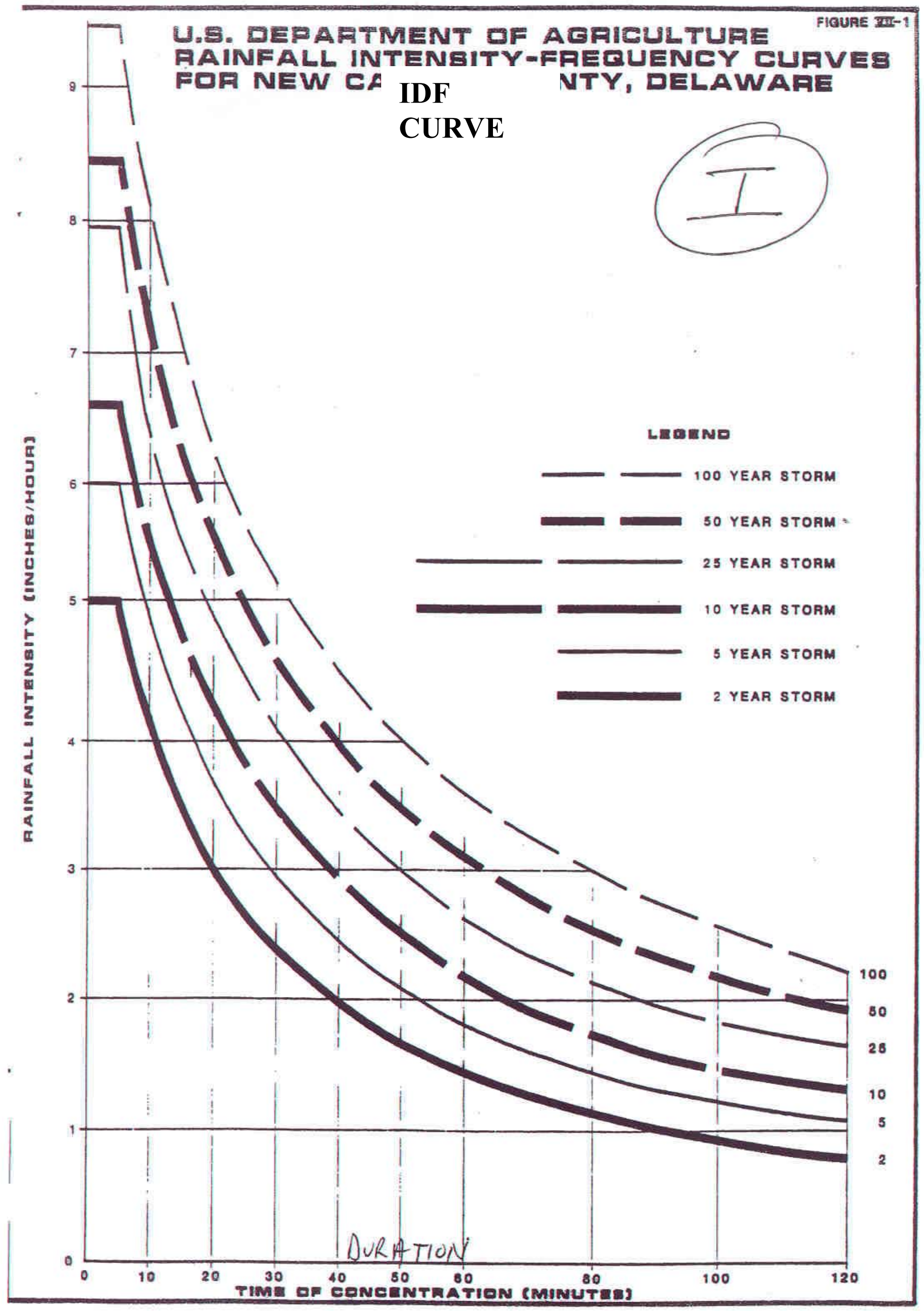


Figure 1.14. Rainfall intensity-duration-frequency curve for New Castle County, Delaware.

Example 1.6

Estimate the precipitation intensity for a 10 – year storm in New Castle County, Delaware assuming the time of concentration for the watershed is 60 minutes.

From Figure 2.8, for a time of concentration = 60 minutes, the 10-year storm intensity is 2.2 in/hr.

In locations where rainfall IDF curves are not available, the engineer may use the following equation from the U. S. National Weather Service to estimate rainfall intensity for the 10 – yr storm event:

$$,i_{10} = \frac{a}{(t+b)^c}$$

Where:

i_{10} = rain fall intensity for the 10 – year, 1 – hour duration storm event.

a = constant related to return interval listed for various cities in Table 1.7.

b = constant related to return interval listed for various cities in Table 1.7.

c = constant related to return interval listed for various cities in Table 1.7.

t = 60 sec for a 1 hr duration storm.

Table 1.7. Intensity Duration Frequency Constants for U.S. Cities for the 10 year Storm (source: Amercian Geophysical Union, 1982)

City	a	b	c
Atlanta	64.1	8.16	0.76
Chicago	60.9	9.56	0.81
Cleveland	47.6	8.86	0.79
Denver	50.8	10.5	0.84
Houston	98.3	9.3	0.80
Los Angeles	10.9	1.15	0.51
Miami	79.9	7.24	0.73
New York	51.4	7.85	0.75
Santa Fe	32.2	8.54	0.76
St. Louis	61.0	8.96	0.78

Example 1.7

Calculate the rainfall intensity for a 10 year sorm in Denver.

$$i_{10} = \frac{a}{(t+b)^c}$$

From Table 1.7:

$a = 50.8$

$b = 10.5$

$c = 0.84$

$T = 60 \text{ min}$

$$i_{10} = \frac{a}{(t+b)^c}$$

$$i_{10} = \frac{50.8}{(60 \text{ min} + 10.5)^{0.84}}$$

$$i_{10} = 0.72 \text{ in/hr}$$

1.5 Evaporation

Evaporation is defined as water that is converted from liquid to vapor that is released into the atmosphere. Evapotranspiration, which is measured from pan evaporation data, is defined as all water vapor emitted to the atmosphere from plants and other sources. Evaporation increases with warmth and plant foliage during the spring, summer, and early fall months and decreases during the cooler months after the leaves fall off of the trees and plants. Table 1.8 summarizes monthly evaporation data for the Wilmington airport, Delaware measured during 1997. Notice that evaporation peaks during the warm summer months and declines during the cooler winter months.

Table 1.8 Evaporation measured at Wilmington Airport, Delaware during 1997.

Month	Evaporation (in)
Jan	0.4
Feb	0.5
Mar	1.2
Apr	1.8
May	3.0
Jun	4.0
Jul	4.8
Aug	5.4
Sep	4.8
Oct	3.2
Nov	2.0
Dec	0.6

Figure 1.15 depicts the average annual lake evaporation in the Unites States as measured by the U. S. National Weather Service. In the arid parts on the U.S. such as in Arizona, evaporation can exceed 40 inches or 3 times the annual precipitation thus causing substantial drawdowns of lakes and reservoirs in the western USA. In the desert southwest, reservoir designers must plan for evaporation in designing water budget yield curves. Irrigation is usually needed to sustain crops and lawns in areas west of the 100th meridian (central Nebraska, to Texas) where evaporation exceeds precipitation by more than a 2 to 1 margin. Arid and semiarid cities where annual evaporation substantially exceeds precipitation are Phoenix, Los Angeles, San Francisco, Salt Lake City, and Denver (Table 1.9).

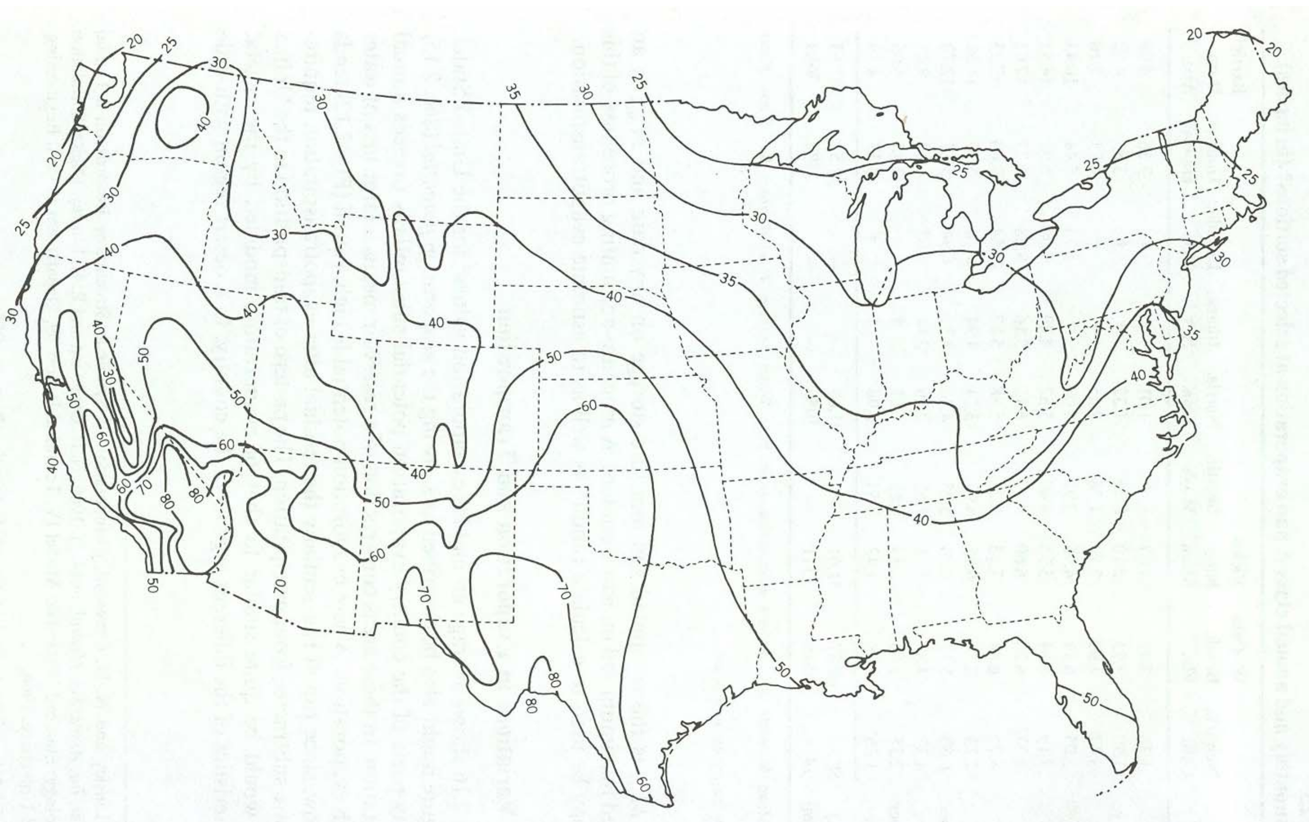


Figure 1.15. Mean annual evaporation in the continental United States in inches (source: U. S. National Weather Service)

Table 1.9. Comparison of annual precipitation and evaporation for major U. S. cities.

City	Annual Precipitation (in)	Annual Lake Evaporation (in)
Atlanta	48	43
Boston	40	28
Chicago	34	30
Cleveland	32	30
Denver	16	40
Houston	48	53

Kansas City	40	45
Los Angeles	16	40
Miami	64	50
Minneapolis	30	30
New Orleans	64	50
New York	48	31
Philadelphia	44	34
Phoenix	16	60
Salt Lake City	20	40
San Francisco	30	40
Seattle	48	25
St. Louis	40	36
Washington, D. C.	48	35

1.6 Transpiration

Transpiration, evaporation, and infiltration are components of precipitation losses in the hydrologic cycle. Transpiration is defined as the water that is absorbed by plants and then released as vapor into the atmosphere through the leaves of the plants. Usually transpiration is combined with evaporation data collectively known as evapotranspiration (ET).

1.7 Infiltration

Infiltration is the component of precipitation that recharges into the groundwater table. The USDA – Natural Resources Conservation Service (NRCS) divides soils into four major groups (A, B, C, and D) depending on runoff potential and infiltration rate.

Group A soils are sands and loams that have low runoff potential and high infiltration rates (greater than 0.3 in/hr)

Group B soils are silt loams that have moderate infiltration rates (0.15 to 0.3 in/hr).

Group C soils are clay loams that have low infiltration rates (0.05 to 0.15 in/hr)

Group D soils silts and clays that have high runoff potential and very low infiltration rates (zero to 0.05 in/hr).

Soil classifications for watersheds are obtained from NRCS soil surveys, which are published for most counties in the United States and are accessed at http://soils.usda.gov/survey/online_surveys/. Figure 1.16 contains an excerpted table of hydrologic soil groups for Bayboro (HSG D), Butlertown (HSG C), and Chester (HSG B) soils from the New Castle County, Soil Survey. Figure 1.17 contains an excerpt of the Sussex County soil survey in Delaware. Soils classifications are then compared to the index of soils in Appendix A of the TR-55 manual to obtain the hydrologic soil group A. B, C, or D.

Table K1.--Water Features--Continued

Map symbol and soil name	Hydro-logic group	Month	Water table		Ponding			Flooding	
			Upper limit	Lower limit	Surface water depth	Duration	Frequency	Duration	Frequency
			Ft	Ft	Ft				
Ba:									
Bayboro--------------------	D								
		January	0.0-1.0	>6.0	---	---	None	---	None
		February	0.0-1.0	>6.0	---	---	None	---	None
		March	0.0-1.0	>6.0	---	---	None	---	None
		April	0.0-1.0	>6.0	---	---	None	---	None
		May	0.0-1.0	>6.0	---	---	None	---	None
		November	0.0-1.0	>6.0	---	---	None	---	None
		December	0.0-1.0	>6.0	---	---	None	---	None
BuA:									
Butlertown-----------------	C								
		Jan-Dec	---	---	---	---	None	---	None
BuB2:									
Butlertown-----------------	C								
		February	2.0-4.0	2.5-4.5	---	---	None	---	None
		March	2.0-4.0	2.5-4.5	---	---	None	---	None
BuC2:									
Butlertown-----------------	C								
		February	2.0-4.0	2.5-4.5	---	---	None	---	None
		March	2.0-4.0	2.5-4.5	---	---	None	---	None
ChA:									
Chester--------------------	B								
		Jan-Dec	---	---	---	---	None	---	None
ChB2:									
Chester--------------------	B								
		Jan-Dec	---	---	---	---	None	---	None
ChC2:									
Chester--------------------	B								
		Jan-Dec	---	---	---	---	None	---	None
ChC3:									
Chester--------------------	B								
		Jan-Dec	---	---	---	---	None	---	None
ChD2:									
Chester--------------------	B								
		Jan-Dec	---	---	---	---	None	---	None
ChD3:									
Chester--------------------	B								

Figure 1.16. Hydrologic soil groups for soils in New Castle County, Delaware (source: USDA- NRCS, New Castle County, Delaware Soil Survey)

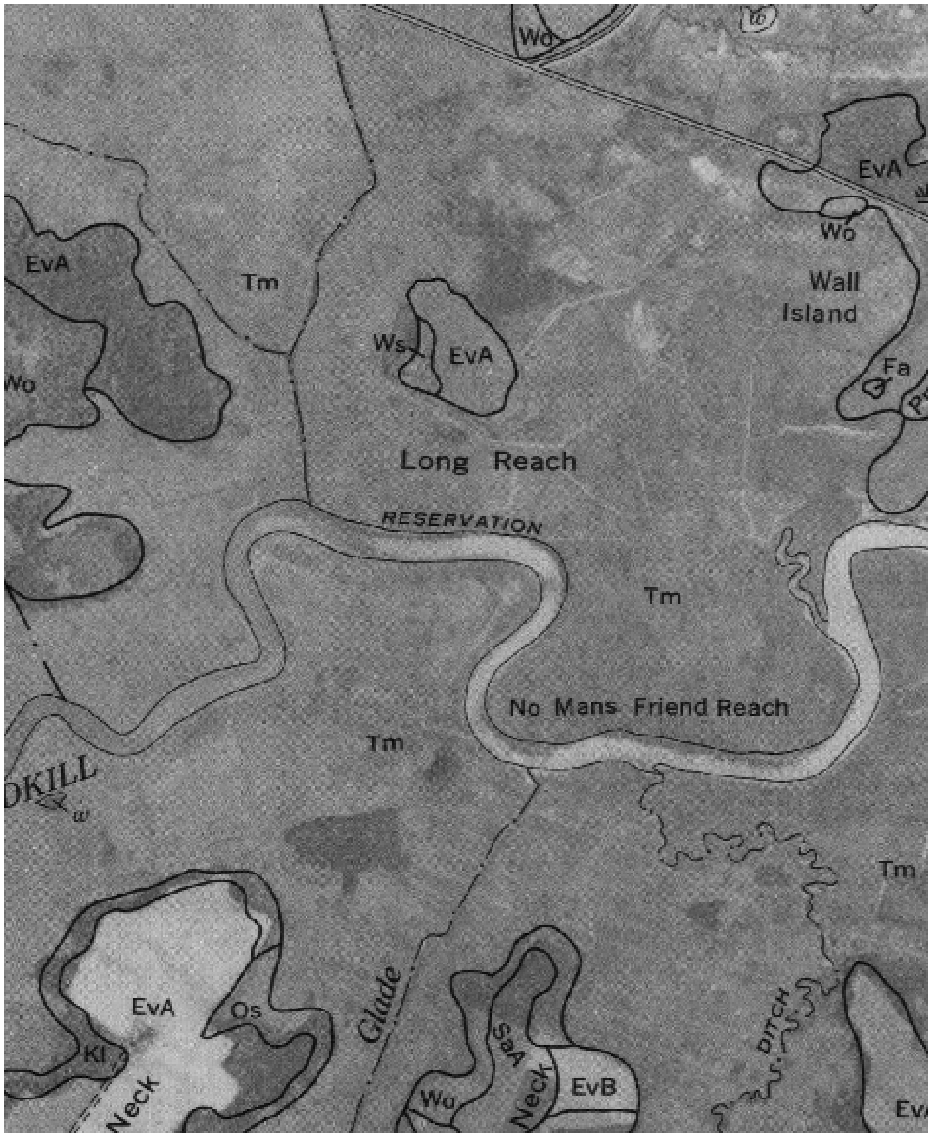


Figure 1.17. Excerpt of soil survey map in Sussex County, Delaware
(source: USDA-NRCS)

The USDA - NRCS curve number method accounts for precipitation losses such as transpiration, evaporation, and infiltration as initial abstraction (I_a) where:

I_a = 0.2(S)

Where:

S = 1000/CN – 10

CN = Runoff curve number obtained for particular land use and hydrologic soil group as defined in Table 2-2 of the TR-55 manual.

Example 1.8

Define the initial abstraction as losses from transpiration, evaporation and infiltration for a watershed with forested land cover with Hydrologic soil group B soils.

From Table 2-2 in the TR55 manual, the curve number (CN) = 65 for forested land and HSG B soils.

I_a = 0.2((1000/CN) – 10)

= (0.2)((1000/65) – 10)

= 1.08 in.

1.8 Runoff Analysis

Water resources engineers commonly perform runoff analysis using two preferred methods:

1. Rational Method – for small watersheds with areas less than 200 acres.

2. USDA NRCS TR-55 Curve Number Method – for watershed areas larger than 200 acres and less than 10 sq mi.

Rational Method

The Rational method is used to estimate runoff for design storms for watersheds less than 200 acres in drainage area. The Rational method equation was developed during the 1850's and is given by:

$$Q = c\ i\ A$$

Where:

Q = runoff (cfs)

c = runoff coefficient

i = rainfall intensity (in/hr)

A = drainage area (acres)

Dimensional analysis of the Rational method indicates the units for Q are in ac-in/hr, which coincidentally correlates to approximately 1.00 cfs. So, the units for Q are measured in cfs. The Rational method is one of the few formulas in engineering where the units do not cancel out.

Runoff coefficients estimate the ratio or percentage of runoff from a particular storm precipitation event. Approximate "c" values for different land use conditions can be obtained from charts in water resources textbooks such as the data in Table 1.10.

Table 1.10. Runoff Coefficients
(Reprinted from Connecticut Department of Transportation. Drainage Manual. October 2000. page 6.9-5.)

Land Use	Coefficient of Runoff (c)
Business: Downtown areas	0.70 – 0.95
Neighborhood areas	0.50 – 0.70
Residential	
Single-family areas	0.30 – 0.50
Multi units, detached	0.40 – 0.60
Multi units, attached	0.60 – 0.75
Suburban	0.25 – 0.40
Residential (> 1.2 ac lots)	0.30 – 0.45
Apartment dwelling areas	0.50 – 0.70
Industrial	
Light areas	0.50 – 0.80
Heavy areas	0.60 – 0.90
Parks, cemeteries	0.10 – 0.25
Playgrounds	0.20 – 0.40
Railroad yard areas	0.20 – 0.40
Unimproved areas	0.10 – 0.30
Street	
Asphalt	0.70 – 0.95
Concrete	0.80 – 0.95
Drives and walks	0.75 – 0.85
Roofs	0.75 – 0.85

The drainage area (A) is measured from topographic maps in usually in terms of acres or square miles.

Rainfall intensity (i) is derived from intensity-duration-frequency (IDF) curves. The storm duration is assumed equal to the time of concentration. The time of concentration is defined as the maximum time for runoff to travel from the farthest and upper most point in a watershed downstream to the outlet point of the watershed. (Figure 1.18)

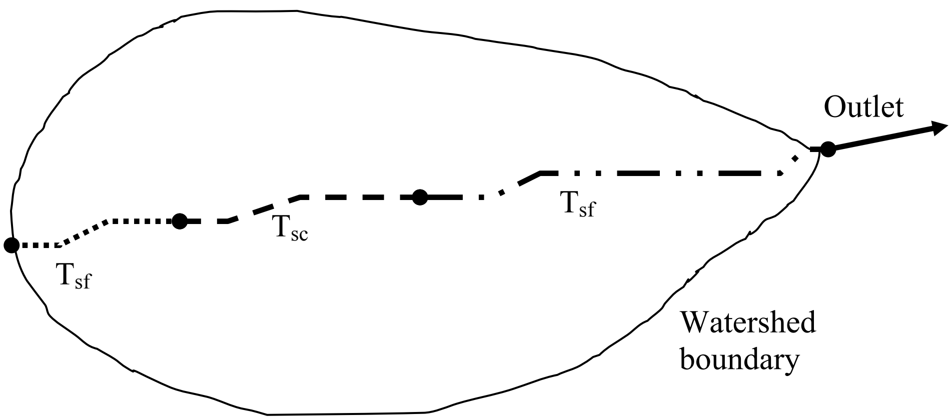


Figure 1.18. Time of concentration in a watershed.

Time of concentration can be estimated using many equations found in water resources engineering textbooks. A preferred and precise method for deriving time of concentration is the equation derived from Worksheet 3 of the TR-55 manual (Figure 1.19) given by:

$T_c = T_{sf} + T_{sc} + T_{ch}$

Where:

T_c = total time of concentration for a particular watershed (hr).

T_{sf} = travel time for sheet flow component no more than 300 feet long (hr).

$$= \frac{0.007\,(nL)^{0.8}}{(P2)^{0.5}(s)^{0.4}}$$

Where:

n = Manning's roughness value from Table 3-1 of TR-55 manual. n = 0.24 for dense grass.

L = length of sheet flow (ft), 300 ft maximum.

P_2 = 2-yr, 24-hr precipitation depth (in) from Figure B-3 of TR-55 (in).

s = slope of sheet flow (ft/ft).

T_{sc} = Travel time for shallow concentrated flow component (hr)

$$= \frac{L}{3600(v)}$$

Where:

L = flow length of shallow concentrated flow (ft).

v = velocity (fps) from Figure 3-1 of TR55, based on slope and paved or unpaved surface.

T_{ch} = travel time for channel flow component (hr).

$$= \frac{L}{3600(v)}$$

Where:
L = flow length of channel flow (ft).

v = velocity in channel derived from Manning's equation (fps)

Once the time of concentration is computed, then select the rainfall intensity (i) from an intensity - duration -frequency curve in Figure 1.14.

Table 3-1 Roughness coefficients (Manning's n) for sheet flow

Surface description	n [1]
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤20%	0.06
Residue cover >20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses [2]	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods:[3]	

Worksheet 3: Time of Concentration (T_c) or travel time (T_t)

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 Tc only)

Segment ID		
1. Surface description (table 3-1)		
2. Manning's roughness coefficient, n (table 3-1)		
3. Flow length, L (total L † 300 ft) .... ft		
4. Two-year 24-hour rainfall, P_2 .... in		
5. Land slope, s .... ft/ft		
6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{\ 0.5}\ s^{0.4}}$ Compute T_t .... 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 (figure 3-1) .... ft/s		
11. $T_t = \frac{L}{3600\ V}$ Compute Tt .... hr	+	=

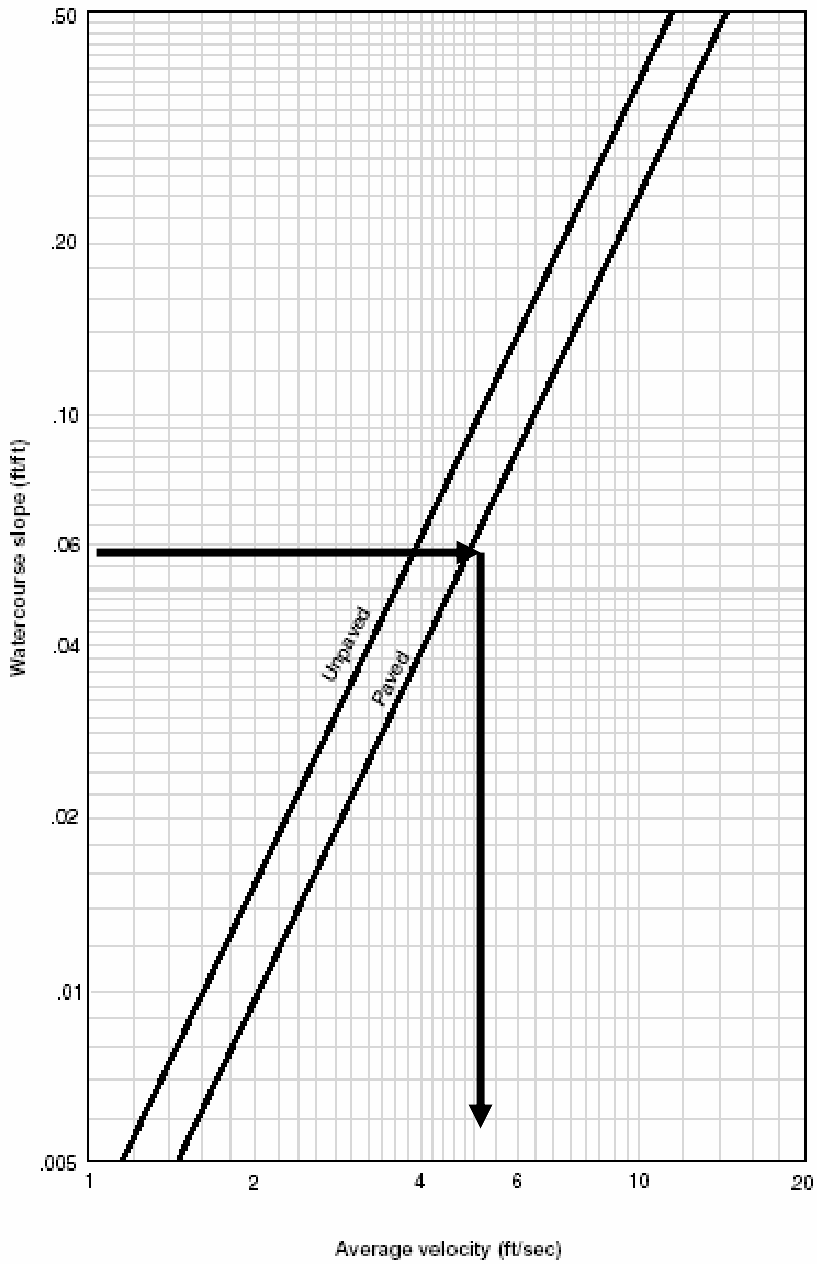
Channel flow

Segment ID		
12. Cross sectional flow area, a .... ft^2		
13. Wetted perimeter, pw .... ft		
14. Hydraulic radius, $r = \frac{a}{Pw}$ Compute r .... ft		
15 Channel slope, s .... ft/ft		
16. Manning's roughness coefficient, n		
17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ....ft/s		
18. Flow length, L .... ft		
19. $T_t = \frac{L}{3600\ V}$ Compute T_t .... hr	+	=

20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19) .... Hr

Figure 1.19. Worksheet 3 from the TR 55 manual for computing Time of Concentration.

Figure 3-1 Average velocities for estimating travel time for shallow concentrated flow



Many watersheds are occupied by more than one land use of land cover condition thus requiring the calculation of a composite runoff coefficient for the Rational Method. The composite "c" value may be calculated by prorating the coefficient by the amount of land cover and then dividing by the total watershed area:

$$c_{total} = \frac{c_1(A_1) + c_2(A_2)}{A_{total}}$$

Where:

c_{total} = composite runoff coefficient

c_1 = runoff coefficient for land use No. 1

c_2 = runoff coefficient for land use No. 2

A_1 = area of land use No. 1

A_2 = area of land use No. 2

Example 1.9

Compute the runoff coefficient for a 100 acre watershed occupied by 50 acres of single family residential land use, 25 acres of light industrial, and 25 acres of park land.

From Table 2.??:

Land use	"c" value	Area (ac)
SF Residential	0.5	50
Light industrial	0.8	25
Park land	0.2	25
		100 ac

$$c_{total} = \frac{0.5(50\text{ ac}) + 0.8(25\text{ ac}) + 0.2(25\text{ ac})}{100\text{ ac}}$$

$$c_{total} = \frac{25 + 20 + 5}{100\text{ ac}} = \frac{50}{100\text{ ac}}$$

$$c_{total} = 0.5$$

Example 1.10

Use the Rational method to estimate the 100 – yr runoff from a 100 – acre watershed covered by single-family residential land use. The dimensions for time of concentration are given as:

Sheet flow

Length = 300 ft

Surface Description = dense grass, n = 0.024 (Figure 2.??, Table 3-1 from TR 55 manual)

2 year, 24 hour rainfall = 3.2 in

Land slope = 0.07 ft/ft

Shallow Concentrated

Surface = unpaved

Flow Length = 1000 ft

Slope = 0.1 ft/ft

Channel Flow

V = 8 fps (from Manning's equation)

Length = 3100 ft

Estimate the coefficient of runoff for single family land use:

c = 0.4 for singe family land use (Table 2.7)

The drainage area is given as:

A = 100 acres

For time of concentration, refer to the formulas in Worksheet 3 from TR 55.

$T_c = T_{sf} + T_{sc} + T_{ch}$

Where:

T_c = total time of concentration for a particular watershed (hr)

T_{sf} = travel time for sheet flow component no more than 300 feet long (hr)

$$= \frac{0.007\ (nL)^{0.8}}{(P2)^{0.5}(s)^{0.4}}$$

$$= \frac{0.007\ (0.024(300\text{ ft}))^{0.8}}{(3.2\text{ in})^{0.5}(0.07\text{ ft/ft})^{0.4}}$$

= 0.35 hr

T_{sc} = travel time for shallow concentrated flow component (hr)

$$= \frac{1000 \text{ ft}}{3600(5 \text{ fps})}$$

$= 0.05$ hr

Where:

L = 1000 ft

v = 5 fps from Figure 3-1 of TR55, based on slope (0.1 ft/ft) and unpaved surface

T_{ch} = travel time for channel flow component (hr)

$$= \frac{L}{3600(v)}$$

$$= \frac{3100 \text{ ft}}{3600 (8 \text{ fps})}$$

$= 0.12$ hr

So the time of concentration is calculated as:

$T_c = T_{sf} + T_{sc} + T_{ch}$

$= 0.35$ hr + 0.05 hr + 0.12 hr

$= 0.52$ hr

From the intensity-duration-frequency curve in Figure 2.??, for Tc = 0.52 hour = 31 minutes, the intensity (i) for a 100 - year storm = 5 in/hr.

Then,

Q_{100} = c i A

Q_{100} = 0.4 (5 in/hr) (100 ac)

Q_{100} = 200 cfs

USDA - NRCS Runoff Curve Number Method

The USDA NRCS runoff curve number (CN) method for calculating runoff is described in the Technical Release 55 (TR-55 manual). All examinees wishing to do well in the water resources portion of the P.E. exam should familiarize themselves with and bring the TR- 55 manual to the exam. According to the NRCS CN method, runoff is calculated by the following formula:

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)}$$

Where:

Q = runoff (in)

P = rainfall (in)

S = potential maximum retention (in) after runoff begins = $I_a/0.2$

I_a = initial abstraction as measured by all losses before runoff begins (in).

$$S = \frac{1000}{CN} - 10$$

CN = Runoff curve number

The TR - 55 manual provides three important worksheets to calculate runoff using the NRCS method:

Worksheet 2: Runoff curve number and runoff

Worksheet 3: Time of Concentration

Worksheet 4: Graphical Peak Discharge Method

Worksheet 2 computes the CN for various soil groups and land cover conditions. Column 1 lists the soil name and hydrologic soil group. Column 2 lists the cover description of the land use or land cover in the watershed. Column 3 lists the curve number for the particular watershed depending on the land cover using Table 2-2 from TR55. Column 4 lists the area of each land use. Column 5 lists the product of the CN times the area. Worksheet 4 is then used to estimate the composite curve number for the particular watershed.

Worksheet 3 is used to compute the time of concentration.

Worksheet 4 is used to compute the peak discharge. The following parameters are needed to compute the peak discharge using the NRCS CN method.

Drainage area (A_m) = _______________ sq mi

Runoff curve number (CN) = _______________ (worksheet 2)

Time of concentration (T_c)	= _______________	hr (worksheet 3)
Rainfall distribution (I, IA, II, III)	= _______________	(Figure B-2,TR 55)
Pond and swamp areas (F_p)	= _______________	(% of A_m)
Frequency	= _______________	yr
Rainfall (P)	= _______________	in
Initial abstraction (I_a)	= _______________	in (Table 4-1)
Compute Ia/P	= _______________	
Unit peak discharge (q_u)	= _______________	(exhibit 4 -)
Runoff (Q)	= _______________	in (worksheet 2)
Pond and swamp areas	= _______________	(% of A_m)
Peak discharge ($q_p = q_u A_m Q F_p$)	= _______________	cfs

Example 1.11

Calculate the 100 - year runoff using the NRCS CN method for a 1000 - acre watershed with 200 acres single family residential (1/4 acre lots), 300 acres row crop in good conditions, and 500 acres wooded in good condition. The time of concentration is 0.5 hrs. Assume a Type II rainfall distribution and that none of the watershed is covered by wetlands. According to the County NRCS soil mapping, all soils in the watershed are Downer soils, Hydrologic Soil Group (HSG) B.

Using Worksheet 2 compute the composite CN:

Soil Name/HSG	**Land Cover**	**CN Table 2-2)**	**Area (ac)**	**Product (CN X Area)**
Downer/B	SF Residential ¼ ac	75	200	15,000
Downer/B	row crop, good condition	78	300	23,400
Downer/B	Wooded, good condition	55	500	27,500
		Total	***1000 ac***	***65,900***

Composite CN = 65,900 /1000 ac= 65.9, rounded to 66.

Using worksheet 3, the time of concentration is computed as 0.5 hrs.

Using worksheet 4, compute the NRCS peak runoff.

Drainage area	A^m	= 1000 ac = 1.6 sq mi
Runoff curve number	CN	= 66 (worksheet 2)
Time of concentration	T_c	= 0.5 hr (worksheet 3)
Rainfall Distribution		= Type II
Pond and swamp areas		= 0 percent of A_m
Frequency		= 100 yr
Rainfall (24 hr)	P	= 7.5 in
Initial abstraction	I_a	= 1.030 in (Table 4-1 using CN = 66)
Compute Ia/P	I_a/P	= 1.030/7.5 = 0.14
Unit peak discharge	q_u	= 500 csm/in (Exhibit 4 – II, for Tc = 0.5 hr and I_a/P = 0.14.
Runoff	Q	= 3.6 in (Figure 2 – 6 for P = 7.5 and CN = 66
Pond and swamp adjustment factor	F_p	= 1.0
Peak discharge	q_p	= q_u(Am)(Q)(F_p)
		= 500 csm/in(1.6 sq mi)(3.6 in)(1.0)
		= 2,880 cfs

Thus, the 100-year runoff using the NRCS TR-55 method is 2,880 cfs.

Worksheet 2: Runoff curve number and runoff

Project	By	Date
Location	Checked	Date

Check one: ☐ Present ☐ Developed

1. Runoff curve number

Soil name and hydrologic group (appendix A)	Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio)	CN [1]			Area ☐ acres ☐ mi^2 ☐ %	Product of CN x area
		Table 2-2	Figure 2-3	Figure 2-4		
				Totals ➡		

[1] Use only one CN source per line

CN (weighted) = $\frac{\text{total product}}{\text{total area}}$ = ________ = ______ ; Use CN ➡ ☐

2. Runoff

		Storm #1	Storm #2	Storm #3
Frequency	yr			
Rainfall, P (24-hour)	in			
Runoff, Q .... (Use P and CN with table 2-1, figure 2-1, or equations 2-3 and 2-4)	in			

Worksheet 3: Time of Concentration (T_c) or travel time (T_t)

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 Tc only)

Segment ID		
1. Surface description (table 3-1)		
2. Manning's roughness coefficient, n (table 3-1)		
3. Flow length, L (total L † 300 ft) .... ft		
4. Two-year 24-hour rainfall, P_2 .... in		
5. Land slope, s .... ft/ft		
6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{\ 0.5}\ s^{0.4}}$ Compute T_t .... 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 (figure 3-1) .... ft/s		
11. $T_t = \frac{L}{3600\ V}$ Compute T_t .... hr	+	=

Channel flow

Segment ID		
12. Cross sectional flow area, a .... ft^2		
13. Wetted perimeter, p_w .... ft		
14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r .... ft		
15 Channel slope, s .... ft/ft		
16. Manning's roughness coefficient, n		
17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ....ft/s		
18. Flow length, L .... ft		
19. $T_t = \frac{L}{3600\ V}$ Compute T_t .... hr	+	=
20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19) .... Hr		

Worksheet 4: Graphical Peak Discharge method

Project	By	Date
Location	Checked	Date

Check one: ☐ Present ☐ Developed

1. Data

Drainage area .... A_m = ______________ mi^2 (acres/640)

Runoff curve number ....CN = ______________ (From worksheet 2)

Time of concentration ....T_c = ______________ hr (From worksheet 3)

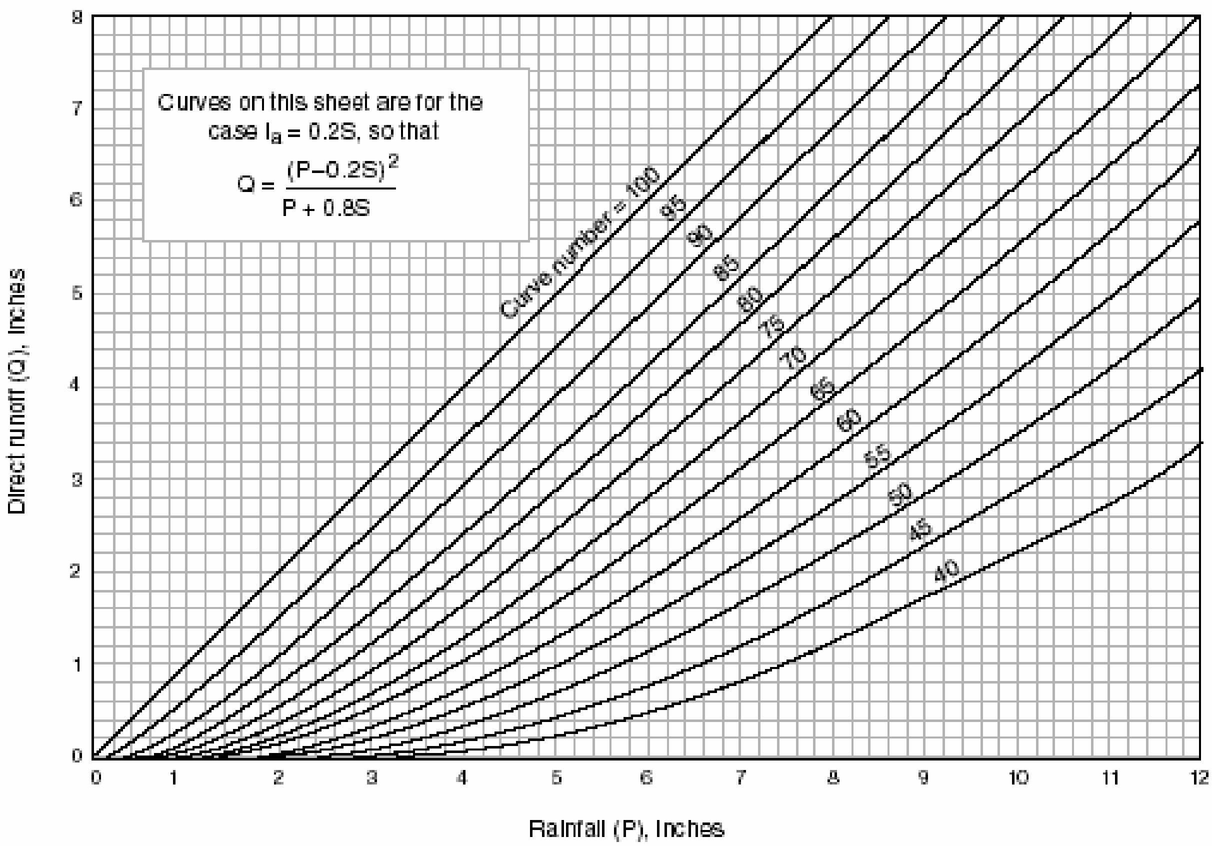
Rainfall distribution .... = _______________ (I, IA, II III)

Pond and swamp areas sprea
throughout watershed .... = ____________ percent of A_m (________ acres or mi^2 covered)

	Storm #1	Storm #2	Storm #3
2. Frequency .... yr			
3. Rainfall, P (24-hour) .... in			
4. Initial abstraction, I_a .... in (Use CN with table 4-1)			
5. Compute I_a / P			
6. Unit peak discharge, q_u .... csm/in (Use T_c and I_a / P with exhibit 4– _____)			
7. Runoff, Q .... in (From worksheet 2) Figure 2-6			
8. Pond and swamp adjustment factor, F_p .... (Use percent pond and swamp area with table 4-2. Factor is 1.0 for zero percent pond ans swamp area.)			
9. Peak discharge, q_p .... ft^3/s			

(Where $q_p = q_u A_m Q F_p$)

Figure 2-1 Solution of runoff equation.



Cover type

Table 2-2 addresses most cover types, such as vegetation, bare soil, and impervious surfaces. There are a number of methods for determining cover type. The most common are field reconnaissance, aerial photographs, and land use maps.

Treatment

Treatment is a cover type modifier (used only in table 2-2b) to describe the management of cultivated agricultural lands. It includes mechanical practices, such as contouring and terracing, and management practices, such as crop rotations and reduced or no tillage.

Hydrologic condition

Hydrologic condition indicates the effects of cover type and treatment on infiltration and runoff and is generally estimated from density of plant and residue cover on sample areas. *Good* hydrologic condition indicates that the soil usually has a low runoff potential for that specific hydrologic soil group, cover type, and treatment. Some factors to consider in estimating the effect of cover on infiltration and runoff are (a) canopy or density of lawns, crops, or other vegetative areas; (b) amount of year-round cover; (c) amount of grass or close-seeded legumes in rotations; (d) percent of residue cover; and (e) degree of surface roughness.

Table 2-1 Runoff depth for selected CN's and rainfall amounts [1/]

Rainfall	Runoff depth for curve number of—												
	40	45	50	55	60	65	70	75	80	85	90	95	98
	inches												
1.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.08	0.17	0.32	0.56	0.79
1.2	.00	.00	.00	.00	.00	.00	.03	.07	.15	.27	.46	.74	.99
1.4	.00	.00	.00	.00	.00	.02	.06	.13	.24	.39	.61	.92	1.18
1.6	.00	.00	.00	.00	.01	.05	.11	.20	.34	.52	.76	1.11	1.38
1.8	.00	.00	.00	.00	.03	.09	.17	.29	.44	.65	.93	1.29	1.58
2.0	.00	.00	.00	.02	.06	.14	.24	.38	.56	.80	1.09	1.48	1.77
2.5	.00	.00	.02	.08	.17	.30	.46	.65	.89	1.18	1.53	1.96	2.27
3.0	.00	.02	.09	.19	.33	.51	.71	.96	1.25	1.59	1.98	2.45	2.77
3.5	.02	.08	.20	.35	.53	.75	1.01	1.30	1.64	2.02	2.45	2.94	3.27
4.0	.06	.18	.33	.53	.76	1.03	1.33	1.67	2.04	2.46	2.92	3.43	3.77
4.5	.14	.30	.50	.74	1.02	1.33	1.67	2.06	2.46	2.91	3.40	3.92	4.26
5.0	.24	.44	.69	.98	1.30	1.65	2.04	2.45	2.89	3.37	3.88	4.42	4.76
6.0	.50	.80	1.14	1.52	1.92	2.35	2.81	3.28	3.78	4.30	4.85	5.41	5.76
7.0	.84	1.24	1.68	2.12	2.60	3.10	3.62	4.15	4.69	5.25	5.82	6.41	6.76
8.0	1.25	1.74	2.25	2.78	3.33	3.89	4.46	5.04	5.63	6.21	6.81	7.40	7.76
9.0	1.71	2.29	2.88	3.49	4.10	4.72	5.33	5.95	6.57	7.18	7.79	8.40	8.76
10.0	2.23	2.89	3.56	4.23	4.90	5.56	6.22	6.88	7.52	8.16	8.78	9.40	9.76
11.0	2.78	3.52	4.26	5.00	5.72	6.43	7.13	7.81	8.48	9.13	9.77	10.39	10.76
12.0	3.38	4.19	5.00	5.79	6.56	7.32	8.05	8.76	9.45	10.11	10.76	11.39	11.76
13.0	4.00	4.89	5.76	6.61	7.42	8.21	8.98	9.71	10.42	11.10	11.76	12.39	12.76
14.0	4.65	5.62	6.55	7.44	8.30	9.12	9.91	10.67	11.39	12.08	12.75	13.39	13.76
15.0	5.33	6.36	7.35	8.29	9.19	10.04	10.85	11.63	12.37	13.07	13.74	14.39	14.76

[1/] Interpolate the values shown to obtain runoff depths for CN's or rainfall amounts not shown.

Table 2-2a Runoff curve numbers for urban areas [1]

Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Average percent impervious area [2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

1 Average runoff condition, and $I_a = 0.2S$.
2 The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.
3 CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.
4 Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.
5 Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4 based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Chapter 4 Graphical Peak Discharge Method

This chapter presents the Graphical Peak Discharge method for computing peak discharge from rural and urban areas. The Graphical method was developed from hydrograph analyses using TR-20, "Computer Program for Project Formulation—Hydrology" (SCS 1983). The peak discharge equation used is:

$$q_p = q_u A_m Q F_p \qquad \text{[eq. 4-1]}$$

where:

q_p = peak discharge (cfs)
q_u = unit peak discharge (csm/in)
A_m = drainage area (mi^2)
Q = runoff (in)
F_p = pond and swamp adjustment factor

The input requirements for the Graphical method are as follows: (1) T_c (hr), (2) drainage area (mi^2), (3) appropriate rainfall distribution (I, IA, II, or III), (4) 24-hour rainfall (in), and (5) CN. If pond and swamp areas are spread throughout the watershed and are not considered in the T_c computation, an adjustment for pond and swamp areas is also needed.

Peak discharge computation

For a selected rainfall frequency, the 24-hour rainfall (P) is obtained from appendix B or more detailed local precipitation maps. CN and total runoff (Q) for the watershed are computed according to the methods outlined in chapter 2. The CN is used to determine the initial abstraction (I_a) from table 4-1. I_a / P is then computed.

If the computed I_a / P ratio is outside the range in exhibit 4 (4-I, 4-IA, 4-II, and 4-III) for the rainfall distribution of interest, then the limiting value should be used. If the ratio falls between the limiting values, use linear interpolation. Figure 4-1 illustrates the sensitivity of I_a / P to CN and P.

Peak discharge per square mile per inch of runoff (q_u) is obtained from exhibit 4-I, 4-IA, 4-II, or 4-III by using T_c (chapter 3), rainfall distribution type, and I_a / P ratio. The pond and swamp adjustment factor is obtained from table 4-2 (rounded to the nearest table value). Use worksheet 4 in appendix D to aid in computing the peak discharge using the Graphical method.

Figure 4-1 Variation of I_a / P for P and CN

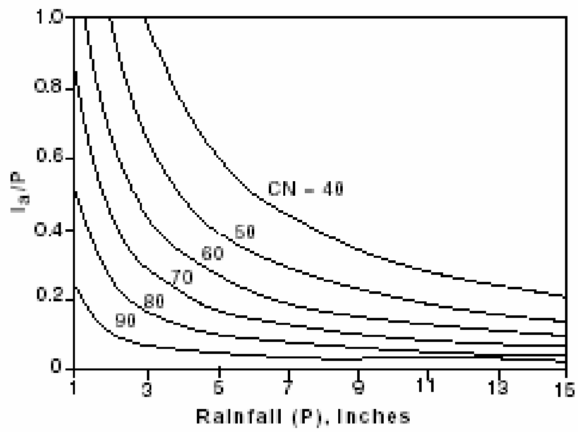
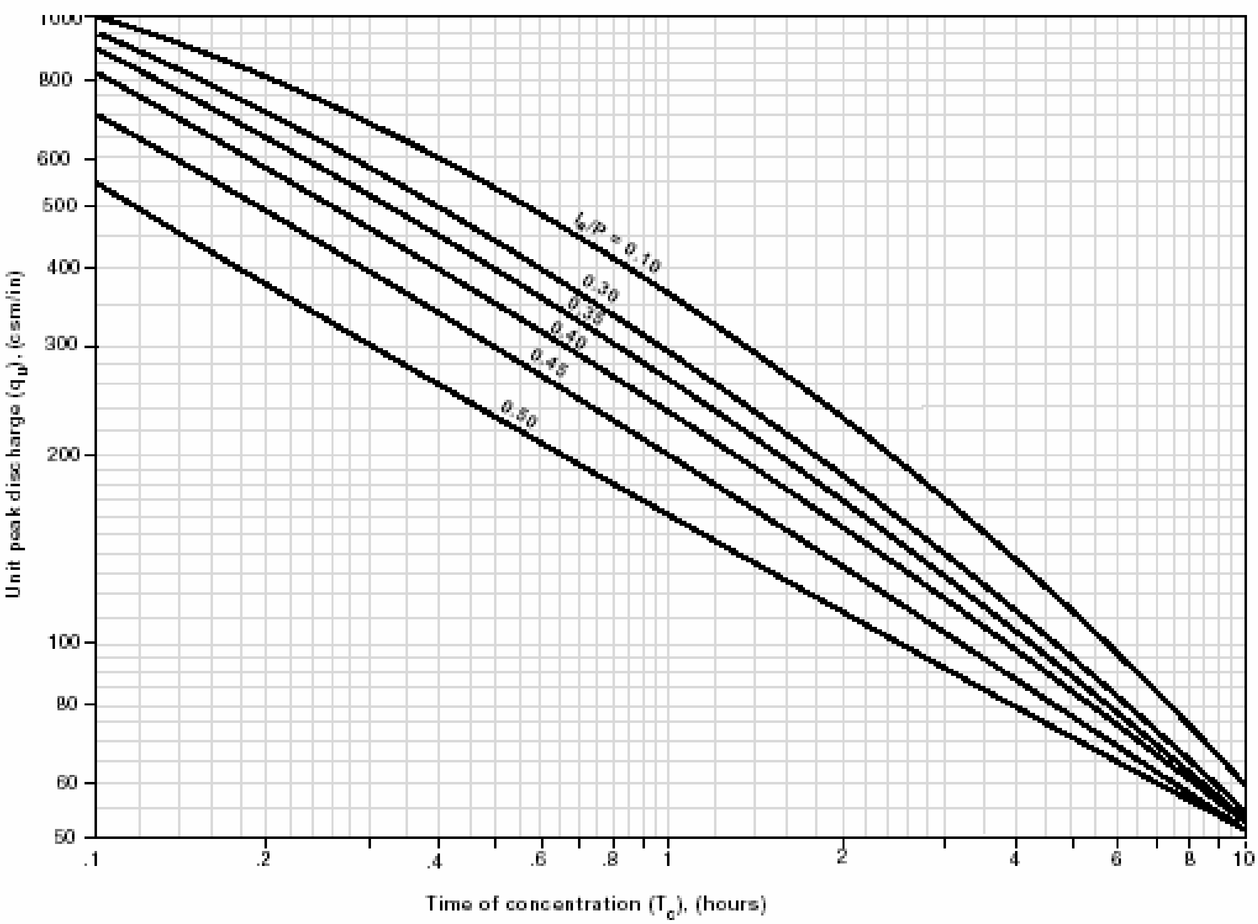


Table 4-1 I_a values for runoff curve numbers

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

Exhibit 4-II Unit peal discharge (q_u) for NRCS (SCS) type II rainfall distribution



DISCOVERY	D
DISHNER	D
DISHNO	C
DISHPAN	C
DISTELL	C
DISWOOD	D
DITCHCAMP	C
DITHOD	B
DITNEY	C
DIVIDE	C
DIVISION	D
DIVOT	C
DIXALETA	D
DIXBORO	B
DIXIEJETT	B
DIXON	B
DIXONVILLE	C
DOANE	B
DOBALT	B
DOBBINS	C
DOBEL	D
DOBENT	C/D
DOBIE	B
DOBSON	D
DOCAS	B
DOCENA	C
DOCKLAKE	B
DOCPAR	B
DODD	D
DODES	B
DODGE	B
DODGECREEK	B
DODGEVILLE	C
DODSON	C
DODY	C/D
DOE	B
DOEL	C
DOGIECREEK	B
DOGLAKE	A
DOGMOUNTAIN	C
DOGTOOTH	D
DOGUE	C
DOKER	C
DOLBEE, Sandy Substratum	B
DOLBEE	C
DOLEKEI	B
DOLEN	B
DOLES	C
DOLLAR	C
DOLLARD	C
DOLLARHIDE	D
DOLLYCLARK	C
DOLMAN	C
DOLUS	C
DOME	B
DOMENGINE	C
DOMERIE	B
DOMEZ	B
DOMINGUEZ	C
DOMINSON	A
DOMKEY	B
DOMO	B
DOMPIER	C
DONA ANA	C
DONAHUE	C
DONALD	C
DONALDSON	C
DONEGAN	C
DONERAIL	C
DONICA	B
DONKEHILL	D
DONLONTON	C
DONNEL	B
DONNELSVILLE	B
DONNING	D
DONNYBROOK	D
DOOH	B
DOOLIN	D
DOONE	B
DORA	B/D
DORERTON	B
DORITTY	B
DORNA	B
DORNA, Thin	C
DOROSHIN	D
DOROTHEA	C
DOROVAN	D
DORRANCE	A
DORS	B
DORSET	B
DORVAL	A/D
DOSA	D
DOSAMIGOS	D
DOSEWALLIPS	D
DOSIE	C
DOSLOMAS	C
DOSS	D
DOSSMAN	B
DOTLAKE	D
DOTSERO	B
DOTSOLOT	D
DOTY	B
DOUBLEDIA	D
DOUBLEO	D
DOUCETTE	B
DOUDLE	B
DOUGAL	D
DOUGAN	C
DOUGCITY	B
DOUGCLIFF	D
DOUGHBOY	B
DOUGHERTY	A
DOUGHSPON	C
DOUGHTY	B
DOUGLAS	B
DOUHIDE	D
DOURO	B
DOUTHIT	B
DOWAGIAC	B
DOWDE	B
DOWELLTON	D
DOWNER	B
DOWNEY	B
DOWNEYGULCH	C
DOWNSOUTH	B
DOWNSVILLE	B
DOWPER	B
DOYLESTOWN	D
DOYN	D
DRAGSTON	C
DRAKE	B
DRAKESFLAT	B
DRAKESPEAK	B
DRAMMEN	A
DRANBURN	B
DRASCO	C
DRAX	B/C
DREKA	D
DRESDEN	B
DREWING	D
DREWSEY	B
DREWSGAP	C
DREXEL	B
DRIFTWOOD	C/D
DRIGGS	B
DRINO	C
DRIVER	C
DROEM	C
DROVAL	D
DRUM	B
DRURY	B
DRY LAKE	C
DRYADINE	C
DRYBED	B
DRYBUCK	B
DRYBURG	B
DRYCK	A
DRYDEN	B
DRYFALLS	B
DRYHOLLOW	B
DRYN	C
DUART	C
DUBACH	B
DUBAY	B
DUBBS	B
DUBBS, Flooded	C
DUBINA	C
DUBLON	B
DUCKABUSH	B
DUCKCLUB	C
DUCKHILL	D
DUCKSTON	A/D
DUELM	A
DUETTE	A
DUFFAU	B
DUFFERN	A
DUFFYMONT	C
DUFFYMONT, Dry	D
DUFUR	B
DUGGINS	D
DUGUESCLIN	D
DUGWAY	C
DUKES	A
DULA	D
DULAC	C
DULANDY	B
DULCE	D
DULEYLAKE	C
DULLAXE	B
DULLES	D
DUMAS	B
DUMFRIES	B
DUMONT	D
DUMPS, Tailings	B
DUNBAR	D
DUNBRIDGE	B
DUNC	C
DUNCANNON	B
DUNFORD	C
DUNGAN	B
DUNGENESS	B
DUNKIRK	B
DUNKLEBER	D
DUNLATOP	B
DUNMORE	B
DUNNBOT	B
DUNSMUIR	B
DUNSMUIR, Nongravelly	C
DUNTON	C
DUPLIN	C
DUPO	C
DUPREE	D
DURADOS	A
DURALDE	C
DURAND	B
DURANGO	B
DURANT	D
DURAZO	A
DURBIN	D
DURELLE	B
DURKEE	C
DURRSTEIN	D
DURSTON	C
DUSEN	B
DUSKPOINT	A
DUSLER	C
DUSON	C
DUSTON	A
DUSTY	B
DUTCHATT	B
DUTCHCANYON	B
DUTCHENRY	C
DUTCHFLAT	C
DUTCHJOHN	B
DUTEK	A
DUTTON	C
DUVAL	B
DUZEL	C
DWARF	D
DWORSHAK	B
DYE	D
DYERHILL	B
DYLAN	D
DYNAL	A
EACHUS	B
EAD	C
EAGAR	B
EAGLECAP	B
EAGLECONE	B
EAGLECREEK	B
EAGLELAKE	B
EAGLEPOINT	D
EAGLEROCK	C
EAGLESNEST	C
EAGLESON	C
EAGLESPRING	B
EAGLETON	D
EAGLEVIEW	A
EAGLEVILLE	D
EAGLEWING	B
EAGLEYE	D
EAGREEK	B
EAKIN	B
EALY	B
EAPA	B
EARLE	D
EARLMONT	C/D
EARP	B
EASBY	C
EASLEY	C
EASPUR	B
EAST LAKE	A
EASTABLE	B
EASTCHOP	A
EASTHAM	D
EASTPARK	D
EASTPINE	B
EASTWOOD	D
EASYCHAIR	B
EATONCREEK	D
EAUCLAIRE	A
EAUGALLIE	D
EBADLOW	B
EBAL	B
EBBERT	C/D
EBBING	C
EBBS	B
EBIC	D
EBODA	B
EBODA, Stony	C
EBRO	D
ECHAW	A
ECHETA	C
ECKERT	D
ECKHART	B
ECKLUND	B
ECKMAN	B
ECKRANT	D
ECKVOLL	B
ECLETO	D
ECLIPSE	B
ECOLA	C
ECON	B
ECONFINA	A
ECUR	B
EDA	A
EDALFRED	A
EDALGO	C
EDDINGS	B
EDDS	B
EDDY	C
EDEMAPS	C
EDENBOWER	D
EDENTON	C
EDENVALLEY	B
EDGAR	B
EDGEHILL	C
EDGELEY	C
EDGEMERE	D
EDGEWATER	C
EDGEWICK	C
EDGINGTON	C/D
EDINBURG	C/D
EDISTO	C
EDJOBE	C
EDMINSTER	D
EDMORE	D
EDMUNDSTON	B
EDOM	C
EDROY	D
EDSON	C
EDWARDS	B/D
EDWARDSVILLE	B/D
EDWIN	B
EELCOVE	D
EELWEIR	C
EENREED	B
EEP	C
EFFIE	C
EFFINGTON	C

1.9 Gauging Stations

Along streams where the United States Geological Survey has established stream gages, discharges can be estimated at various locations upstream and downstream by the ratio of drainage areas method where:

$$\frac{Q_s}{A_s} = \frac{Q_g}{A_g}$$

Where:

As = drainage area (sq mi)to the site in question.

A_g = drainage area (sq mi)to the stream gage.

Q_s = peak flow (cfs) to the site in question.

Q_g = peak flow (cfs) as measured at the stream gage.

Example 1.12

Determine the 100 year peak discharge at a site along the Red River with a drainage area of 100 sq mi. The 100 year peak discharge at the USGS stream gage along the Red Rover is 200 cfs and the stream gage has a drainage area of 150 sq. mi.

$$\frac{Q_s}{A_s} = \frac{Q_g}{A_g}$$

The 100 year flow at the site is expressed as:

$$Q_s = \frac{Q_g(A_s)}{(A_g)}$$

$$= \frac{200 \text{ cfs}(100 \text{ sq mi})}{(150 \text{ sq mi})}$$

$$Q_s = 133 \text{ cfs}$$

1.10 Floodplain and Floodway Analysis

Many jurisdictions seek to prevent flood damage by enacting floodplain ordinances. Most ordinances seek to regulate and prevent development in the 100 –year floodway and floodplain of waterways. Floodplains are delineated by calculating the 100 – year flood depth (D_{100}) using the Manning's equation and the standard step back water calculations. Once the flood depth is known, it is plotted on topographic maps to delineate the floodplain. The floodway is the portion of the floodplain that includes the stream channel and the adjacent portion of the floodplain reserved to carry the greatest portion of the flood waters. The bottom of the channel is called the *thalweg*. Figure 1.20 depicts the floodway and floodplain on a stream cross-section. Figure 1.21 delineates a floodplain on a topographic map.

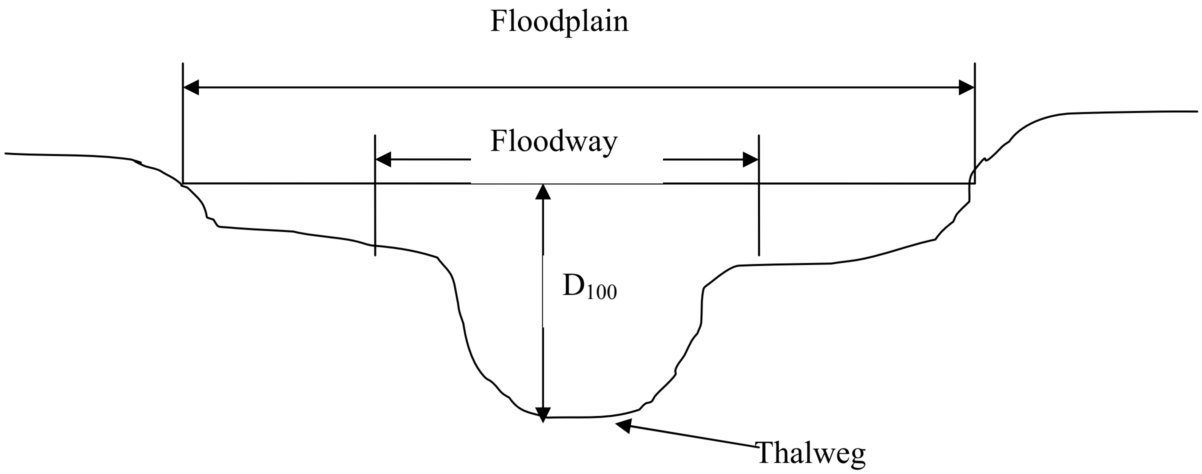


Figure 1.20. The floodplain and floodway.

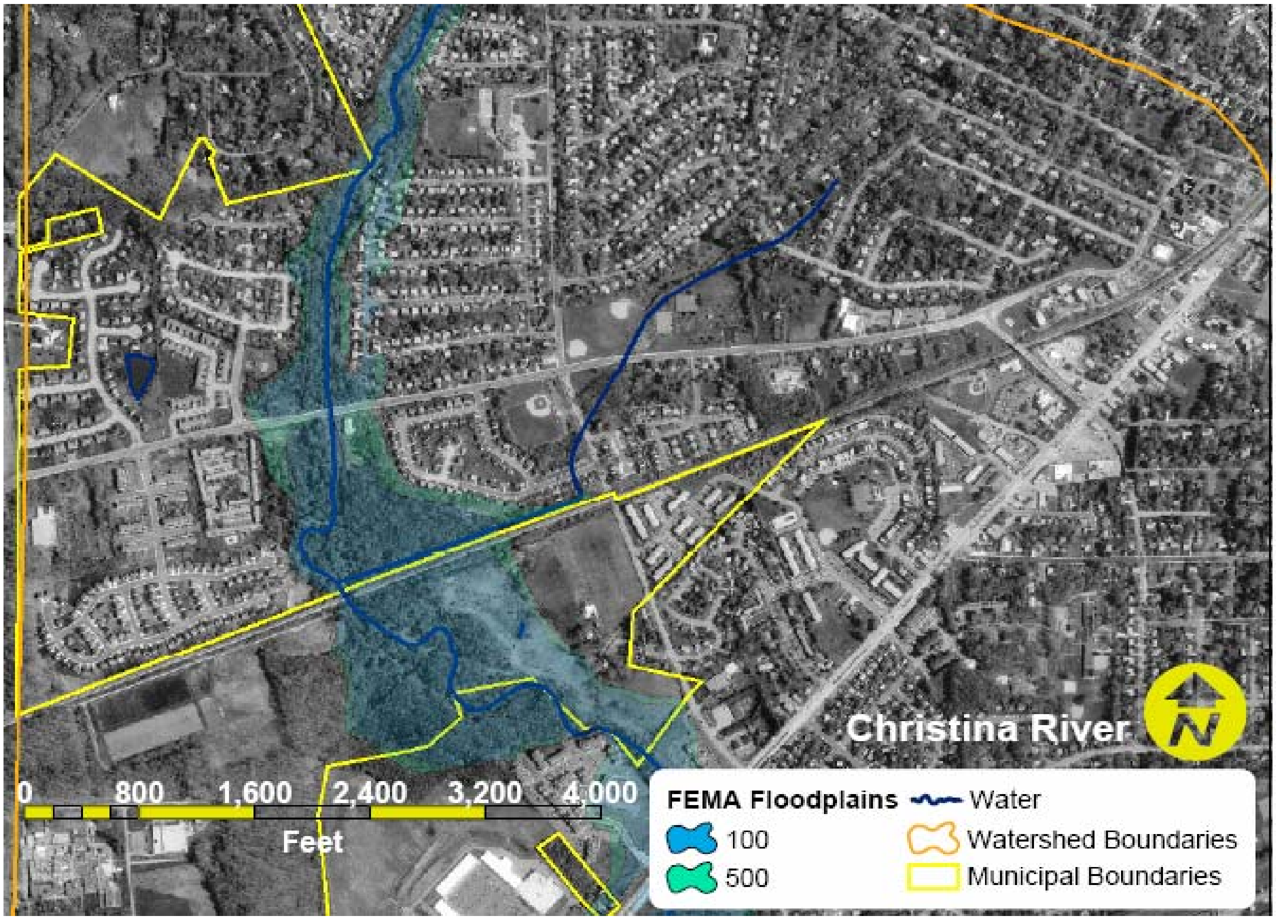


Figure 1.21. Floodplain mapping along the Christina River in Newark, Delaware. (sources: Federal Emergency Management Agency (FEMA) and University of Delaware, Water Resources Agency).

The floodplain is delineated by calculating the flood elevation and then plotting the elevation on aerial photographs and topographic maps as depicted in Figure 1.21. The flood elevation is calculated by hand using Manning's equation or the standard step backwater method. Since the floodplain condition often varies from the channel roughness, several Manning's "n" values are often used in a composite Manning's formula equation. Hydraulic computer models such as the U.S. Army Corps of Engineers HEC-RAS model are often used to delineate floodplains for the Federal Emergency Management Agency (FEMA) Flood Insurance Studies.

Example 1.13

Calculate the 100 – yr flood flow for the floodplain cross section shown in Figure 1.22 where the 100 year flood elevation = 100 ft msl. The left and right overbanks of the floodplain are forest and the channel is a natural stream. The thalweg of the stream cross section 100 ft downstream is at elevation 88 ft msl.

5 ft

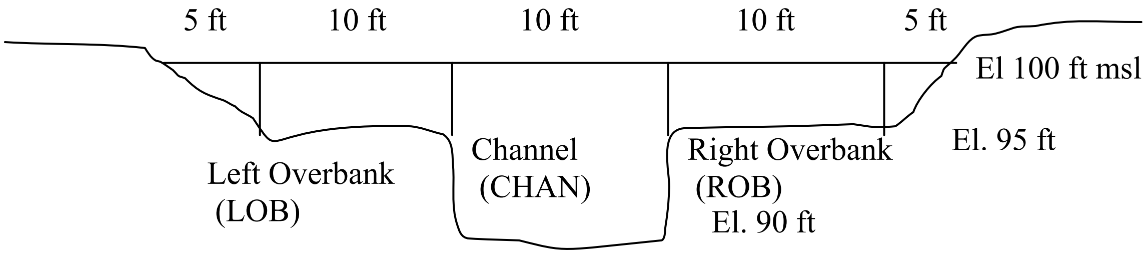


Figure 1.22. Floodplain cross section

Use Manning's equation to calculate the 100 year flood flow (Q_{100}).

$$Q_{100} = \frac{1.49}{n} R^{2/3} S^{1/2} (A)$$

Left Over Bank (LOB)

$n = 0.150$ (forest, Table 1.??)

$A_{LOB} = 5 \text{ ft}(100 \text{ ft} - 95 \text{ ft})/2 + 5 \text{ ft}(10 \text{ ft}) = 5(5)/2 + 50 \text{ ft} = 62.5 \text{ sf}$

$WP_{LOB} = 10 \text{ ft} + (5^2 + 5^2)^{1/2} = 10 \text{ ft} + 7 \text{ ft} = 17 \text{ ft}$

$R_{LOB} = A/P = 62.5 \text{ sf}/17 \text{ sf} =$

$S = (90 \text{ ft} - 88 \text{ ft})/1000 \text{ ft} = 0.002 \text{ ft/ft}$

$$Q_{LOB} = \frac{1.49}{0.150} R^{2/3}\, 0.002^{1/2} (62.5)$$

$Q_{LOB} =$ cfs

Right Over Bank (ROB)

$n = 0.150$ (forest, Table 1.??)

$A_{ROB} = 5 \text{ ft}(100 \text{ ft} - 95 \text{ ft})/2 + 5 \text{ ft}(10 \text{ ft}) = 5(5)/2 + 50 \text{ ft} = 62.5 \text{ sf}$

$WP_{ROB} = 10 \text{ ft} + (5^2 + 5^2)^{1/2} = 10 \text{ ft} + 7 \text{ ft} = 17 \text{ ft}$

$R_{ROB} = A/P = 62.5 \text{ sf}/17 \text{ sf} =$

$S = (90 \text{ ft} - 88 \text{ ft})/1000 \text{ ft} = 0.002 \text{ ft/ft}$

$$Q_{ROB} = \frac{1.49}{0.150} R^{2/3}\, 0.002^{1/2}\, (62.5)$$

Q_{ROB} = cfs

Channel (CHAN)

n = 0.0.030 (natural channel, Table 1.??)

A_{CHAN} = 10 ft(100 ft – 90 ft) = 10(10) = 100 sf

WP_{CHAN} = 10 ft + 5 ft + 5 ft = 20 ft

R_{CHAN} = A/P = 100 sf/20 ft = 5 ft

S = (90 ft – 88 ft)/1000 ft = 0.002 ft/ft

$$Q_{CHAN} = \frac{1.49}{0.030} R^{2/3}\, 0.002^{1/2}\, (100)$$

Q_{CHAN} = cfs

The 100 year flood flow in the stream cross section is then:

$Q100 = Q_{LOB} + Q_{ROB} + Q_{CHAN}$
Q100 = cfs + cfs + cfs = cfs

1.11 Sedimentation

The flood carrying capacity of streams and waterways is often diminished by sedimentation that reduces the hydraulic cross section of the channel and floodplain. Sedimentation also can pollute streams by increasing turbidity and damaging stream bottom habitat

Total suspended sediment (TSS) loads produced by a watershed can be estimated by the "simplified method (Schueler, 1987):

L = (A) (P) (R) (C) (0.2260)

Where:

L = annual pollutant load, lb/yr.

A = watershed area, acres.

P = mean annual precipitation, in.

R = runoff coefficient for various land uses (Table 1.11).

C = mean pollutant concentration (mg/l) for TSS (Table 1.11).

Table 1.11. Total Suspended Sediment Load Variables
(Source: USEPA)

Land Use	**Mean TSS Concentration (C) (mg/l)**	**Runoff Coefficient (R)**
Single family residential (1/2 acre/DU and greater)	140	0.30
High Density/Multi-family resid.	180	0.65
Office	175	0.60
Industrial	251	0.72
Transportation/Utility	350	0.90
Commercial	168	0.85
Institutional	128	0.55
Open Space/Parks	20	0.20
Wooded/Forested	20	0.20
Agriculture	300	0.30

Example 1.14

Compute the total suspended sediment load for a 198 ac watershed with the following land cover values: forest (99 ac), residential ½ ac lots (16 ac), and agricultural hay (83 ac). The annual precipitation is given as 41 in.

Compute the sediment loads for each of the land uses according to the simplified method:

L = (A) (P) (R) (C) (0.2260)

L_{Forest} = (99 ac) (41 in.) (0.20) (20 mg/l) (0.226) = 3,669 lb/yr

$L_{Residential}$ = (16 ac) (41 in) (0.30) (140) mg/l) (0.226) = 6,226 lb/yr

$L_{Agriculture}$ = (83 ac) (41 in) (0.30) (300 mg/l) (0.226) = 69, 217 lb/yr

Total Suspended Sediment Load = 79,112 lb/yr

Unit TSS Load= 79,112 lb/yr /198 ac. = 399 lb/ac/yr.

1.12 Hydrology Sample Problems

Problem 1.1

In the mid Atlantic region of the United States, a meteorological station records the following readings in 2005. The annual precipitation is 45 in, infiltration is 10 in, evapotranspiration is 24 inches, and change in moisture storage is 0. Calculate annual runoff in (a) inches and (b) acre-feet if the watershed area is 1 square mile.

Using the water budget equation:

$$P = R + I + ET - \Delta S$$

$$45 \text{ in} = R + 10 \text{ in} + 24 \text{ in} -$$

$$R = 45 \text{ in} - 10 \text{ in} - 24 \text{ in}$$

$$R = 11 \text{ in}$$

Convert runoff to ac-ft:

$$R = 11 \text{ in } (1 \text{ ft} / 12 \text{ in}) (1 \text{ sq mi}) (640 \text{ ac} / \text{sq mi})$$

$$R = 587 \text{ ac} - \text{ft}$$

Example 1.2

What is the probability f(n) that a 10 – yr storm occurs twice in 5 years.

Utilize the equation:

$$f(n) = \frac{(N!)\,(P)^n\,(1-p)^{N-n}}{n!\,(N-n)!}$$

Where:

N = 5 yr

n $= 2$

p $= 1/T = 1/10 \text{ yr} = 0.1$

$$f(2) = \frac{(5!)\,(0.1)^2\,(1-0.1)^{5-2}}{2!(5-2)!}$$

$$= \frac{(5)(4)(3)(2)(1)\,(0.01)\,(0.729)}{(2)(1)(3)(2)(1)}$$

$$= 0.074$$

f(2) $= 7.4\ \%$

There is a 7.4 % probability that a 10-year storm can occur twice in 5 years, or a 1 in 13 chance.

Problem 1.3

Define the unit hydrograph for the hydrograph in Figure 1.?? assuming the watershed area is 1.0 sq mi. Then define the hydrograph for this watershed for a 3 inch storm.

Find the average precipitation for the watershed.

P $= V/A$

$$= \frac{1{,}368{,}000 \text{ cf (12 in/ft)}}{1.0 \text{ sq mi (640 ac/sq mi)(43560 sf/ac)}}$$

$$= 0.6 \text{ in}$$

The coordinates for the 1.0 inch unit hydrograph (column 3) are computed by dividing each runoff point in column 2 by 0.6I inch as shown in the following table.

The hydrograph for a 3 inch storm in the watershed (column 4) is computed by multiplying the unit hydrograph runoff in column 3 by 3 inches.

(1) Time (hrs)	(2) 0.6 inch Hydrograph Runoff (cfs)	(3) Unit Hydrograph Runoff (cfs/in)	(4) 3.0 in Hydrograph Runoff (cfs)
12.6	60	60/0.6 = 100	100(3) = 300
14.0	155	155/0.6 = 258	258(3) = 774
15.4	250	417	1,251
16	137	228	684
17	49	82	246

Problem 1.4

Define a synthetic unit triangular hydrograph where the peak runoff is estimated as 150 cfs and the time of concentration is 1.5 hrs.

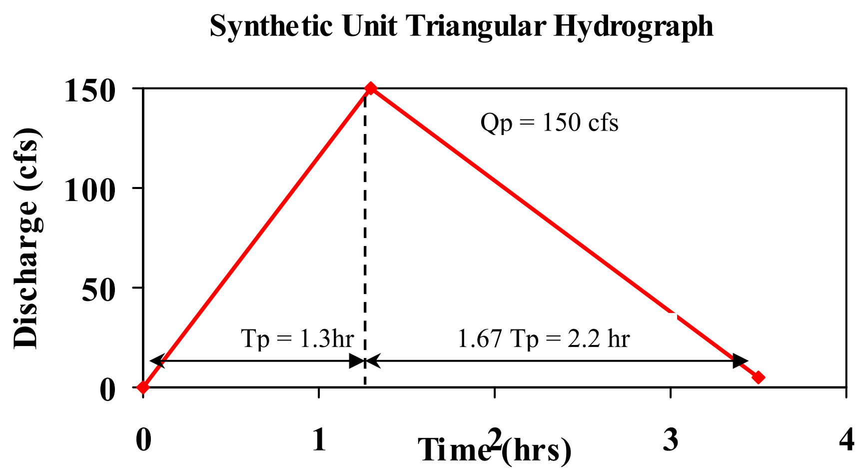


Figure 1.23. Synthetic unit triangular hydrograph.

Referring to Figure 1.23 the peak runoff is plotted at 150 cfs.

The ascending limb of the hydrograph = time to peak (Tp) = 0.67(Tc) = 0.67(2.0 hr) = 1.3 hr.

The receding limb of the hydrograph = 1.67 (Tp) = 1.67 (1.3 hr) = 2.2 hr.

Problem 1.5

Use the Rational method to estimate the 10 – yr runoff from a 200 – acre watershed covered by business land use. The dimensions for time of concentration are given as:

Sheet flow

  Length = 300 ft
  Surface Description = dense grass
  2 year, 24 hour rainfall = 3.2 in
  Land slope = 0.02 ft/ft

Shallow Concentrated

  Surface = unpaved
  Flow Length = 500 ft
  Slope = 0.1 ft/ft

Channel Flow

  V = 3 fps (from Manning's equation)
  Length = 2000 ft

A = 200 acres

C = 0.6 for single family land use (Table 2.7)

For time of concentration, refer to calculations in Worksheet 3 from TR 55.

$T_c = T_{sf} + T_{sc} + T_{ch}$

Where:

T_c = total time of concentration for a particular watershed (hr)

T_{sf} = travel time for sheet flow component no more than 300 feet long (hr)

$$= \frac{0.007\ (nL)^{0.8}}{(P2)^{0.5}(s)^{0.4}}$$

$$= \frac{0.007\ (0.24(300\ ft))^{0.8}}{(3.2\ in)^{0.5}(0.02\ ft/ft)^{0.4}}$$

= 0.13 hr

T_{sc} = travel time for shallow concentrated flow component (hr)

$$= \frac{500\ ft}{3600(5\ fps)}$$

= 0.03 hr

Where:

L = 1000 ft

v = 5 fps from Figure 3-1 of TR55, based on slope (0.1 ft/ft) and unpaved surface

T_{ch} = travel time for channel flow component (hr)

$$= \frac{L}{3600(v)}$$

$$= \frac{2000\ ft}{3600\ (3\ fps)}$$

= 0.18 hr

So the time of concentration is calculated as:

$$T_c = T_{sf} + T_{sc} + T_{ch}$$

$$= 0.13 \text{ hr} + 0.03 \text{ hr} + 0.18 \text{ hr}$$

$$= 0.34 \text{ hr}$$

From the intensity-duration-frequency curve in Figure 1.??, for Tc = 0.34 hour = 20 minutes, the intensity (i) for a 10 - year storm = 4.2 in/hr.

Then,

$$Q_{10} = c\ i\ A$$

$$Q_{10} = 0.6\ (4.2 \text{ in/hr})\ (200 \text{ ac})$$

$$Q_{10} = 504 \text{ cfs}$$

Problem 1.6

Calculate the 100 - year runoff using the NRCS CN method for a 500 - acre watershed in the State of Delaware with 100 acres single family residential (1/3 acre lots), 100 acres row crop in good conditions, and 300 acres wooded in good condition. The time of concentration is 1.0 hr. Assume a Type II rainfall distribution and that none of the watershed is covered by wetlands. According to the County NRCS soil mapping, all soils in the watershed are Downer soils Hydrologic Soil Group (HSG) B. Using Worksheet 2 compute the composite CN:

Using Worksheet 2 compute the composite CN:

Soil Name/HSG	**Land Cover**	**CN (Table 2-2)**	**Area (ac)**	**Product (CN X Area)**
Downer/B	SF Residential 1/3 ac	72	100	7,200
Downer/B	row crop, good condition	78	100	7,800
Downer/B	Wooded, good condition	55	300	16,500
		Total	***500 ac***	***31,500***

Composite CN = 31,500 /500 ac= 63.

Using worksheet 3, the time of concentration is given as 1.0 hr.

Using worksheet 4, compute the NRCS peak runoff.

Drainage area	A^m	= 500 ac = 0.8 sq mi
Runoff curve number	CN	= 63 (worksheet 2)
Time of concentration	T_c	= 1.0 hr (worksheet 3)
Rainfall Distribution		= Type II
Pond and swamp areas		= 0 percent of A_m
Frequency		= 100 yr
Rainfall (24 hr)	P	= 7.5 in
Initial abstraction	I_a	= 1.175 in (Table 4-1 using CN = 63)
Compute Ia/P	I_a/P	= 1.175 in/7.5 in = 0.16
Unit peak discharge	q_u	= 340 csm/in (Exhibit 4 – II, for Tc = 1.0 hr and I_a/P = 0.16
Runoff	Q	= 3.4 in (Figure 2 – 6 for P = 7.5 and CN = 63
Pond and swamp adjustment factor	F_p	= 1.0
Peak discharge	q_p	= q_u(Am)(Q)(F_p)
		= 340 csm/in(0.8 sq mi)(3.4 in)(1.0)
		= 925 cfs

Thus, the 100-year runoff using the NRCS TR-55 method is 92 cfs.

Problem 1.7

Determine the 100 year peak discharge at a site along the White River with a drainage area of 1,000 sq mi. The 100 year peak discharge at the USGS stream gage along the White River is 10,000 cfs and the stream gage has a drainage area of 900 sq. mi.

The ratio of drainage areas formula is:

$$\frac{Q_s}{A_s} = \frac{Q_g}{A_g}$$

The 100 year flow at the site is expressed as:

$$Q_s = \frac{Q_g(A_s)}{(A_g)}$$

$$= \frac{10{,}000 \text{ cfs}(1{,}000 \text{ sq mi})}{(900 \text{ sq mi})}$$

$$Q_s = 11{,}111 \text{ cfs}$$

Problem 1.8

Compute the total suspended sediment load for a 1000 - ac watershed with the following land cover values: forest (500 ac), residential ½ ac lots (200 ac), and agricultural hay (300 ac). The normal annual precipitation is given as 50 in.

Compute the sediment loads for each of the land uses according to the simplified method:

L	= (A) (P) (R) (C) (0.2260)	
L_{Forest}	= (500 ac) (50 in.) (0.20) (20 mg/l) (0.226)	= 22,600 lb/yr
$L_{Residential}$	= (200 ac) (50 in) (0.30) (140) mg/l) (0.226)	= 94,920 lb/yr
$L_{Agriculture}$	= (300 ac) (50 in) (0.30) (300 mg/l) (0.226)	= 305,100 lb/yr
Total Suspended Sediment Load		= 422,620 lb/yr
Unit TSS Load = 422,620 lb/yr /1000 ac.		= 423 lb/ac/yr.

Problem 1.9

Given the 500 - acre watershed in Figure 1.25 with land use as single family residential (1/3 acre lots) and underlain by hydrologic soil group B Soils, compute the 100-year peak discharge (cfs) using the Rational method.

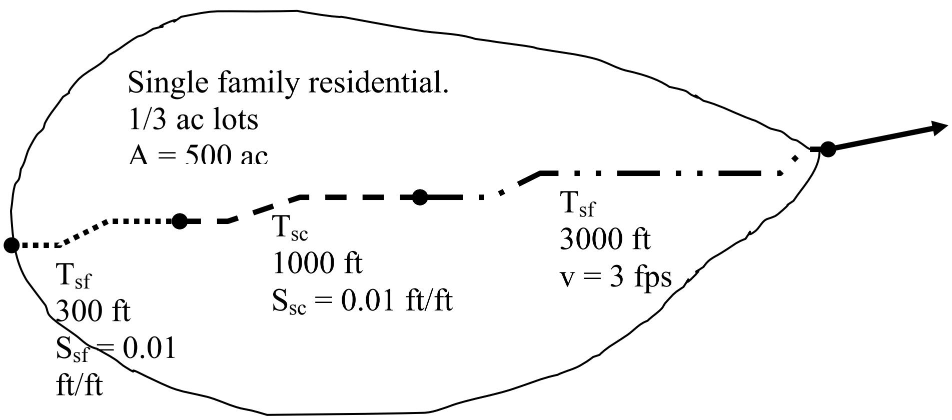


Figure 1.25. Watershed diagram.

Using the Rational method,
Q_{100} = c i A

Given the time of concentration parameters:

Sheet flow:
S_{sf} = 0.01 ft/ft
n for sheet flow = 0.03, forest cover
L for sheet flow = 300 ft.

Shallow concentrated flow:
S_{sc} = 0.01 ft/ft, paved
L = 1000 ft.

Channel Flow:
Velocity in Channel (v_{ch}) = 3 fps
L = 3000 ft.

Determine time of concentration (T_c) using worksheet 3:

$$T_{sf} = \frac{0.007(0.03(300\text{ ft}))^{0.8}}{(3.5\text{ in})^{0.5}\,(0.01\text{ ft/ft})^{0.4}} = 0.13\text{ hr.}$$

Tsc: For slope (s) = 0.01 ft/ft and surface = paved, then velocity (v) = 2 fps.

$$T_{sc} = \frac{1000\text{ ft}}{3600(2\text{ fps})} = 0.14\text{ hr.}$$

$$T_{ch} = \frac{3000\text{ ft}}{3600\,(3\text{ fps})} = 0.28\text{ hr}$$

$$T_c = 0.13 + 0.14 + 0.28 = 0.55\text{ hr} = 33\text{ min.}$$

Determine rainfall intensity (i)

For Tc = 33 min and 100 yr storm, i = 4.9 in/hr from IDF curve Figure 2.??

Determine runoff coefficient (c):

Refer to Table 2.??, for single family residential 1/3 acre lots, select c = 0.5.

Determine 100-yr runoff using Rational method:

Q_{100} = c I A

= 0.5 (4.9 in/hr) (500 acres)

Q_{100} = 1225 cfs

Problem 1.10

Compute the 100-year peak discharge for Philadelphia, Pennsylvania for the same watershed in Figure 2.20 using the NRCS TR-55 Curve number peak discharge method.

Using Worksheet 4:

A	= 500 acres/640 acres/sq. mi. = 0.78 sq.mi.
CN	= 72 for HSG B and land use = 1/3 acre residential
T_c	= 0.55 hr (computed above in Problem 2.18)
Rainfall Distribution	= Type II
Pond/Swamp adjustment	= 1.0
Frequency	= 100-yr
100-yr, 24-hr Rainfall (P)	= 7.5 in/hr (Philadelphia, Pennsylvania)
I_a	= 0.778 in for CN = 72
I_a/P	= 0.778 in/7.5 in = 0.10
Unit Peak Discharge (q_u)	= 500 csm/in for T_c = 0.55 hr. and Ia/P = 0.10
Runoff (Q)	= 4.3 in. interpolating for P = 7.5 in. and CN = 72
100-yr. Peak Discharge (Q_p)	= (q_u)(A)(Q) = 500 csm/in(0.78 sq. mi.)(4.3 in) = 1,677 cfs

2.0 Hydraulics

Hydraulics is the study of the flow of water in motion through natural and engineered systems, usually closed conduit pipe and open channel flow. Closed conduit flow is flow under pressure usually through pipes or storage tanks in a drinking water distribution network or a reservoir pump storage pipeline. Open channel flow is open to the atmosphere (not under pressure) through streams, rivers and floodplains or flow through storm water collection systems or sanitary sewers. Fundamental equations of closed conduit flow include the Darcy – Weisbach and Hazen –Williams formulas. The preeminent equation of open channel flow (and one that the engineer should commit to memory) is Manning's equation. Common design problems on the exam include sizing of pipes, open channels, weirs, and culverts. Hydraulic engineering is needed for the design of stormwater systems, water supply distribution networks, flood control channels, and roadway culverts and bridges. Key terms and units in the discipline of hydraulics include flow (cfs, mgd, or gpm), velocity (cfs), and pressure (psi).

2.1 Energy/Continuity Equation

Two of the most essential and fundamental equations in hydraulics and water resources engineering are the energy and continuity equations. These equations are based on the principles of conservation of energy and mass. That is, what flows into a river or pipe section must flow out. Before we delve into the conservation of energy and mass equations, let us look at the basic formula of hydrostatic pressure:

Hydrostatic Pressure

The hydrostatic (water at rest) equation is defined as:

$p = \gamma h + p_{atm}$

Where:

p = absolute pressure of the fluid (water) at rest (psi)

γ = specific weight of water (62.4 lb/ft^3 at 50 deg F)

h = height of the water column (ft)

p_{atm} = atmospheric pressure (14.7 psi at sea level)

In most open channel flow exercises where the water level is exposed to the atmosphere, the atmospheric pressure term is assumed = 0.

Example 2.1

Calculate the water pressure at the base of a water supply tank (Figure 2.1) where the water level is at 100 ft above mean sea level and the base of the tank is at 50 ft msl.

$p \quad = \gamma h + p_{atm}$

Where:

$p \quad$ = absolute pressure of the fluid (water) at rest (psi)

$\gamma \quad$ = specific weight of water (62.4 lb/ft^3 at 50 deg F)

$h \quad$ = 100 ft – 50 ft = 50 ft

$p_{atm} \quad$ = atmospheric pressure (14.7 psi at sea level)

$p \quad$ = 62.4 lb/ft^3 (50 ft) + 14.7 psi

$p \quad$ = (3120 lb/ft^2)(1 ft/12 in)^2 + 14.7 psi

$p \quad$ = 21.7 psi + 14. 7 psi = 36.4 psi

Since the surface of the tank is open to the atmosphere, the atmospheric pressure term is often assumed to be 0. Therefore p = 21.7 psi at the bottom of the water tank.

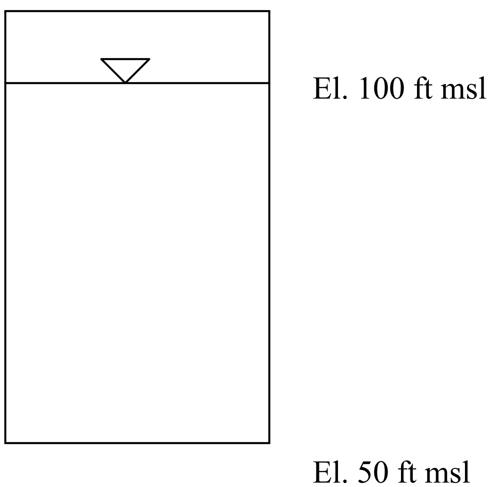


Figure 2.1. Water tank

Energy Equation

The conservation of energy (Bernoulli's) equation for closed conduit (pipe) flow at two points along a channel or pipeline is given as:

$p_1/\gamma + v_1^2/2g + z_1 = p_2/\gamma + v_2^2/2g + z_2$

Where:

p = water pressure (psi)

γ = specific weight of water = 62.4 pcf (U.S.) or 1000 kg/m^3 (S. I.) at 68 deg F or 20 deg C

v = velocity (fps)

g = acceleration due to gravity = 32.2 ft/s^2 (U.S.) or 9.8 m/s^2 (S.I.)

z = elevation of the water surface above a datum (ft), usually mean sea level datum.

In an open channel flow regime, flow is open to the atmosphere and the pressure (p) = 0. Therefore, for open channel flow, Bernoulli's energy equation becomes:

$v_1^2/2g + z_1 = v_2{}^2/2g + z_2$

Figure 2.2. Open channel flow water surface profile.

Example 2.2

Referring to Figure 2.2, calculate the velocity at cross section 2 in an open channel where the velocity at cross - section 1 is 5 fps and the water surface elevation above mean sea level at cross – section 1 is 100 ft and the water surface elevation at cross - section 2 is 99 ft.

$p_1/\gamma + v_1^2/2g + z_1 = p_2/\gamma + v_2{}^2/2g + z_2$

Open channel flow is at atmospheric pressure so p = 0.

$0 + (5 \text{ fps})^2/2g + 100 \text{ ft} = 0 + v_2{}^2/2g + 99 \text{ ft}$

$(25)/2(32.2) + 100 \text{ ft} = 0 + v_2{}^2/2(32.2) + 99 \text{ ft}$

The velocity at cross section 2 is then:

$v_2 = 9.4$ cfs

Using a derivation of Bernoulli's equation, one can compute the velocity exiting an orifice from the bottom of a tank (Figure 2.3)

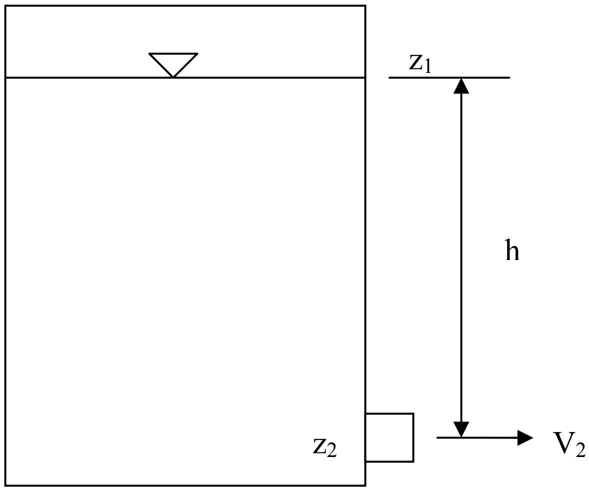


Figure 2.3. Velocity of flow exiting a tank

Bernoulli's equation is given as:

$p_1/\gamma + v_1^2/2g + z_1 = p_2/\gamma + v_2^2/2g + z_2$

For the tank which is open to the atmosphere, the pressure (p) at point 1 (the water surface) and point 2 (flow exiting the orifice) are assumed to be 0. Therefore, the equation becomes:

$v_1^2/2g + z_1 = v_2^2/2g + z_2$

The velocity (v_1) at the water surface in the tank (point 1) is 0. The equation becomes:

$z_1 = v_2^2/2g + z_2$

The distance (h) between point 1 (z_1) and point 2 (z_2) = $z_1 - z_2$. Therefore,

$h = v_2^2/2g$

And the velocity at the outlet is:

$v_2 = (2gh)^{1/2}$

Example 2.3

Calculate the velocity at the outlet of the tank in Figure 2.3 where the water level is situated 10 ft above the centerline of the orifice outlet. The diameter of the orifice is 6 in. Ignore friction losses at the outlet of the tank.

$v_2 = (2gh)^{1/2}$

$v_2 = (2(32.2\ ft/s^2)(10\ ft))^{1/2}$

$v_2 = 25.4$ fps

Continuity Equation

The continuity or conservation of mass equation for two or more points along a channel or pipeline is expressed as:

$Q_1 = Q_2 = Q_3 = Q_n$

$A_1v_1 = A_2v_2 = A_3v_3 = A_nv_n$

Where:

Q = flow rate (cfs)

A = channel cross section area (sf)

v = velocity of flow (fps)

Example 2.4

Streamflow through a creek at section 1 has a velocity of 4 fps flowing through a cross section area of 20 sf. The velocity is measured at 2 fps at cross section 2 just downstream. What is the flow through the stream? What is the area at cross section 2?

By the continuity equation,

$Q = A_1v_1 = A_2v_2$

Q = 20 sf (4 fps) = 80 cfs

If the flow is 80 cfs,

80 cfs = A_2(2 fps)

Then the area at cross section 2 is 80 cfs/2 fps

$A_2 = 40$ sf

Example 2.5

A large pipe 1 splits into two smaller pipe branches 2 and 3 as depicted in Figure 2.4. The flow through pipe 1 is 5 cfs and the pipe diameter is 24 in. The flow through pipe 2 is 2 cfs and the diameter is 12 in. The diameter of pipe 3 is 18 in. What is the velocity in pipe 1? What is the flow through pipe 3?

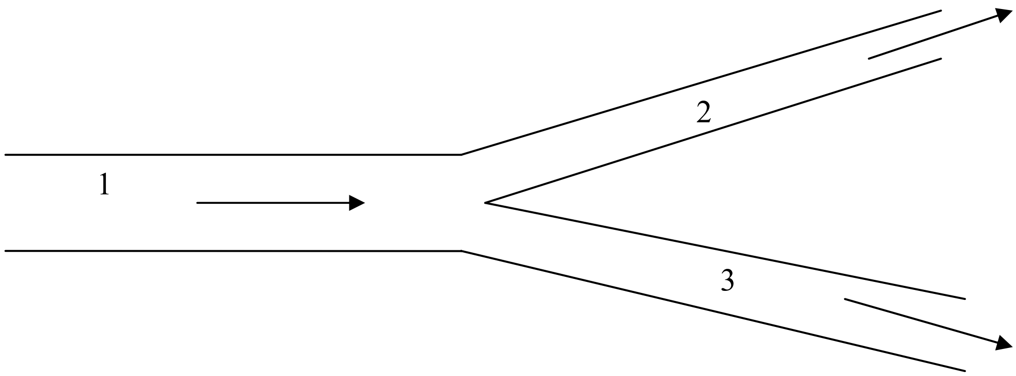


Figure 2.4. Branched pipe flow.

The flow through pipe 1 is

$Q_1 = A_1 v_1$

And if the area of a circular pipe is
$A = \pi\, r^2$

$5 \text{ cfs} = 3.1416((24 \text{ in}/2)(1 \text{ ft} /12 \text{ in}))^2 \,(v_1)$

Then the velocity in pipe 1 is

$v_1 = 5 \text{ cfs}/ 3.1416 \text{ sf} = 1.6 \text{ fps}$

By the continuity equation,

$Q_1 = Q_2 + Q_3$

$5 \text{ cfs} = 2 \text{ cfs} + Q_3$

Then the flow through pipe 3 is

$Q_3 = 5 \text{ cfs} – 2 \text{ cfs} = 3 \text{ cfs}$

Example 2.6

A 12 in diameter concrete pipe at section 1 expands to a 24 in pipe at section 2. The velocity through pipe 1 is 10 fps. What is the velocity through pipe 2?

Using the continuity equation:

$Q = A_1v_1 = A_2v_2$

Where:

$A_1 \quad = \pi\ r^2 = \pi\ ((12\ \text{in})(1\ \text{ft}/12\ \text{in})/2)^2 = \pi(0.25) = 0.78\ \text{sf}$

$Q \quad = A_1v_1 = 0.78\ \text{sf}(10\ \text{fps}) = 7.8\ \text{cfs}$

The velocity through pipe 2 is calculated by:

$Q \quad = A_2v_2$

$7.8\ \text{cfs} = \pi\ r^2 = \pi\ ((24\ \text{in})(1\ \text{ft}/12\ \text{in})/2)^2\ (v_2) = \pi(1.0)\ (v_2)$

$v_2 \quad = 7.8\ \text{cfs}/\ \pi \quad = 2.5\ \text{cfs}$

2.2 Flow Equations

There are 4 general types of flow regimes in hydraulic engineering:

1. Steady flow – The depth of flow does not change over time. The peak flow from a hydrograph (Figure 2.5) is an example of steady flow used to design storm sewers, culverts, and open channels. Simple runoff equations such as the Rational method are used to calculate peak flows in a steady flow regime.

2. Unsteady Flow – The depth of flow changes with time. An example of unsteady flow is a flood hydrograph where with increasing rainfall volume, the flow rate increases to a peak and then decreases after the storm.

3. Uniform flow – The depth and velocity of flow along the channel is the same for an engineered trapezoidal water supply canal (Figure 2.6) or a pipeline. The shape and slope of the channel is uniform along the entire length. Most engineered open channels such as canals and pipelines are examples of uniform flow

4. Nonuniform, varied flow – The depth and velocity changes along the channel with varying cross sections and channel slopes. Nonuniform flow occurs along natural rivers and streams (Figure 2.7) where the channel sections have different shapes and areas and longitudinal slopes range between mild and steep depending on the stream reach.

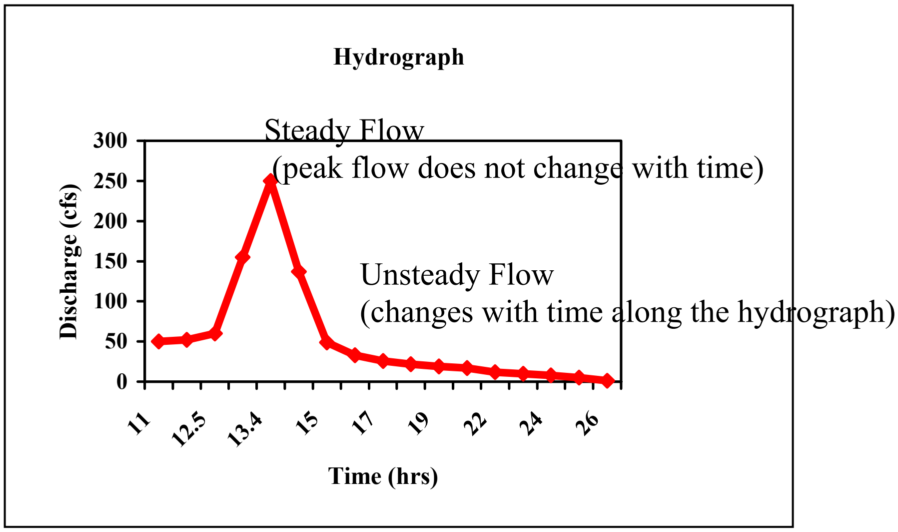


Figure 2.5. Flood hydrograph illustrating steady flow and unsteady flow

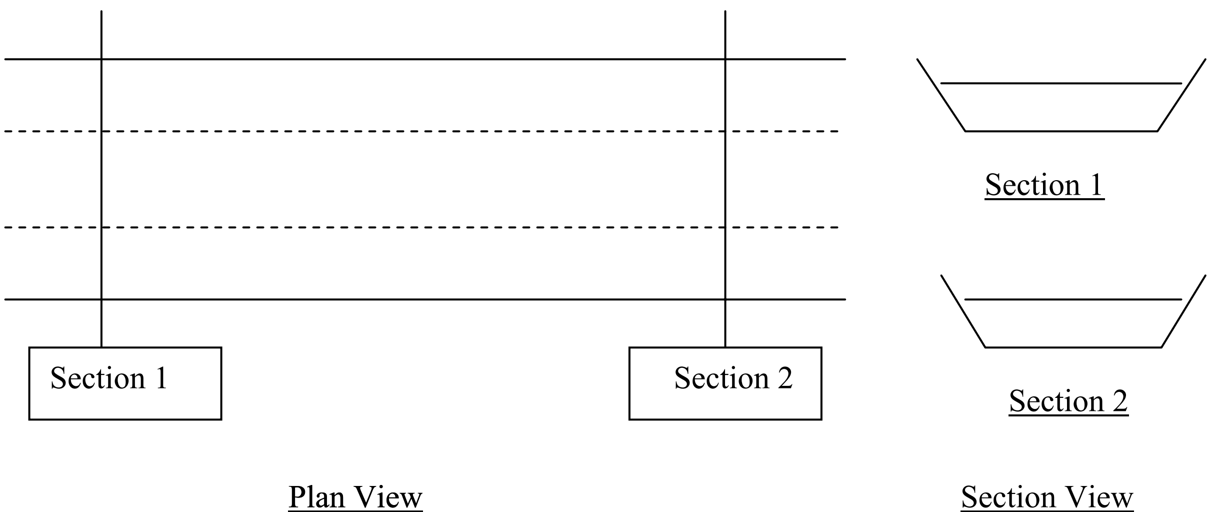


Figure 2.6. Uniform flow through an engineered canal

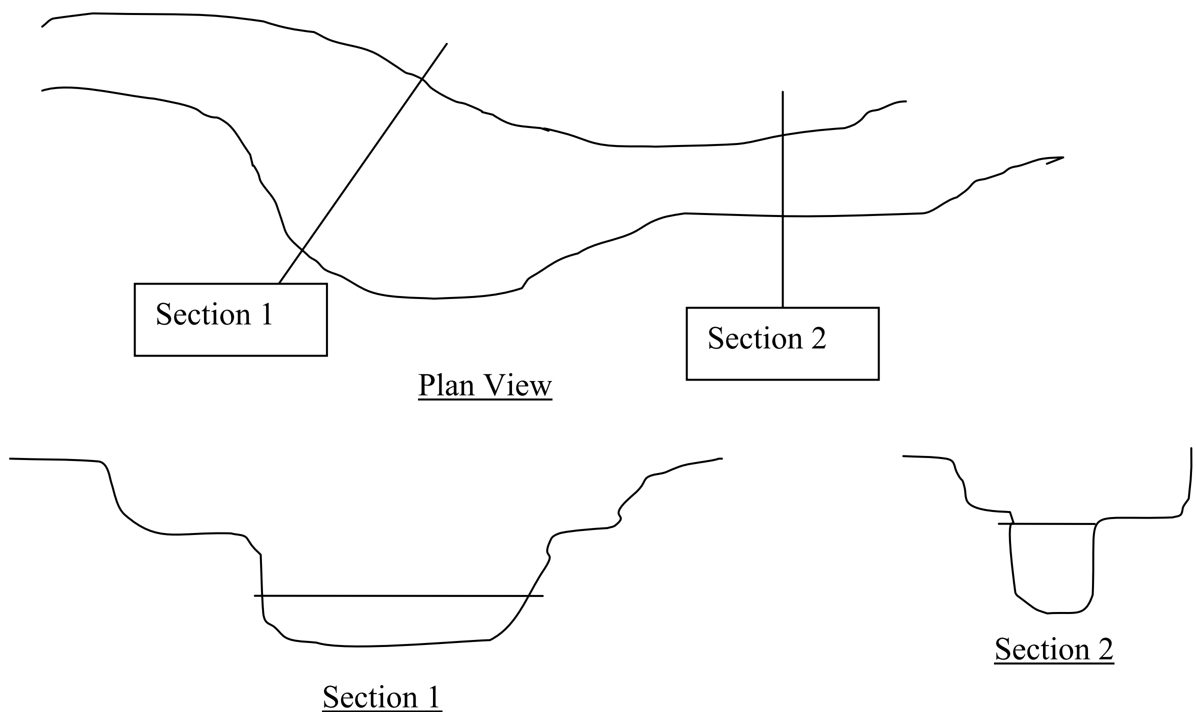


Figure 2.7. Nonuniform, varied flow through a natural river system

2.3 Pressure Conduits

Pressure conduits or pipes are used to transmit fluids under pressure such as in drinking water distribution systems or reservoir pipelines. Flow through pressure conduits or pipes is calculated by estimating the head losses from (1) friction along the wall of the pipe or the conduit and (2) turbulence created by gates, valves and bends in the system.

Darcy – Weisbach Equation

The Darcy – Weisbach equation can be used to estimate the head loss due to pipe friction as:

$$h_L = \frac{f\,L\,v^2}{D\,2g}$$

Where:

h_L = head loss due to pipe friction (ft)

f = friction factor for pipe determined from the Moody diagram (Figure 2.8)

L = length of pipe (ft)

v = velocity through the pipe (fps) = Q/A

D =   diameter of the pipe (ft)

g =   acceleration due to gravity = 32.2 fps

Values of friction factor (f) are determined from the Moody diagram using the Reynolds number (R) and relative roughness (e/D) of the pipe.

R =   Dv/υ

Where:

R =   Reynolds number

v =   velocity (fps)

υ =   kinematic viscosity of water = 1.41 x 10-^{5} ft^{2}/sec at 50 deg F

and

e =   roughness of interior pipe surface = 0.001 (ft)

D =   diameter of pipe (ft)

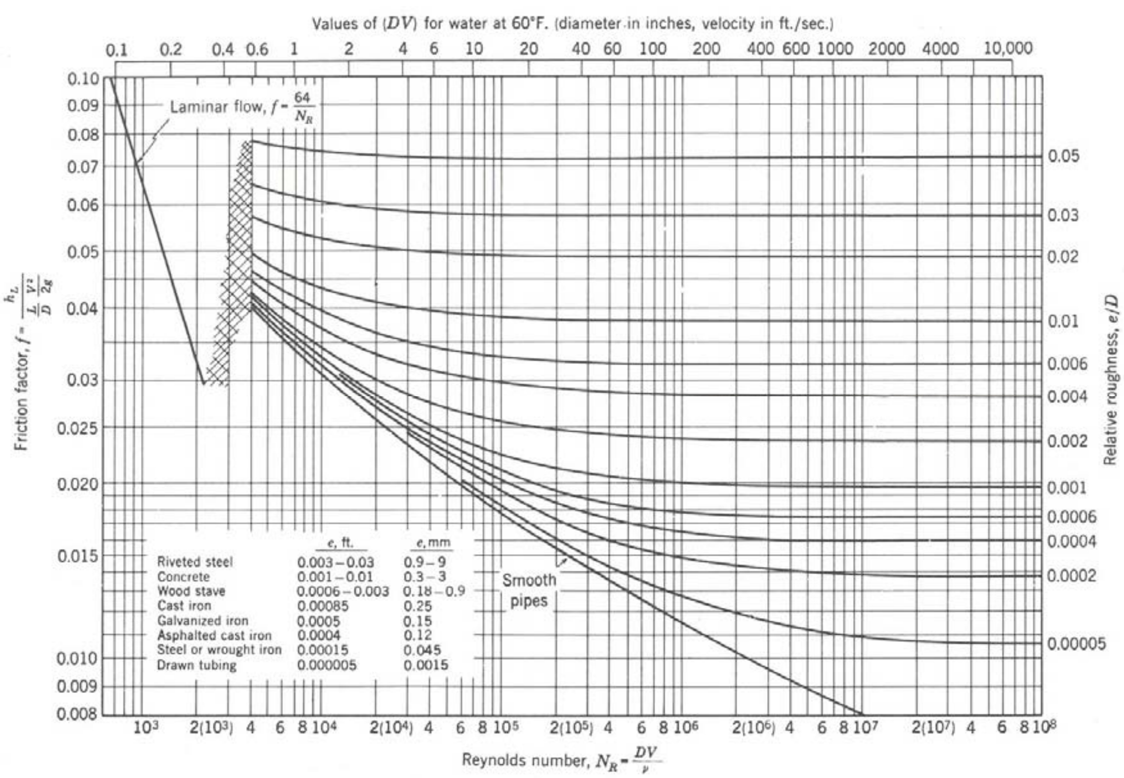


Figure 2.8. Moody diagram (source: Clark, J. W., W. Viessman, Jr., and M. J. Hammer, 1977)

Example 2.7

Estimate the headloss through a 5 ft diameter pipe, 2000 ft long conveying 50 cfs of water using the Darcy –Weisbach equation.

$$h_L = \frac{f\,L\,v^2}{D\,2g}$$

Compute the Reynolds number

$$R = Dv/\upsilon$$

$$\begin{aligned} v &= Q/A \\ &= 50 \text{ cfs}/\pi(5 \text{ ft}/2)^2 \\ &= 2.5 \text{ fps} \end{aligned}$$

$$R = 5 \text{ ft}(2.5 \text{ fps})/\ 1.41 \times 10^{-5}$$
$$R = 8.9 \times 10^6$$

Compute the relative roughness of the pipe

$$\begin{aligned} &= e/D \\ &= 0.001 \text{ ft}/5 \text{ ft} \\ &= 0.0002 \end{aligned}$$

Compute the friction factor (f) for pipe determined from the Moody diagram (Figure 2.8)

From the Moody diagram, for $R = 8.9 \times 10^6$ and $e/D = 0.0002$, then $f = 0.014$

Compute the head loss

$$h_L = \frac{f\,L\,v^2}{D\,2g}$$

$$= \frac{0.014(2000 \text{ ft})(2.5 \text{ fps})^2}{(5 \text{ ft})\ 2\ (32.2 \text{ ft/sec}^2)}$$

$$= 0.45 \text{ ft}$$

Hazen – Williams Formula

Flow under pressure through pipes or closed conduits such as drinking water distribution systems is expressed by the Hazen – Williams formula. One of the principal parameters in this formula is the Hazen Williams roughness coefficient (C) which is an indication of the friction caused by the interior of a pipe. Smoother pipes such as new ductile iron pipes have higher “C” values than older brick - lined pipes with more interior roughness. Thus, newer and smoother pipes can convey more flow than older and rougher pipes. With the Hazen-Williams formula, it is important to convert units as the flow rate (Q) is needed in gpm but the flow is often given in mgd or cfs.

$$h_L = \frac{L\,Q^{1.85}}{17{,}076\,(C)^{1.85}(D)^{4.87}}$$

Where:

h_L = pressure head loss in the pipe or conduit due to friction (ft)
L = pipe length (ft)
C = Hazen - Williams roughness coefficient as given by Table 2.1.
D = pipe diameter (ft)
Q = flow rate (gpm)

Table 2.1 Hazen - Williams Roughness Coefficients

Pipe Type	C
Brick	100
Cast iron, 20 yr old	100
Riveted steel	110
Ductile iron pipe, 20 yr old	110
Clay pipe	110
Cast iron, 5 yr old	120
Welded steel	120
Wood plank pipe	120
Concrete	130
Cast iron, new	130
Ductile iron pipe, new	130
Plastic pipe	140

Example 2.8

Calculate the pressure head loss through a 100 ft long, 12 in diameter, new ductile iron pipe where the flow rate is 1.44 mgd.

Given:

L = 100 ft
D = 12 in = 1 ft
C = 130 (from Table 1.1, new ductile iron pipe)
Q = 1.44 mgd (1,000,000 gal/MG) (1 day/24 hr) (1 hr/60 min) = 1000 gpm

So the pressure head loss is calculated by the Hazen - Williams formula as:

$$h_L = \frac{100\text{ ft }(1000\text{ gpm})^{1.85}}{17{,}076\,(130)^{1.85}(1\text{ ft})^{4.87}}$$

$h_L = 0.25$ ft

Parallel Piping

Frequently the designer is asked to compute the flow through pipes in parallel such as for a water distribution network or reservoir pipe line problem. For two or more pipes in parallel, the energy equation combined with the Hazen – Williams formula indicates:

$h_L = (h_L)_1 = (h_L)_2 = (h_L)_3$

Where:

h_L = head loss according to the Hazen – Williams formula.

Given the three – pipe network in Figure 2.9, the continuity equation indicates:

$Q = Q_1 + Q_2 + Q_3$

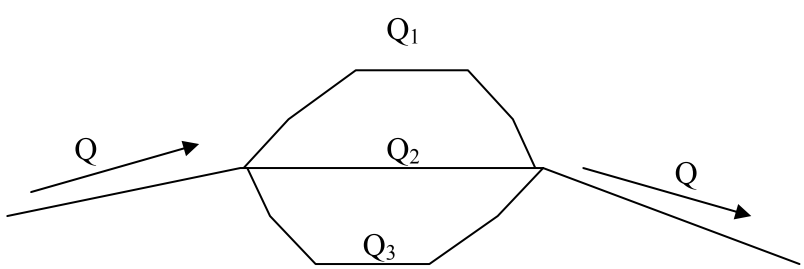


Figure 2.9 Piping system in parallel

The flows through each pipe can be calculated according to the following iterative process.

1. Compute the headloss in each pipe using the Hazen – Williams formula.
2. Assume a headloss in each pipe.
3. Solve for Q_1, Q_2, and Q_3 using the continuity equation, $Q = vA$.
4. If the continuity equation is not solved, assume a new head loss through each pipe and repeat the first three steps.

Example 2.9

Determine the flow through each pipe in a parallel system as shown in Figure 2.9 assuming the flow is 10,000 gpm and the following data:

Pipe	L (ft)	D (in)	C
1	120	12	100
2	100	12	100
3	80	12	100

Solve for head losses through each pipe using the Hazen – Williams formula:

Pipe 1: $(h_L)_1 = \dfrac{120\text{ ft }(Q_1)^{1.85}}{17{,}076\ (100)^{1.85}(1\text{ ft})^{4.87}} = 0.0000014(Q_1)^{1.85}$

Pipe 2: $(h_L)_2 = \dfrac{100\text{ ft }(Q_2)^{1.85}}{17{,}076\ (100)^{1.85}(1\text{ ft})^{4.87}} = 0.0000011(Q_2)^{1.85}$

Pipe 3: $(h_L)_3 = \dfrac{80\text{ ft }(Q_3)^{1.85}}{17{,}076\ (100)^{1.85}(1\text{ ft})^{4.87}} = 0.0000009(Q_3)^{1.85}$

After several iterative steps, assume $(h_L) = 3.5$ ft, then:

Pipe 1: $(h_L)_1 = 3.5\text{ ft} = 0.0000014(Q_1)^{1.85}$

$(Q_1)^{1.85} = 2{,}500{,}000$

$Q_1 = 2{,}900$ gpm

Pipe 2: $(h_L)_2 = 3.5\text{ ft} = 0.0000011(Q_2)^{1.85}$

$(Q_2)^{1.85} = 3{,}181{,}818$

$Q_2 = 3{,}400$ gpm

Pipe 3: $(h_L)_3 = 3.5\text{ ft} = 0.0000009(Q_3{}^{1.85}$

$(Q_3)^{1.85} = 3{,}888{,}888$

$Q_3 = 3{,}700$ gpm

By the continuity equation:

$Q \quad = Q_1 + Q_2 + Q_3$

10,000 gpm = 2,900 gpm + 3,400 gpm + 3,700 gpm (OK)

Branched pipe networks

Figure 2.10 depicts a branched pipe network where three pipes are each connected to a reservoir. This is one of the more common problems on the water resources portion of the exam. For each of the three pipes, Bernouli's conservation of energy equations are derived as:

Pipe 1: $(h_{L})_1 = z_A - z_B$

Pipe 2: $(h_{L})_2 = z_B - z_C$

Pipe 3: $(h_{L})_3 = z_B - z_D$

Where:

z = reservoir water surface elevation (ft)

(h_L) = head loss through each pipe according to the Hazen – Williams formula

$$= \frac{L\,Q^{1.85}}{17{,}076\,(C)^{1.85}(D)^{4.87}}$$

By the continuity equation:

$\sum Q \quad = Q_1 - Q_2 - Q_3 = 0$

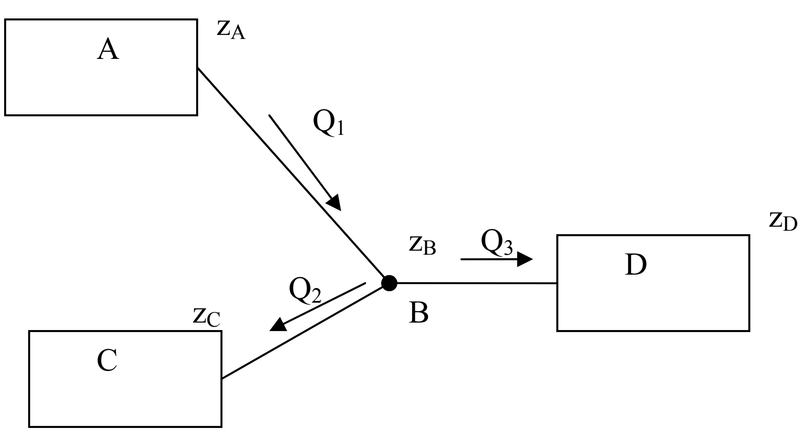


Figure 2.10. Branched pipe network to reservoirs.

The reservoir elevations (z_A, z_C, z_D) are usually given and the unknowns are the elevation at the intersection of the three pipes (z_B) and the flow quantity and direction through each of the pipes (Q_1, Q_2 and Q_3).

Through an iterative process, the flows through each pipe can be calculated according to the following steps:

1. Assume Q_1 through pipe 1.
2. Assume a hydraulic grade line elevation at the intersection of the pipes (z_B).
3. Solve for Q_1, Q_2 and Q_3 by the continuity equation. If the $\sum Q$ is not equal to 0, then assume a new z_B and recalculate.
4. Repeat steps 1, 2, and 3 until the continuity equation $\sum Q = Q_1 - Q_2 - Q_3 = 0$.

Example 2.10

For the reservoir system illustrated in Figure 2.10 and the parameters listed in the table below, estimate the flow through each of the three branched pipes. The elevations of each of the reservoirs are 110 ft, 50 ft, and 65 ft for reservoirs A, C, and D, respectively.

Pipe	L (ft)	D (in)	C
1	100	12	100
2	100	12	100
3	100	12	100

Solve for the head losses in each pipe.

Pipe 1: $(h_L)_1 = z_A - z_B$ $\qquad \dfrac{(100\text{ ft})\,(Q_1)^{1.85}}{17{,}076\,(100)^{1.85}(1\text{ ft})^{4.87}} = 110\text{ ft} - z_B$

$0.0000011(Q_1)^{1.85} = 110 - z_B$

Pipe 2: $(h_L)_2 = z_B - z_C$ $\qquad \dfrac{(100\text{ ft})\,(Q_2)^{1.85}}{17{,}076\,(100)^{1.85}(1\text{ ft})^{4.87}} = z_B - 50\text{ ft}$

$0.0000011(Q_2)^{1.85} = z_B - 50\text{ ft}$

Pipe 3: $(h_L)_3 = z_B - z_D$ $\qquad \dfrac{(100\text{ ft})\,(Q_3)^{1.85}}{17{,}076\,(100)^{1.85}(1\text{ ft})^{4.87}} = z_B - 65\text{ ft}$

$0.0000011(Q_3)^{1.85} = z_B - 65\text{ ft}$

After several iterations, assume $z_B = 70$ ft. Then solve for Q_1, Q_2, and Q_3.

Pipe 1: $0.0000011(Q_1)^{1.85} = 110\text{ ft} - z_B = 110\text{ ft} - 70\text{ ft} = 40\text{ ft}$

$(Q_1)^{1.85}$ = 26,363,636
Q_1 = 12,300 gpm

Pipe 2: $0.0000011(Q_2)^{1.85}$ = z_B – 50 ft = 70 ft – 50 ft = 20 ft
$(Q_2)^{1.85}$ = 18,181,818
Q_2 = 8,400 gpm

Pipe 3: $0.0000011(Q_3)^{1.85}$ = z_B – 65 ft = 70 ft – 65 ft = 5 ft
$(Q_3)^{1.85}$ = 4,545,454
Q_3 = 4,000 gpm

By the continuity equation:

$\sum Q = Q_1 - Q_2 - Q_3 = 0$

12,300 gpm – 8,400 gpm – 4000 gpm = - 100 gpm ≈ 0 (OK)

Therefore,

Q_1 = 12,300 gpm
Q_2 = 8,400 gpm
Q_3 = 4,000 gpm

2.4 Friction/Minor Losses

Friction caused by the flow of water through pipes is measured by the Hazen Williams roughness coefficients (C) summarized in Table 1.1. Higher "C" values are selected for newer and smoother pipes. Lower "C" values are selected for older and rougher pipes.

Minor losses from gates, valves and bends in a closed conduit (pipe) flow system are proportional to the velocity head given by:

$$h_L = \frac{K\,v^2}{2\,g}$$

where:

h_L = minor head losses

K = friction loss coefficient given in Table 1.2.

v = velocity (fps)

g = acceleration due to gravity = 32.2 ft/sec^2

Table 2.2. Friction loss coefficients

Parameter	Friction Loss Coefficient (K)
Globe valve	6
Check valve	3
Gate valve	6
Pipe tee	2
Pipe elbow	1
45 deg elbow	0.5
Square entrance	0.5
Contraction	0.4
Exit	1.0

2.5 Pump Application and Analysis

Pumping stations are usually of two types: axial or centrifugal pumps. Axial flow pumps generate lower pressures at high flow rates by expelling the fluid inline through the pipe as through the propeller of an airplane. Centrifugal flow pumps generate pressure by accelerating the flow by an impeller, which gives velocity to the fluid through centrifugal force.

Referring to Figure 2.11, total dynamic pumping head is described as:

$TDH = h_L + h_F + h_V$

Where:

TDH = Total dynamic head (ft)

h_L = Total static head which is the vertical distance between fluid water levels or from the pump centerline to the fluid water level (ft)

h_F = Friction head losses from Hazen - Williams formula and the minor head losses formula (ft)

h_V = Velocity head ($v^2/2g$)

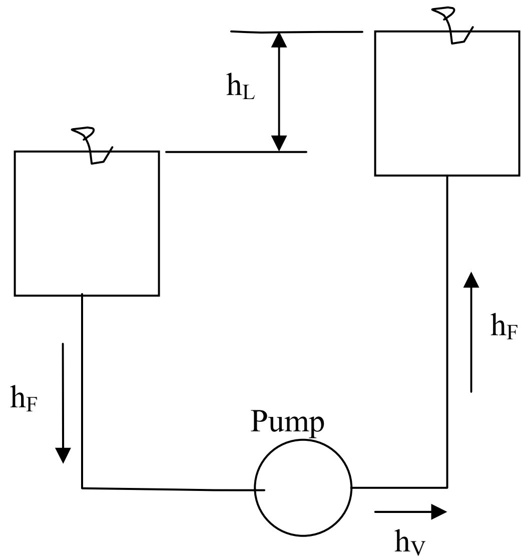


Figure 2.11. Total dynamic head schematic.

Example 2.11

Design a pump station for a total capacity of 1000 gpm at a head of 50 ft.

Figure 2.12 plots the curve for total dynamic head versus discharge. Three pumps are shown on the curve. Pump A will provide 1000 gpm at a head of 50 ft. Therefore select Pump A.

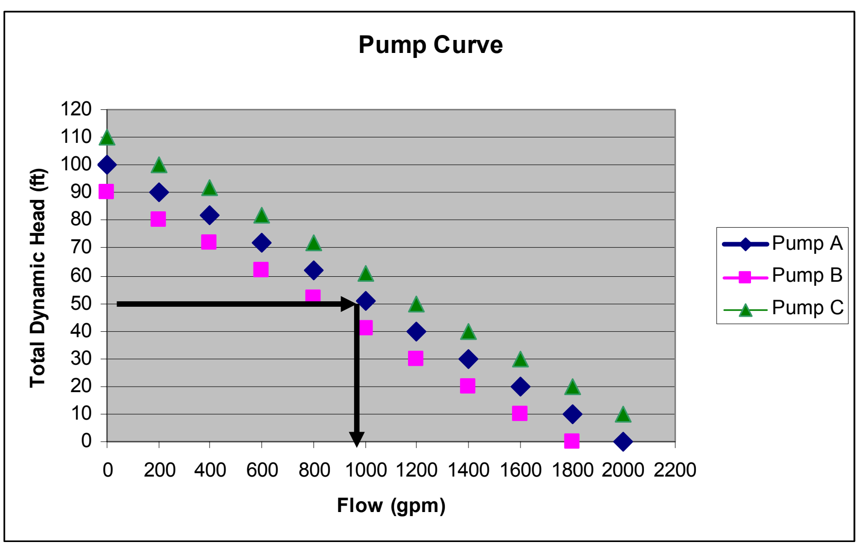


Figure 2.12. Pump curves

2.6 Cavitation

Cavitation occurs when internal water pressures drop due to decreasing vapor pressure thus causing collapse of the pump or pipe system. The net positive suction head equation is used to design pumps with proper criteria to avoid cavitation:

$$NPSH \quad \leq \frac{p_{atm} - p_{vap}}{\gamma} - \Delta z - h_L$$

Where:

NPSH = net positive suction head for a pump (ft)

p_{atm} = atmospheric pressure = 14.7 psi (U.S.) or 101 kPa (S.I.)

p_{vap} = vapor pressure of water (psi) = 0.5 psi at 80 deg F.

Δz = $z_2 - z_1$ which is the difference in elevation (ft) between locations 1 and 2.

h_L = head loss (ft) by the Hazen – Williams formula on the suction pipe between locations 1 and 2.

γ = specific density of water = 62.4 pcf (U.S.) or 1000 kg/m^3

Example 2.12

The allowable net positive suction head for a pump is 25 ft for water pumped through a 100 ft long pipe where the diameter is 12 in, the flow rate is 1000 gpm, and C = 130 for new ductile iron pipe. What is the height (z) above the pipe inlet water surface that the pump should be located to avoid cavitation? Assume p_{vap} = 0.5 psi for temperature = 80 deg F.

The allowable net positive suction head equation is:

$$NPSH \quad \leq \frac{p_{atm} - p_{vap}}{\gamma} - \Delta z - h_L$$

$$25 \text{ ft} = \frac{(14.7 \text{ psi} - 0.5 \text{ psi})(144 \text{ sq in/sq ft})}{62.4 \text{ pcf}} - \Delta z - \frac{(100 \text{ ft})(1000 \text{ gpm})^{1.85}}{17{,}076(130)^{1.85}(1 \text{ ft})^{4.87}}$$

$$25 \text{ ft} = 33 \text{ ft} - \Delta z - 0.25 \text{ ft}$$

$$\Delta z = 7.75 \text{ ft.}$$

Therefore, the pump should be placed no higher than 7.75 ft above the inlet end of the pipe.

2.7 Pipe Network Analysis

Pipe network analysis for closed conduits under pressure is accomplished by the Hardy - Cross method, which is designed to balance the head losses in each pipe. Pipe networks are designed assuming that the sum of the head losses in any pipe loop must equal zero. This is the fundamental method used to design water supply distribution networks.

According to the continuity or conservation of mass equation, the flow entering a particular node must equal the flow leaving it. For instance if Pipe A with flow of 1000 cfs enters node 1, then the sum of flows leaving node 1 through pipes B and C must equal 1000 cfs.

The Hardy - Cross method is one of the more complex calculations in water resources engineering and is often remembered as an iterative five step process:

1. Number each of the various pipe loops by identifying the junctions (nodes) by alphabet or number.

2. Assume a flow direction and assume a flow through each pipe. Clockwise flow through a pipe loop is assumed to be positive (+) and counter clockwise flow is assumed to be negative (-).

3. Calculate the head loss through each pipe in a loop using the Hazen - Williams formula.

4. Sum the head losses in all pipes in each loop being careful to utilize the correct positive or negative signs depending on the flow direction.

5. If the sum of the head losses for is less than one, then the assumptions for flow and direction through the pipes is correct and the calculation is complete. If the sum of the head losses is greater than 1, then employ a flow correction factor (q) as follows. Add the flow correction factor to the assumed flow for each pipe and repeat the calculation starting with Step 2. Continue the calculation cycle until the sum of head losses in each loop is less than 1.

$$q = \frac{-\Sigma h_L}{\Sigma 1.85(h)/Q \text{ (for each loop)}}$$

Where:

Q = flow correction factor (gpm)

Σh_L = sum of the headlosses in each pipe loop (ft).

H = head loss in each pipe (ft).

Q = assumed flow rate through each pipe (gpm).

Example 2.13

Referring to the water supply network depicted in Figure 2.13, calculate the flow through each pipe in loop A, B, C using the Hardy - Cross method. Assume Hazen – Williams roughness coefficient (C) of 130 for new, ductile iron pipe.

According to the five step process:

1. Number each of the various pipe loops with nodes A, B, C, and D.

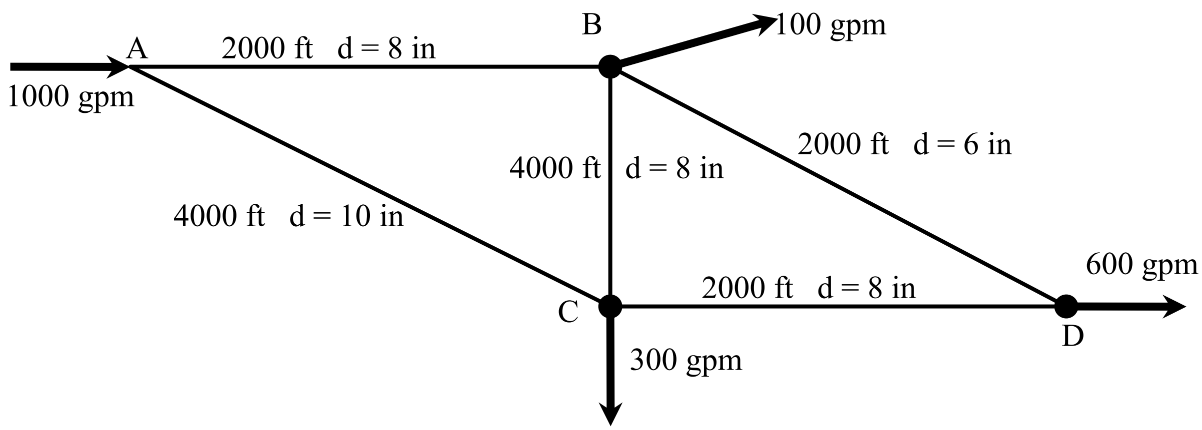
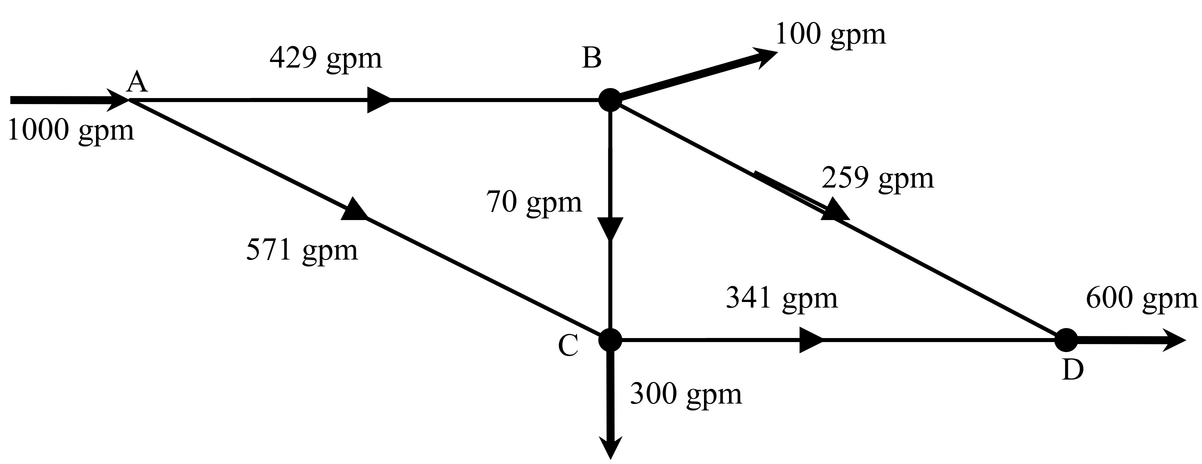


Figure 2.13. Hardy – Cross method pipe network

1. Assume a flow direction, clockwise (+) and counterclockwise (-) and assume a flow through each pipe.



2. Calculate the head losses through each pipe using the Hazen – Williams formula.

$$h_L = \frac{L\,(Q)^{1.85}}{17{,}076\,(C)^{1.85}(D)^{4.87}}$$

Pipe AB:

$$h_{AB} = \frac{2000\text{ ft }(429\text{ gpm})^{1.85}}{17{,}076\,(130)^{1.85}(0.67\text{ ft})^{4.87}} = \frac{2000\text{ ft }(74{,}138)}{17{,}076\,(8{,}143)(0.139)}$$

$$h_{AB} = (+)\ 7.63\text{ ft}$$

Pipe BC:

$$h_{BC} = \frac{4000\text{ ft }(70\text{ gpm})^{1.85}}{17{,}076\,(130)^{1.85}(0.67\text{ ft})^{4.87}}$$

$$h_{BC} = (+)\ 0.54\text{ ft}$$

Pipe AC:

$$h_{AC} = \frac{4000\text{ ft }(571\text{ gpm})^{1.85}}{17{,}076\,(130)^{1.85}(0.83\text{ ft})^{4.87}}$$

$$h_{AC} = (-)\ 8.80\text{ ft}$$

3. Sum head losses in all pipes in loop A, B, C. If the sum is less than or equal to 1 ft, than the assumed flows through each pipe is correct.

$$\Sigma h_L = h_{AB} + h_{BC} - h_{AB} \leq 1\text{ ft}$$

$$= 7.63\text{ ft} + 0.54\text{ ft} - 8.8\text{ ft}$$

$$\underline{\Sigma h_L} = (-)\ 0.63\text{ ft} \leq 1\text{ ft} \qquad \text{(OK)}$$

4. If the sum of the head losses is greater than 1 ft, than calculate the flow correction factor. For this example one would stop after step 4 since the sum of head losses in all pipes in loop A, B, C is less than or equal to 1 ft. However, step 5 is performed here to demonstrate the concept of the flow correction factor.

$$q = \frac{-\,\Sigma h_L}{\Sigma 1.85(h)/Q\text{ (for each loop)}}$$

$$= \frac{(-)\ 0.63\text{ ft}}{\frac{(1.85)(7.63\text{ ft})}{429\text{ gpm}} + \frac{(1.85)(0.54\text{ ft})}{70\text{ gpm}} + \frac{(1.85)(-8.8\text{ ft})}{571\text{ gpm}}}$$

$$q = 39\text{ gpm}$$

Therefore, add 39 gpm to flow through each pipe and repeat calculations starting in step 2. Repeat each iteration until the sum of head losses in all pipes in each loop is less than or equal to 1 ft.

2.8 Open Channel Flow

Open channel flow open to the atmosphere through streams, rivers, storm sewers, canals and other conveyances where flow is not under pressure. The velocity and flow rate in open channels are computed using Manning's Equation where:

$$v = \frac{1.49}{n} (R)^{2/3} (S)^{1/2} \qquad \text{(U.S.)}$$

$$v = \frac{1.00}{n} (R)^{2/3} (S)^{1/2} \qquad \text{(S.I.)}$$

Where:

v = velocity

n = Manning's roughness value obtained from Table 2.3.

R = hydraulic radius = A / WP

A = cross section area of channel

WP = wetted perimeter or the length of the channel cross section perimeter wetted by the flow.

S = longitudinal slope of channel

According to the continuity equation:

$$Q = v\,A$$

Therefore, Manning's equation can be modified to calculate the flow rate in an open channel as:

$$Q = v\,A$$

$$= \frac{1.49}{n} (R)^{2/3} (S)^{1/2} (A) \qquad \text{(U.S.)}$$

$$= \frac{1.00}{n} (R)^{2/3} (S)^{1/2} (A) \qquad \text{(S.I.)}$$

Table 2.3. Manning's roughness values

Channel Type	Manning's Roughness values (n)
Natural Streams	0.030
Earth Channel	0.022
Farm	0.035
Meadow	0.050
Heavy Brush	0.075
Trees	0.150
Concrete	0.013
Cast Iron	0.013
Steel	0.012
Corrugated Metal	0.022
Clay Tile	0.014
Brick	0.015
Masonry, stones	0.025
Riprap, Rock	0.035

The hydraulic radius (R) is defined as the cross section area (A) divided by the wetted perimeter (WP).

Cross Section Area (A)

The area of rectangular, triangular, or trapezoidal channels can be computed through geometry by summing the area of the triangles and rectangles. It is important to sketch out and mark the triangles and squares on the the cross section to calculate the area and wetted perimeter.

Rectangular Channel

The area of a rectangular channel (Figure 2.14) can be computed as:

A = b (d)

Where:

b = width of the base

d = depth of flow

For instance, the area of a 3 ft deep by 10 ft wide rectangular channel is:

A = 3 ft (10 ft) = 30 sf.

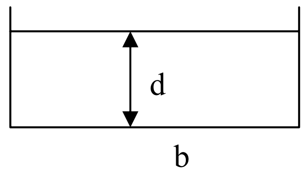


Figure 2.14. Rectangular open channel

Triangular Channel

The area of a triangular channel can be computed as:

A = ½(w)(d)

Where:

w = top width

d = depth of flow

The area of a 3 ft deep by 10 ft top width triangular channel is:

A = 3 ft (10 ft)/2 = 15 sf

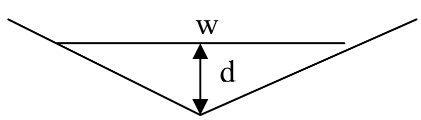


Figure 2.15. Triangular open channel

Trapezoidal Channel

The area of a trapezoidal channel can be computed by:

A = b(d) + z(d)/2 + z(d)/2

Where:

b = width of the base

d = depth of flow

z = ?

The area of a 3 ft deep by 10 ft bottom width trapezoidal channel with 1 ft horizontal to 1 ft vertical sideslopes is:

A = 3 ft (10 ft) + 3 ft(3 ft)/2 + 3 ft(3 ft)/2 = 30 ft + 4.5 ft + 4.5 ft = 39 ft

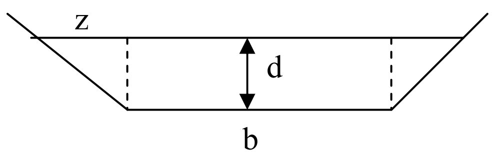


Figure 2.16. Trapezoidal open channel.

Circular Channel

The cross sectional area of a circular pipe flowing full can be computed as:

$$A = \pi r^2$$

The area of a circular pipe flowing half full can be computed as:

$$A = \pi r^2/2$$

For pipes flowing less than or more than half full, the area can be computed using Table 1.??

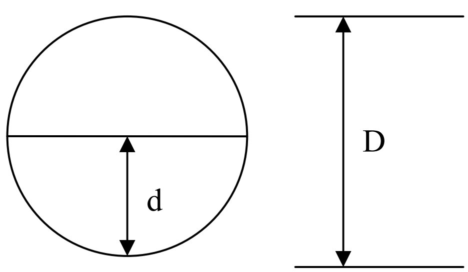


Figure 2.17. Circular open channel

Table 2.4. Area Dimensions of Circular Pipes

d/D	A / D^2	WP/D	R/D
0.05	0.0147	0.4510	0.0326
0.10	0.0409	0.6435	0.0635
0.15	0.0739	0.7954	0.0929
0.20	0.1118	0.9273	0.1206
0.25	0.1535	1.0472	0.1466
0.30	0.1982	1.1593	0.1709

0.35	0.2450	1.2661	0.1935
0.40	0.2934	1.3694	0.2142
0.45	0.3428	1.4706	0.2331
0.50	0.3927	1.5708	0.2500
0.55	0.4426	1.6710	0.2649
0.60	0.4920	1.7722	0.2776
0.65	0.5404	1.8755	0.2881
0.70	0.5872	1.9823	0.2962
0.75	0.6318	2.0944	0.3017
0.80	0.6736	2.2143	0.3042
0.85	0.7115	2.3462	0.3033
0.90	0.7445	2.4981	0.2980
0.95	0.7707	2.6906	0.2864
1.00	0.7854	3.1416	0.2500

The area of a 4 ft diameter pipe flowing 1 ft deep can be computed as:

From Table 2.4:

d/D = 1 ft/4 ft = 0.25.

A/D^2 = 0.1535

A = $0.1535(4 \text{ ft})^2$ = 2.46 sf

Wetted Perimeter

The wetted perimeter of rectangular, triangular, and trapezoidal channels can be computed using geometry.

Rectangular channels

The WP of a rectangular channel is:

WP = 2(d) + (b)

The wetted perimeter of a 3 ft deep by 10 ft wide rectangular channel is:

WP = 2(3 ft) + (10 ft) = 16 ft

Triangular channels

Using Pythagoras's theorem, the WP of a triangular channel is:

$WP = 2(d^2 + (w/2)^2)^{1/2}$

The wetted perimeter of a 3 ft deep by 10 ft top width triangular channel is:

$WP = 2(3^2 + (10/2)^2)^{1/2} = 2(9 + 25)^{1/2} = 2(34)^{1/2} = 2(5.8 \text{ ft}) = 11.7 \text{ ft}$

Trapezoidal channel

The wetted perimeter of a trapezoidal chanell is computed as:

$WP = b + 2(d^2 + (d(h/v)^2)^{1/2}$

The wetted perimeter area of a 3 ft deep by 10 ft bottom width trapezoidal channel with 1 ft horizontal to 1 ft vertical sideslopes is:

$WP = 10 \text{ ft} + 2(3^2 + (3)^2)^{1/2} = 10 \text{ ft} + 2(9 \text{ ft} + 9 \text{ ft})^{½} = 10 \text{ ft} + 2(18)^{1/2} = 10 \text{ ft} + 2(4.2) = 18.4 \text{ ft}$

Circular Pipes

The wetted perimeter of circular pipes flowing full or half full can be calculated using geometry:

$WP = 2\pi r$ (flowing full)

$WP = 2\pi r/2 = \pi r$ (half full)

For pipes flowing more or less than half full, the wetted perimeter can be calculated using Table 1.??.

The wetted perimeter of a 4 ft diameter pipe flowing 1 ft deep can be computed as:

From Table 2.4:

$d/D = 1 \text{ ft}/4 \text{ ft} = 0.25.$

$WP/D = 1.0472$

$WP = 1.0472\ (4 \text{ ft}) = 4.2 \text{ ft}$

Channel slope

The slope of the channel or pipe can be computed as:

S = y1 – y2/x

Where:

y1 = elevation of the channel thalweg or pipe invert at section 1.

,y2 = elevation of the channel thalweg or pipe invert at section 2.

,x = horizontal distance between the two sections.

The thalweg is defined as the lowest elevation in a stream channel or waterway.

The invert is defined as the lowest elevation in a pipe section.

Calculate the slope of a stream channel where the thalweg elevation at section 1+00 is 100 ft above sea level and the thalweg elevation at section 2+00 is 99 ft.

S = (100 ft – 99 ft)/(200 ft – 100 ft) = 1 ft/100 ft = 0.01 ft/ft

Example 2.14

Compute the flow through a rectangular concrete channel with depth = 5 m and width = 10 m. The channel slope decreases 1 m over the next 1 km.

Use Manning's equation (S.I units):

$$v = \frac{1.00}{n} (R)^{2/3} (S)^{1/2}(A)$$

Where:

,n = 0.013 (Table 1.??, concrete)

A = 5 m(10 m) = 50 m^2
WP = 2(5 m) + 10 m = 20 m

R = A/WP = 50 m/20 m = 2.5 m

S = 1.0 m/1 km (1000m/km) = 0.001 m/m

The flow rate is calculated:

$$Q = \frac{1.00}{0.013}(2.5\text{ m})^{2/3}(0.001\text{ m/m})^{1/2}(50\text{ m}^2)$$

$Q \quad = 77(1.9)(0.031)(50\ m^2)$

$Q \quad = 227$ m/s

Example 2.15

Compute the flow through an earth trapezoidal channel (Figure 2.18) with a 10 feet bottom width, side slopes at 1 ft horizontal to 1 ft vertical (1:1), depth of flow of 3 feet, and the difference in the channel bottom elevation is 10 feet over 500 feet in length.

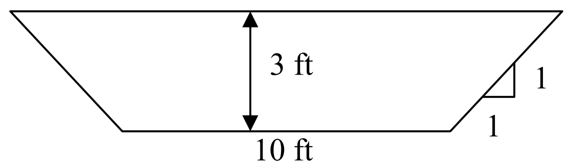


Figure 2.18. Trapezoidal channel

$n \quad = 0.022$ (earth from Table 2.3)

$A \quad = 10\text{ ft }(3\text{ ft}) + 3\text{ ft}(3\text{ ft})/2 + 3\text{ ft}(3\text{ ft})/2 = 39\text{ sf}$

$WP \quad = ((3\text{ ft})^2 + (3\text{ ft})^2)^{1/2} + ((3\text{ ft})^2 + (3\text{ ft})^2)^{1/2} + 10\text{ ft} = 4.2\text{ ft} + 4.2\text{ ft} + 10\text{ ft} = 18.4\text{ ft}$

$R \quad = A/WP = 39\text{ sf}/18.4\text{ ft} = 2.1\text{ ft}$

$S \quad = 10\text{ ft} / 500\text{ ft} = 0.02\text{ ft/ft}$

Using Manning's equation:

$$Q = \frac{1.49}{n} R^{2/3} S^{1/2} (A)$$

$$Q = \frac{1.49}{0.022}(2.1\text{ ft})^{2/3}(0.02\text{ ft/ft})^{1/2}(39\text{ sf})$$

$Q \quad = 67.7(1.6)(0.14)(39\text{ sf})$

$Q \quad = 591$ cfs

Example 2.16

Compute the flow depth necessary to convey the 10 - year flow (Q_{10}) of 175 cfs through a trapezoidal natural channel with a bottom width of 20 ft at elevation 92 ft above sea level (msl) with

5 feet horizontal to 1 ft vertical sideslopes. The longitudinal slope of the stream channel is 0.002 ft/ft

The solution is most efficiently calculated using a matrix format as summarized in Table 2.6. Use Manning's equation and solve for depth.

n = 0.03 (natural channel)

S = 0.002 ft/ft

At Elevation 92 ft:

The depth of flow = 0, therefore A, WP, v, and Q = 0

At Elevation 93 ft:

D = 1 ft

1.49/n = 1.49/0.03 = 49.5

S = 0.002 ft/ft

$S^{1/2}$ = 0.045 ft/ft

A = 5 ft(1 ft) + 20 ft(1 ft) = 25 sf

WP $= 2(5^2 + 1^2)^{1/2}$ + 20 ft = 2(5.1) + 20 ft = 30.2 ft

R = A/WP = 25 sf/30.2 ft = 0.83 ft

$R^{2/3} = (0.83)^{2/3} = 0.88$

The velocity at depth = 1 ft is then calculated by Manning's equation:

$$v = \frac{1.49}{n} R^{2/3} S^{1/2}$$

v = 49.5(0.83)(0.045) = 1.96 fps

The flow rate at depth = 1 ft is then:

Q = vA = 1.96 fps(25 sf) = 49 cfs

Q = 49 cfs < Q10 = 175 cfs

Therefore compute the flow rate at depth = 2 ft or Elevation 94 ft,, At Elevation 94 ft:

D = 2 ft

1.49/n = 1.49/0.03 = 49.5

S = 0.002 ft/ft

$S^{1/2}$ = 0.045 ft/ft

A = 10 ft(2 ft) + 20 ft(1 ft) = 60 sf

WP = 10 ft + 10 ft + 20 ft = 40 ftt

R = A/WP = 60 sf/40 ft = 1.50 ft

$R^{2/3}$ = $(1.50)^{2/3}$ = 1.33

The velocity at depth = 2 ft is then calculated by Manning's equation:

$$v = \frac{1.49}{n} R^{2/3} S^{1/2}$$

v = 49.5(1.33)(0.045) = 2.92 fps

The flow rate at depth = 2 ft is then:

Q = vA = 2.92 fps(60 sf) = 175 cfs

Q = 175 cfs = Q10 = 175 cfs (OK)

The depth of flow is 2 ft calculated for a 10 – year flow rate of 175 cfs. The 10 - year flow elevation is 94 ft (92 ft + 2 ft).

Table 2.6.. Manning's equation solution matrix.

Elev. (ft)	Depth (ft)	1.49/n	$S^{1/2}$	A (sf)	WP (ft)	R = A/WP	$R^{2/3} = (A/WP)^{2/3}$	$v = \frac{1.49}{n} R^{2/3} S^{1/2}$ (fps)	Q = vA (cfs)
92	0	1.49/0.03 = 49.5	$(0.002)^{1/2}$ = 0.045	0	0	0	0	0	0
93	1	49.5	0.045	5(1)+20(1) = 25	5.1+20+5.1 = 30.2	25/30.2 = 0.83	0.88	1.96	25(1.96) = 49
94	2	49.5	0.045	10(2)+20(1) = 60	10+20+10 = 40	60/40 = 1.5	1.31	2.92	60(2.92) = 175

Example 2.17

Calculate the channel slope necessary to convey a flow of 100 cfs through a 20 ft wide by 5 ft deep concrete rectangular channel.

$$Q = \frac{1.49}{n} R^{2/3} S^{1/2} (A)$$

n = 0.013 (concrete, Table 1.??)

A = 20 ft(5 ft) = 100 sf

R = A/ WP = 20 ft(5 ft)/20 ft +2(5 ft) = 100 sf /30 ft = 3.33 ft

Q = 100 cfs

$$100 = \frac{1.49}{0.013} (3.33 \text{ ft})^{2/3} S^{1/2} (100 \text{ sf})$$

$$S^{1/2} = 0.0039$$

S = 0.062 ft/ft or 6.2 % slope

Example 2.18

A 48 – in circular concrete pipe conveys flow at a depth of half full (Figure 2.19). If the longitudinal slope of the pipe is 0.02 ft/ft, compute the velocity and flow rate.

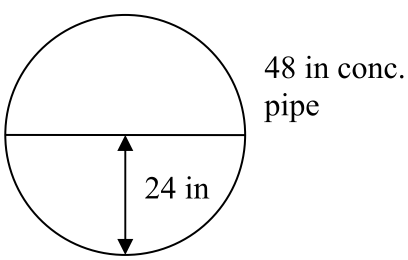


Figure 2.19. Flow through a circular pipe.

Using Manning's equation, the flow is:

$$v = \frac{1.49}{n} (R)^{2/3} (S)^{1/2}$$

$n \quad = 0.013$ (concrete from Table 1.4)

$R \quad = A/WP = 6.3 \text{ sf}/6.3 \text{ ft} = 1 \text{ ft}$

$$A = \frac{\pi(r)^2}{2} = \frac{\pi(24 \text{ in}/12 \text{ in/ft})^2 = 6.3 \text{ sf}}{2}$$

$$WP = \frac{2\,\pi\, r}{2} = \frac{2\,\pi\,(24 \text{ in}/12 \text{ in/ft})}{2} = 6.3 \text{ ft}$$

$S \quad = 0.02$ ft/ft

$$v = \frac{1.49}{0.013} (1 \text{ ft})^{2/3} (0.02 \text{ ft/ft})^{1/2}$$

$v \quad = 16.2$ fps

The flow rate is computed as:

$Q \quad = v\, A$

$\quad = 16.2 \text{ fps}(6.3 \text{ sf})$

$Q \quad = 102$ cfs

Example 2.19

A 48 – in circular concrete pipe conveys flow at a depth of 3 ft. If the longitudinal slope of the pipe is 0.02 ft/ft, compute the velocity and flow rate.

Using Manning's equation, the flow is:

$$v = \frac{1.49}{n} (R)^{2/3} (S)^{1/2}$$

$n \quad = 0.013$ (concrete from Table 1.4)

$A \quad =$ From Table 1.??, for $d/D = 3\text{ft}/4\text{ft} = 0.75$, $A/D^2 = 0.6318$

$A \quad = 0.6318(4 \text{ ft})^2 \qquad = 0.6318(16) = 10.1$ sf

WP = From Table 2.5, for d/D = 0.75, WP/D = 2.0944

WP = 2.0944(4ft) = 8.4 ft

R = A/WP = 10.1 sf/8.4 ft = 1.2 ft

S = 0.02 ft/ft

$$v = \frac{1.49}{0.013} (1.2 \text{ ft})^{2/3} (0.02 \text{ ft/ft})^{1/2}$$

v = 18.2 fps

The flow rate through a 48 in pipe at 3 ft deep is computed as:

Q = v A

= 18.2 fps(10.1 sf)

Q = 184 cfs

2.9 Energy Dissipation

Hydraulic jump often forms downstream from spillways causing high velocity flow thus requiring some form of energy dissipation to prevent erosion of the channel. The hydraulic jump is a transition between rapid, turbulent flow at high velocity and placid, laminar flow at low velocity (Figure 2.20). The hydraulic jump equation for a rectangular channel is expressed as:

$$y_2 = -y_1 +/- \frac{(y_1^2 + 2v_1^2(y_1))}{4g}$$

Where:

y_1 = flow depth upstream from the hydraulic jump (ft)

y_2 = flow depth downstream from the hydraulic jump (ft)

v_1 = velocity upstream from the hydraulic jump (fps)

g = acceleration due to gravity = 32.2 ft/sec^2.

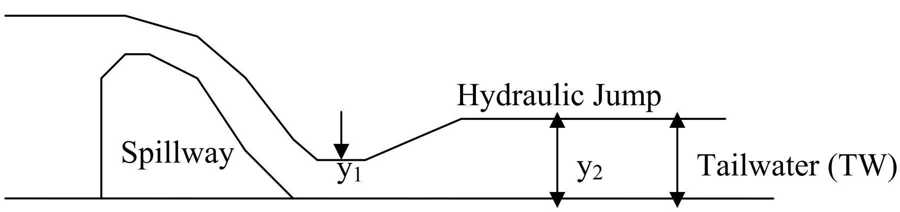


Figure 2.20. Hydraulic jump downstream from spillway

The required energy dissipation below a spillway depends on the depth downstream from the hydraulic jump (y_2) compared to the tailwater depth (TW). If the tailwater depth is greater than y_2, then the tailwater submerges the hydraulic jump and flow continues at high energy along the floor of the channel thus requiring some form of energy dissipation. The most common forms of energy dissipation downstream from spillways consist of stilling basins or scour pools, which allow the tailwater depth to exceed and drown out the hydraulic jump. Table 2.7 lists the various types of energy dissipation or scour protection for the various forms of hydraulic jump.

Table 2.7. Various forms of energy dissipation and scour protection

Class of Flow	Depth (y_2) vs. Tailwater (TW)	Hydraulic Jump	Scour Protection
1	y_2 = TW	Toe of the spillway	Concrete apron with walls.
2	TW > y_2	Jump submerged by tailwater, little energy dissipated	Sloping concrete apron or concrete deflector bucket at base of spillway.
3	y_2 > TW	Toe of spillway	Stilling pool or stilling basin to increase tailwater depth.

2.10 Subcritical and Supercritical Flow

Critical flow is the most efficient flow, from an energy standpoint, in an open channel given by the equation:

$$y_c = \frac{(q^2)^{1/3}}{(g)^{1/3}}$$

Where:

y_c = critical depth (ft)

q = specific discharge in a rectangular channel (cfs/ft) = Q/b

Q = discharge (cfs)

b = flow width (ft)

g = acceleration due to gravity = 32.2 ft/sec^2.

Example 2.20

Calculate the critical depth in a rectangular channel where the discharge (Q) = 50 cfs and the flow width (b) = 10 ft.

Solving for the critical depth equation for a rectangular channel:

$$y_c = \frac{(q^2)^{1/3}}{(g)^{1/3}}$$

$$y_c = \frac{((50 \text{ cfs}/10 \text{ ft})^2)^{1/3}}{(32.2 \text{ ft/sec}^2)^{1/3}}$$

$$y_c = \frac{(25)^{1/3}}{(32.2)^{1/3}} = \frac{2.9}{3.1}$$

The critical depth in the channel is:

y_c = 0.9 ft

If the normal depth (d_n) calculated by Manning's equation is greater than the critical depth (y_c), then the flow is subcritical and the velocity is low as along the placid pool of a stream.

If the normal depth (d_n) is equal to the critical depth (y_c) , then critical flow exists as at a hydraulic jump at the base of a dam.

If the normal depth (d_n) is less than the critical depth (y_c), then the flow is supercritical and the velocity is high as at the riffles or rapids section (the whitewater) of a stream.

So:

If d_n $> y_c$, then flow is at subcritical depth.

If d_n $> y_c,$ then flow is at critical depth.

If d_n $> y_c$, then flow is at supercritical depth.

2.11 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 where 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 water falls along rivers and streams. Figure 2.21 describes the characteristics of hydraulic jump.

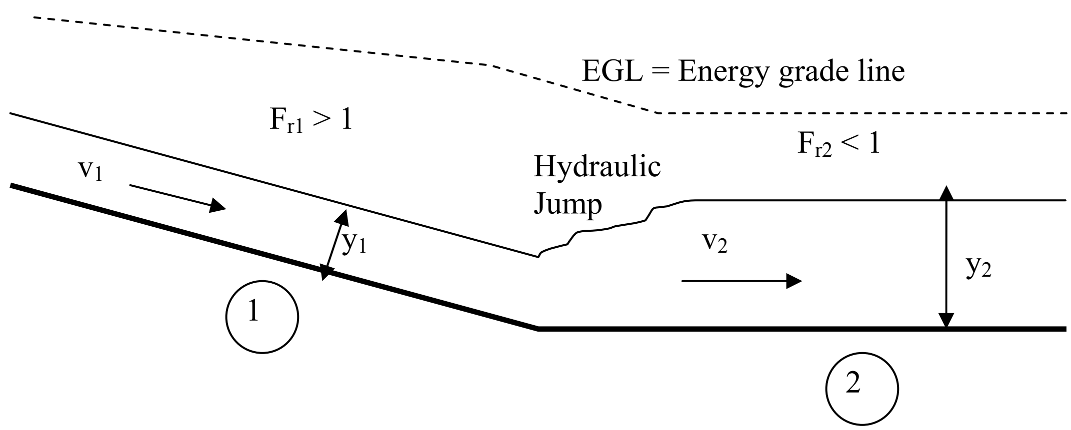


Figure 2.21. Hydraulic jump.

To compute the parameters of hydraulic jump through a rectangular channel, three equations are necessary:

One, the critical depth equation:

$$y_c = \frac{(q^2)^{1/3}}{(g)^{1/3}}$$

Two, the Froude number equation upstream from the hydraulic jump:

$$F_{r1} = \frac{q}{(g(y_1)^3)^{1/2}}$$

Where:

q = specific discharge in a rectangular channel (cfs/ft) = Q/b

Q = discharge (cfs)

b = flow width (ft)

g = acceleration due to gravity = 32.2 ft/sec^2.

y = depth of flow (ft)

And three, the hydraulic jump equation:

$$\frac{y_2}{y_1} = \frac{1((1+8(F_{r1})^2)^{1/2} - 1)}{2}$$

Upstream from the hydraulic jump, where supercritical flow is at high velocity, the depth is lower and the Froude number is greater than 1. Downstream from the hydraulic jump, the flow is subcritical at lower velocity, the depth is higher, and the Froude number is less than 1. Table 2.8 summarizes the various facets of hydraulic jump.

Table 2.8. Facets of hydraulic jump

Parameter	Section 1	Section 2
Location	Upstream	Downstream
Flow Regime	Supercritical	Subcritical
Velocity (v)	Higher	Lower
Flow Depth (d)	Lower	Higher
Froude Number (F_r)	> 1	< 1

Example 2.21

Compute the depth downstream from a hydraulic jump in a 10 ft wide rectangular channel where the flow is 100 cfs and the depth upstream from the jump is 1 ft. Calculate the upstream velocity and the downstream velocity.

Compute the specific discharge in a rectangular channel:

q = Q/b = (100 cfs)/(10 ft) = 10 cfs/ft

Compute the Froude number upstream from the hydraulic jump at section 1:

$$F_{r1} = \frac{q}{(g(y)^3)^{1/2}}$$

$$= \frac{(10 \text{ cfs/ft})}{(32.2 \text{ ft/sec}^2)(1 \text{ ft})^3)^{1/2}}$$

$$F_{r1} = \frac{10}{5.7} = 1.7$$

Compute the flow depth downstream from the hydraulic jump:

$$y_2 = \frac{1((1+(8)(F_{r1})^2)^{1/2}}{2} - 1)\,(y_1)$$

$$= \frac{1((1+8(1.7)^2)^{1/2}}{2} - 1)\,(1)$$

$$= 1.5 \text{ ft}$$

The downstream depth (y_2) = 1.5 ft

By the continuity equation:

$Q = A_1v_1 = A_2v_2$

$Q = 100 \text{ cfs} = 10 \text{ ft}(1 \text{ ft})(v_1)$

$v_1 = 100 \text{ cfs}/10 \text{ sf} = 10 \text{ fps}$

The upstream velocity ($v_{1)}$ = 10 fps

Then, solving for the downstream velocity (v_2):

$Q = 100 \text{ cfs} = A_2\, v_2$
$= (10 \text{ ft})(1.5 \text{ ft})(v_2)$

$v_2 = 100 \text{ cfs}/15 \text{ sf} = 6.7 \text{ fps}$

The downstream velocity ($v_{2)}$ = 6.7 fps

2.12 Surface Water Profile

The standard step method is commonly used to estimate water surface profiles and floodplain delineation for streams and open channels. The method described below is an example from The United States Army Corps of Engineers "Volume 6 – Water Surface Profiles", Hydrologic Engineering Center, Davis, California, 1976 with excerpts from the New Jersey Department of Environmental Protection Stream Encroachment Manual, 1983, .

The standard step method calculation form appears as Figure 1.9 in the text. Columns 2 and 4 through 12 are used to solve Manning's Equation to obtain the energy loss due to friction. Columns 13 and 14 contain calculations for the velocity distribution across the section. Columns 15 through 17 contain the average kinetic energy. Column 18 contains calculations for other losses (expansion and contraction) and column 19 contains the computed change in water surface elevation.

Column 1 – Cross Section No. is the cross section ID number.

Column 2 – ASSUMED, is the assumed water surface elevation in feet (WS) that must agree to within +/- 0.05 feet for the resulting computed water surface elevation calculations to be successful. The starting water surface elevations may be obtained from normal depth as calculated by Manning's Equation.

Column 3 – COMPUTED, is the rating curve value for the first section, but thereafter is the value calculated by adding Δ WS to the computed water surface elevation for the previous cross section.

Column 4 – A is the cross section area (sf). If the section is complex and has been subdivided into several parts (left overbank, channel, and right overbank) use one line of the form for each cross section and sum to get A_t, the total cross section area.

Column 5 – R is the hydraulic radius (ft) which equals A/WP where WP = wetted perimeter (ft).

Column 8 – K is the conveyance and is defined as $(1.49)(A)(R)^{2/3}/n$.

Column 9 – K_t is average conveyance for the reach and is $0.5(k_{td} + k_{tu})$ where "d" refers to downstream and "u" refers to upstream sections of the reach, respectively.

Column 10 – S_f is the average friction slope (ft/ft) through the reach or $(Q/K_t)^2$

Column 11 – L is the distance (ft) between cross sections.

Column 12 – h_f is the energy loss due to friction and is calculated as $h_f = s_f(L) = (Q/K_t)^2 (L)$.

Column 13 - $(K/A)^2$ relates flow velocity distributed across the section to the average velocity.

Column 14 - α is the velocity distribution coefficient usually equal to 1.

Column 15 – v is the average velocity (fps) calculated as Q/A_t.

Column 16 – $\alpha v^2/2g$ is the average velocity head corrected for flow distribution.

Column 17 – $\Delta(\alpha V^2/2g)$ is the difference in velocity heads at the upstream and downstream sections. A positive value indicates the velocity is increasing so use a contraction coefficient (C_c) = 0.1. A negative value indicates that the velocity is decreasing so use an expansion coefficient (C_e) = 0.3.

Column 18 – h is known as other losses and is calculated as C_e or C_c multiplied by $\Delta(\alpha V^2/2g)$.

Column 19 – ΔWS is the change is water surface elevation (ft) from the previous cross section or the sum of columns 12, 17, and 18.

Example 2.22

Compute a water surface profile for the Red Fox River, Colorado for a discharge of 5,000 cfs and assume a starting water surface elevation of 5,709.0 feet. Figure 2.20 contains the form, which summarizes the standard step backwater calculations for the Red Fox River example. Figures 2.21, 2.22. and 2.23 refer to the river plan view, stream cross sections, and values of cross section area and hydraulic radius for the Red Fox River example, respectively. Expansion and contraction coefficients are assumed to be 0.3 and 0.1, respectively. The water surface profile is solved correctly when the difference between the assumed water surface elevation in column 2 is within +/- 0.05 feet of the computed water surface elevation in column 3.

Table 2.9 summarizes the water surface elevation calculations for the Red Fox River.

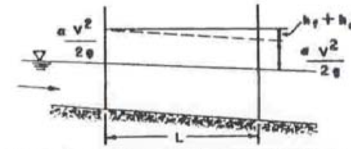
Table 2.9. Water surface profile calculations for the Red Fox River

Cross Section No.	Assumed Water Surface Elevation (ft)	Computed Water Surface Elevation (ft)	Assumed and Computed WS within 0.05 ft?
1		5709.00	
2	5712.60	5713.00	no
	5713.00	5713.00	yes
3	5716.20	5715.90	no
	5715.90	5715.950	yes
4	5720.90	5716.70	no
	5716.90	5716.15	no
	5716.10	5715.49	no

PROJECT: Red Fox River

Q = 5000 cfs

C_e = 0.3 C_c = 0.1



CROSS SECTION NO.	WATER SURFACE ELEVATION ASSUMED	COMPUTED	AREA A	HYDRAULIC RADIUS R	$R^{2/3}$	n	K	$\overline{K}_t$	$\overline{S}_f$	L	h_f	K^3/A^2	α	V	$\alpha \frac{V^2}{2g}$	$\Delta(\alpha \frac{V^2}{2g})$	h_o	Δ(WATER SURFACE ELEVATION)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
												x 10^6						
1		5709.0	355	6.1	3.35	0.03	58,900	---					1.0	14.1	3.08			
2	5712.6	5713.0	360	6.1	3.3	0.03	58,800					1568						
			80	1.6	1.4	0.05	3,300					6						
			440				62,100	60,500		500	3.42	1574	1.27	11.4	2.54	+.54	.05	4.01
	5713.0	5713.0	380	6.3	3.4	0.03	64,000					1815						
			100	1.8	1.5	0.05	4,460					9						
			480				68,460	63,700		500	3.08	1824	1.31	10.4	2.21	+.87	.09	4.04
3	5716.2	5715.9	820	5.4	3.1	0.03	126,600	97,200		400	1.06	---	1.0	6.1	0.58	+1.63	.16	2.85
	5715.9	5715.95	780	5.1	2.9	0.03	112,000	90,200		400	1.22	---	1.0	6.4	0.64	+1.57	.16	2.95
4	5720.9	5716.7	780	6.4	3.4	.036	109,000	110,500		400	0.82	---	1.0	6.4	0.64	0	0	0.82
	5716.9	5716.15	380	4.5	2.7	.036	42,400	77,200		400	1.68	---	1.0	13.2	2.69	-2.05	0.62	0.25
	5716.1	5715.45	315	4.1	2.56	.036	33,300	72,650		400	1.89	---	1.0	15.9	3.92	-3.28	.98	-0.41

NOTE: At 5716.1 V = 15.9 fps, celerity = 12.0 fps, Froude No. = V/c = 1.32. Flow is supercritical.

(8) $K = C_m AR^{2/3}/n$

C_m = 1.486 for English units, 1.0 for Metric units

(9) $\overline{K}_t = .5(K_{upstream} + K_{downstream})$

(10) $\overline{S}_f = (Q/\overline{K}_t)^2$

(12) $h_f = L\,\overline{S}_f$

(14) $\alpha = \frac{(A_t)^2 \Sigma (K_i^3/A_i^2)}{(K_t)^3}$

where: i = incremental value; t = total value

(15) $V = Q/A_t$

(17) $\Delta(\alpha \frac{V^2}{2g}) = (\alpha \frac{V^2}{2g})_{downstream} - (\alpha \frac{V^2}{2g})_{upstream}$

(18a) $h_o = C_e \mid \Delta(\alpha \frac{V^2}{2g}) \mid$ for $\Delta(\alpha \frac{V^2}{2g}) < 0$

(18b) $h_o = C_c \mid \Delta(\alpha \frac{V^2}{2g}) \mid$ for $\Delta(\alpha \frac{V^2}{2g}) > 0$

(19) Δ (water surface elevation) $= \Delta(\alpha \frac{V^2}{2g}) + h_f + h_o$

Figure 2.20. Standard step water surface profile calculations.
(source: U. S. Army Corps of Engineers, Hydrologic Engineering Center, 1976)

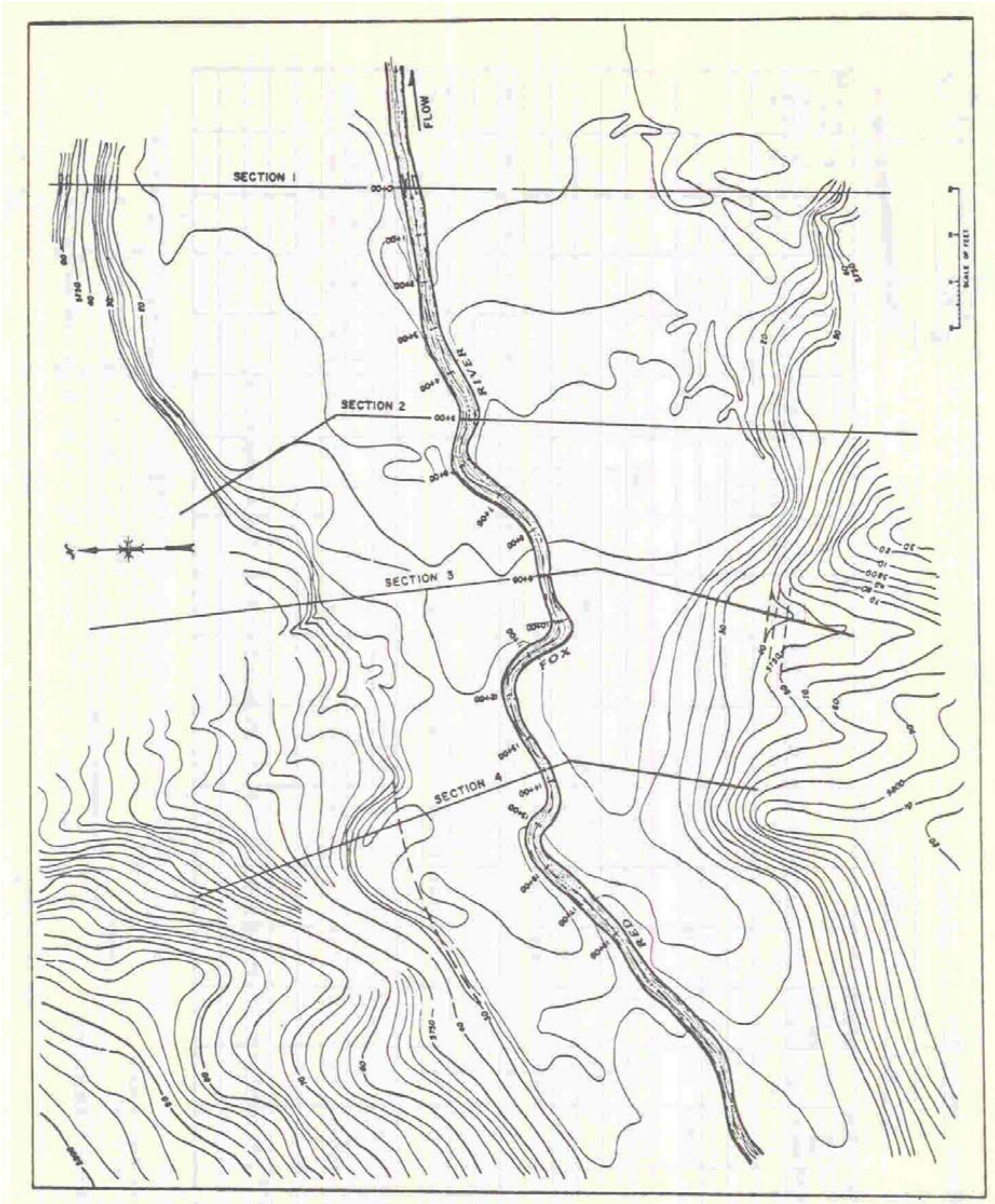


Figure 2.21. Plan view of the Red Fox River, Colorado.
(source: U. S. Army Corps of Engineers, Hydrologic Engineering Center, 1976)

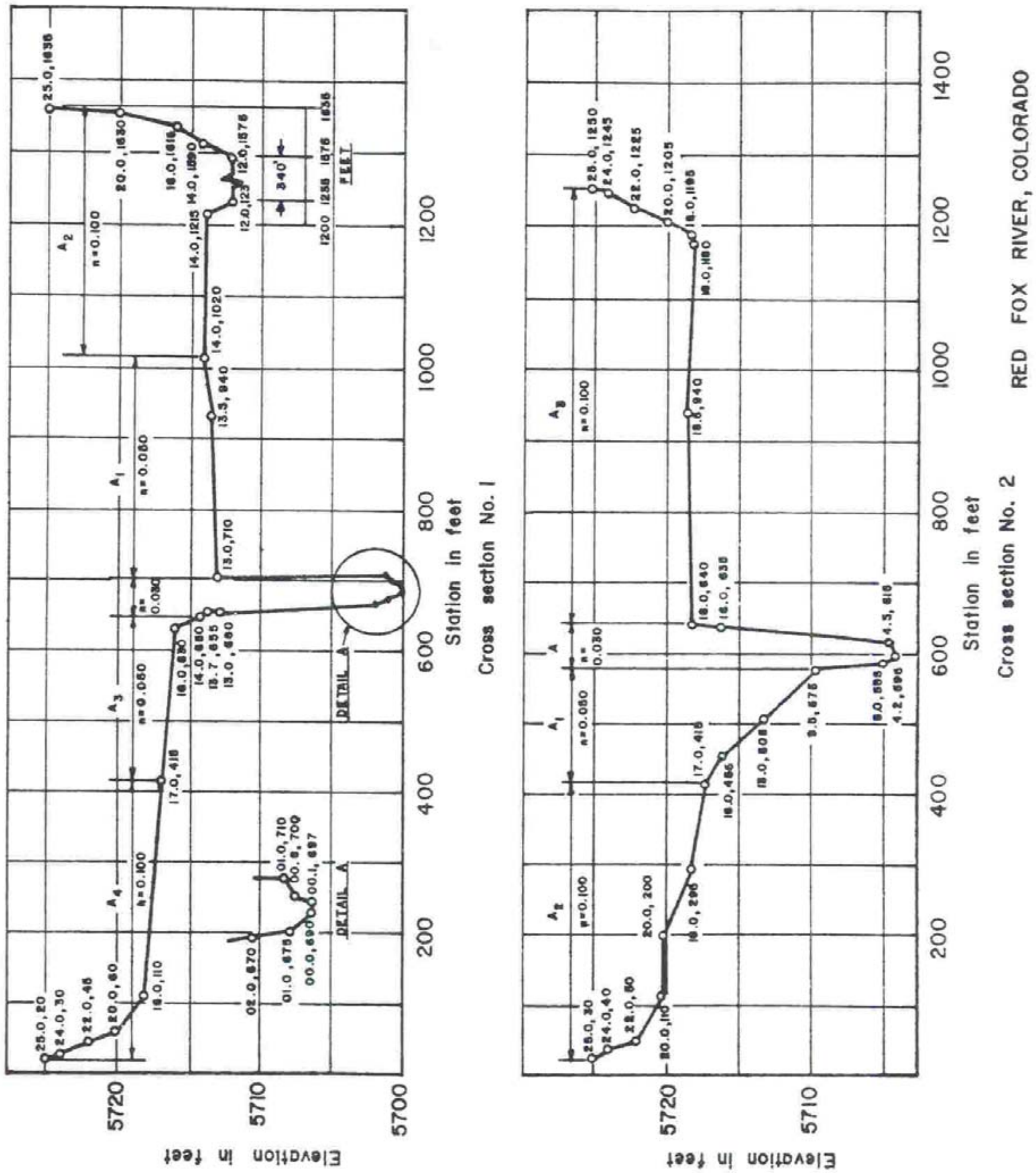


Figure 2.22. Stream cross-sections along the Red Fox River, Colorado (source: U. S. Army Corps of Engineers, Hydrologic Engineering Center, 1976)

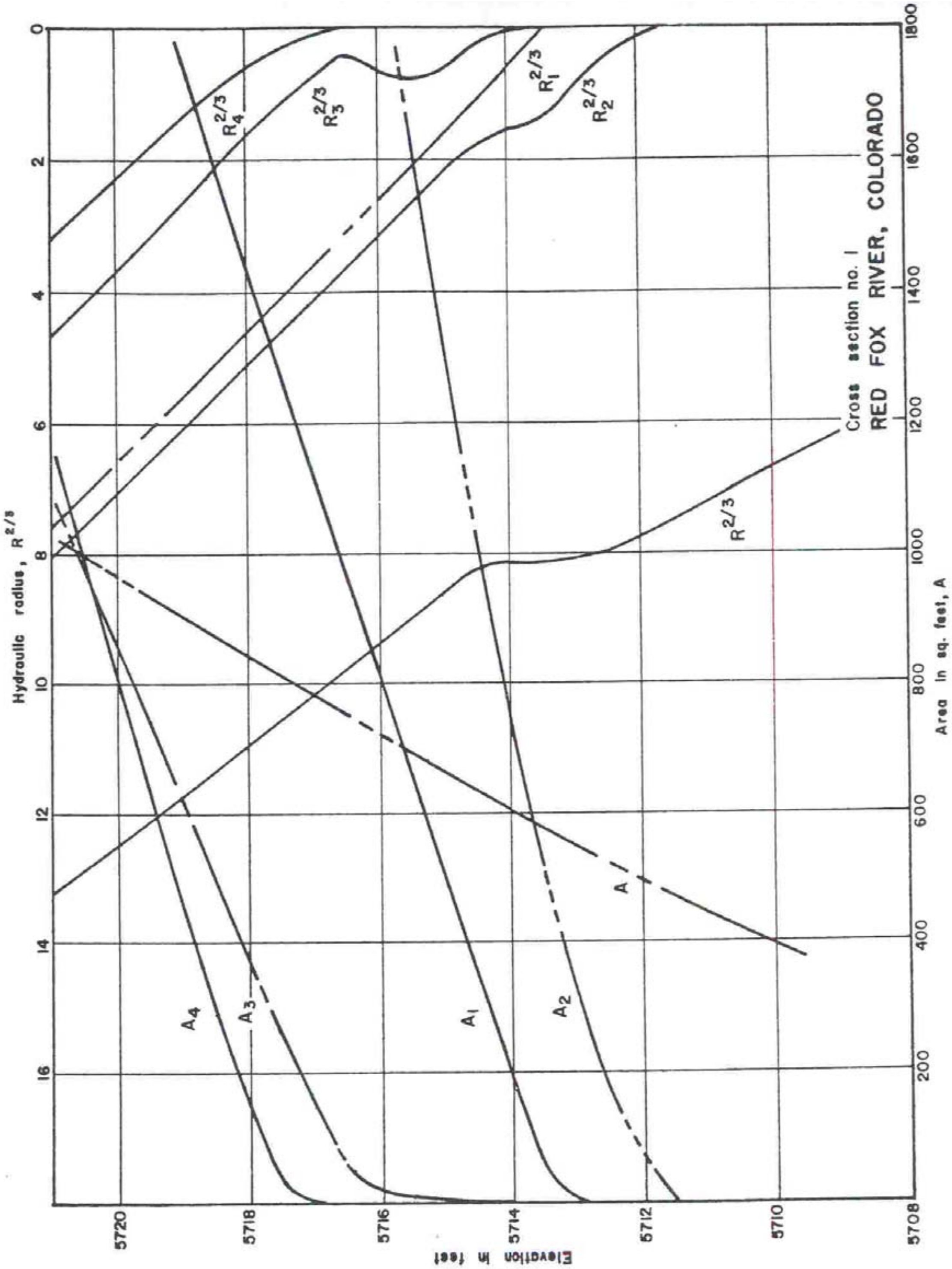


Figure 2.23. Area and hydraulic radius curves for Cross section No. 1.
(source: U. S. Army Corps of Engineers, Hydrologic Engineering Center, 1976)

2.13 Stormwater Collection

Storm sewers are usually designed for the 10 -year design flow (Q_{10}). Manning's equation can be used to calculate design pipe diameters and slopes. Figure 2.24 provides a convenient nomograph for the design of pipe flow using Manning's equation. Figure 2.25 depicts a typical storm sewer system in plan and profile view. The minimum and maximum pipe velocity for storm sewers should range from 2 to 6 fps. The minimum velocity criteria are to prevent settling of sediment and debris in the storm sewer. The maximum velocity criteria is to minimize turbulence in the storm sewer system

Storm sewer design can be accomplished using the following 7 - step process:

1. Sketch out the plan and profile of the storm sewer system. Manholes should be spaced at junctions of the storm sewer pipes and no greater than 500 feet apart to allow access to the system for maintenance. Inlets should be located at intersections and at low points in the gutter.

2. Calculate the 10 – year design flow for each storm sewer section using the Rational method.

3. Estimate invert-elevations of the pipes at each manhole or inlet assuming first that the pipes parallel the ground surface. Calculate the change in invert elevation.

4. Measure the length of the pipe between each node. Calculate the slope of each pipe by dividing the change in invert elevation by the pipe length.

5. Estimate the Manning's roughness value of the storm sewer (n = 0.013 for concrete)

6. Estimate the diameter of each pipe using the Manning's equation nomograph (Figure 2.24). Round up the design pipe size to a standard 6 in diameter increment (i.e. 12 in, 18 in, 24 in, etc). For instance, if the nomograph indicates a 43 in pipe is needed, round up select a 48 inch diameter pipe.

7. Check the selected pipe size for the recommended velocity criteria of 2 (min.) to 6 fps (max.).

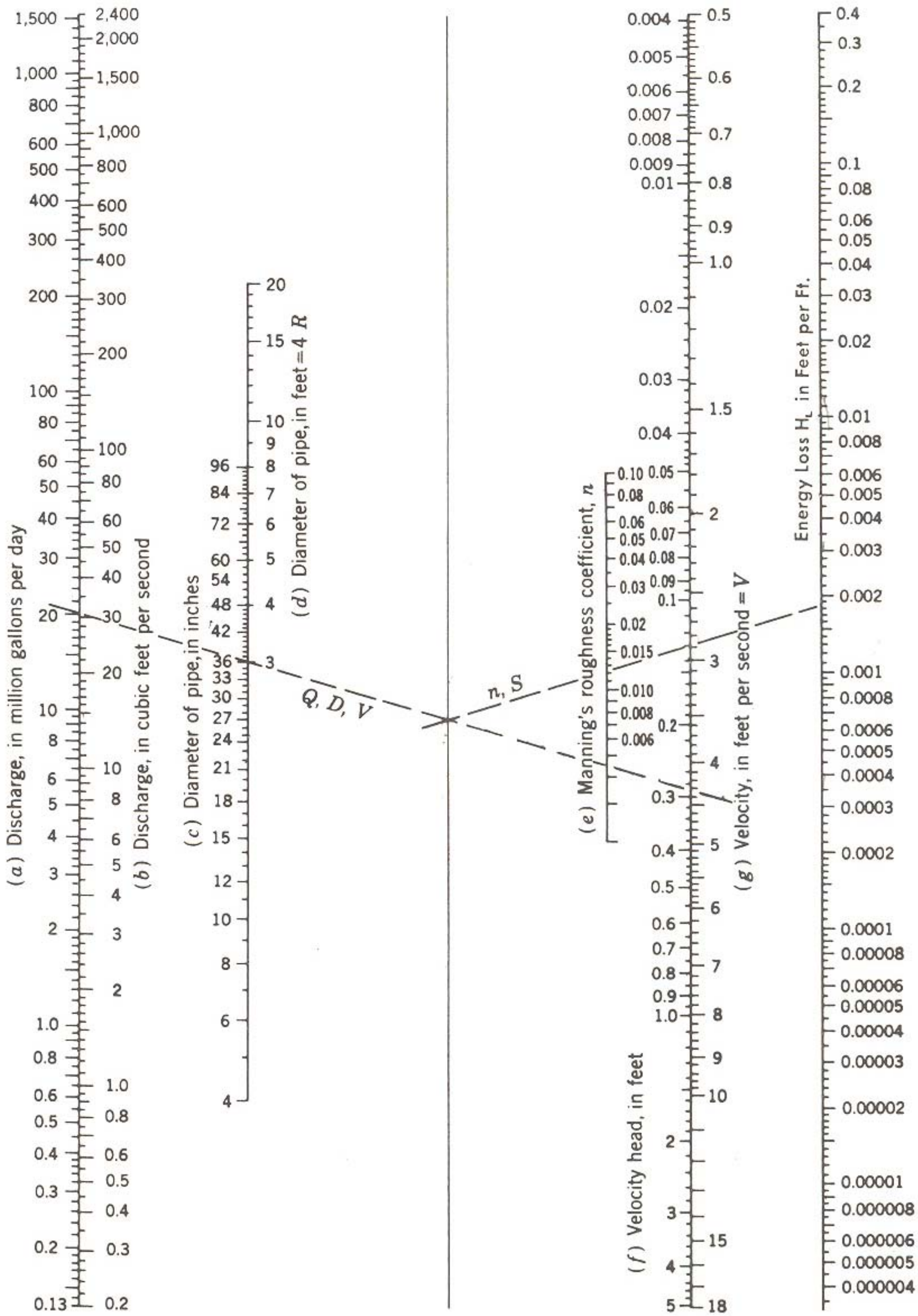


Figure 2.24. Manning's equation nomograph for pipe flow (source: Modern Sewer Design, 1980).

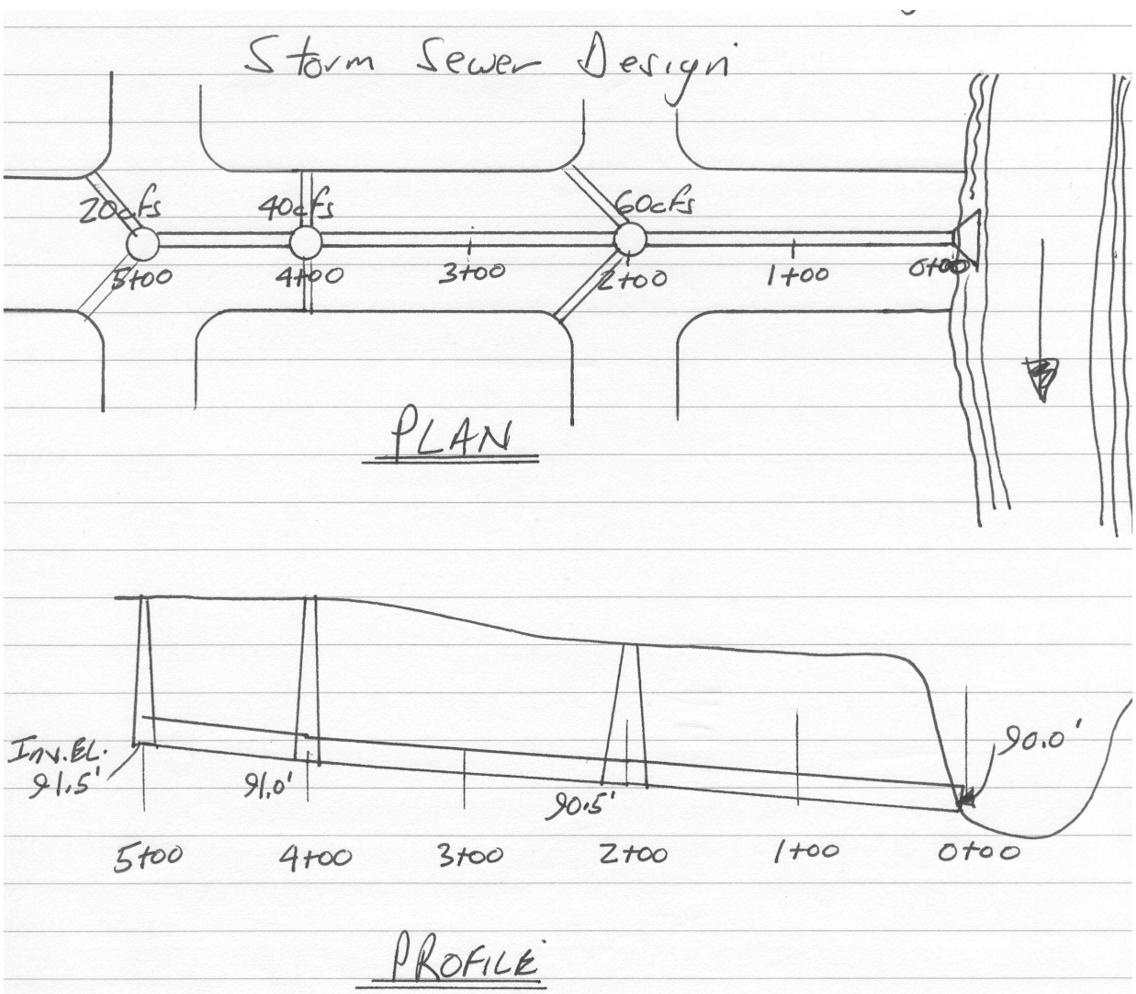


Figure 2.25. Plan and profile of a storm sewer system.

Example 2.22

Design a circular concrete storm sewer pipe at 0.2 % slope for a ten year design flow (Q10) for a drainage area of 100 acres with single family residential land use and the rain fall intensity is given by the following Table 2.11.

Table 2.11. Recurrence interval and rainfall intensity

Recurrence Interval	Intensity (in/hr)
2 yr	6
10 yr	5
25 yr	4
50 yr	3
100 yr	2

Calculate the ten year design flow (Q10) using the Rational formula:

Q_{10} = ciA

C = 0.4 (Table 2.??, single family land use.

i = 5 in/hr

A = 100 ac

Q_{10} = 200 cfs

To size the storm sewer pipe, use the Mannings equation nomograph (Figure 1.??)

S = 0.2% = 0.002 ft/ft

Q_{10} = 200 cfs

,n = 0.013 (concrete pipe)

From the nomograph,

D = 72 in circular concrete pipe.

Check for velocity,

v = 4.5 fps which is within 2 to 6 fps (OK)

Example 2.23

Referring to plan and profile in Figure 2.25, design a storm sewer system given that the 10 –year design flow (Q_{10}) at station 2+00 is 60 cfs, station 4+00 is 40 cfs, and station 5+00 is 20 cfs. Design for a reinforced concrete circular pipe. Check for minimum velocity.

Refer to Table 1.6 containing calculations to design the pipe between Stations 0 +00 and 2+00.

At station 2 + 00, the Q_{10} is 60 cfs.

The change in invert elevation between Station 0+00 and 2+00 is 90.5 ft – 90.0 ft = 0.5 ft.

The length of the pipe is 200 ft.

The slope of the pipe is 0.5 ft/200 ft = 0.0025 ft/ft or 0.25 %.

The Manning's roughness value (n) for concrete pipe is 0.013.

According to the Manning's equation nomograph, the estimated pipe diameter is 48 in.

The estimated velocity is 4.7 fps, which is within the recommended velocity range 2 to 6 fps (OK).

Continue the calculation for the storm sewer pipes between Stations 2+00 to 4+00 and 4+00 to 6+00 as described in Table 2.12.

Table 2.12. Storm sewer design summary

Station	**Q (cfs)**	**Invert Elevation (ft)**	**Δ Inv. El. (ft)**	**Length (ft)**	**Slope (ft/ft)**	**n**	**Find Diameter (in)**	**Check Velocity (fps)**
0+00	0	90	--	--	--	--	--	--
2+00	60	90.5	0.5	200	0.0025	0.013	48"	4.7
4+00	40	91.0	0.5	200	0.0025	0.013	42"	4.7
5+00	20	91.5	0.5	100	0.005	0.013	30"	4.7

2.14 Spillway Capacity

Flow through spillways from dams, reservoirs, and stormwater detention basins may be estimated using the weir flow equations. Generally, three types of weirs are utilized in water resources engineering:

- Rectangular weir
- V - notch weir
- Trapezoidal weir

Rectangular Weir

The rectangular weir flow equation is given by:

$$Q = CLH^{3/2}$$

Where:

Q = flow through the weir (cfs)

C = weir flow coefficient ranging from 3.0 to 4.0, depending on the depth of water flowing over the weir. Usually, the assumed value is C = 3.5 for a rectangular weir.

L = length of the weir (ft)

H = depth of the water surface flowing over the weir (ft)

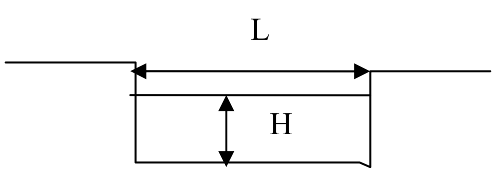


Figure 2.26. Rectangular weir

Triangular Weir

The flow through a V- notch or triangular weir is given by:

$$Q = C\,(0.5)\tan(0/2)\,(2g)^{1/2}\,(H)^{5/2}$$

Where:

C = V – notch weir flow coefficient for = 0.6

0 = angle of the V - notch weir (NOTE TO EDITOR: Can't find Greek letter theta symbol),

g = acceleration due to gravity = 32.2 ft/sec^2

H = depth of the water surface flowing over the weir (ft)

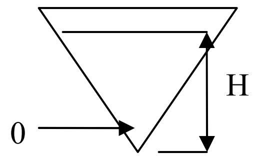


Figure 2.27. V-notch or triangular weir

Trapezoidal Weir

Flow through a trapezoidal weir is given by:

$$Q = CLH^{3/2}$$

Where:

Q = flow through the weir (cfs)

C = trapezoidal weir flow coefficient = 3.3.

L = length of the weir (ft)

H = depth of the water surface flowing over the weir (ft)

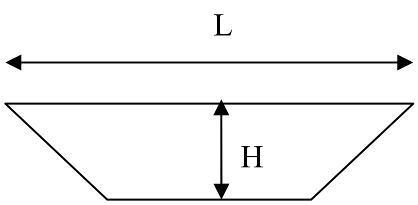


Figure 2.28. Trapezoidal weir

Example 2.24

Calculate the flow through a rectangular weir where the weir length (L) is 20 ft and the depth (H) of flow over the weir is 3 feet.

Using the rectangular weir flow formula:

$$Q = CLH^{3/2}$$

$$Q = 3.5(20\text{ ft})(3\text{ ft})^{3/2}$$

$$Q = 364\text{ cfs}$$

Example 2.25

Estimate the rectangular weir length necessary at the outlet of a detention basin where the water level in the basin is at elevation 101 ft above mean sea level, the crest of the weir is set at Elevation 100 ft msl, and the outlet flow from the basin is 100 cfs.

Using the rectangular weir flow equation:

$Q \quad = CLH^{3/2}$

$Q \quad = 100$ cfs

$C \quad = 3.5$

$H \quad = 101 \text{ ft} - 100 \text{ ft} \qquad = 1 \text{ ft}$

$100 \text{ cfs} = 3.5(L)(1\text{ft})^{3/2}$

$L \quad = 30$ ft

Example 2.26

Calculate the flow through a V- notch weir where the theta angle is 90 degrees and the flow through the weir is 5 ft deep.

Using the V – notch weir formula:

$Q \quad = C\,(0.5)\tan(0/2)\,(2g)^{1/2}\,(H)^{5/2}$

$Q \quad = 0.6\,(0.5)\tan(90/2)\,(2(32.2 \text{ ft/sec}^2))^{1/2}\,(5 \text{ ft})^{5/2} \qquad = (0.6)(0.5)(1)(8)(55.9)$

$Q \quad = 135$ cfs

Example 2.27

Calculate the flow through a trapezoidal weir where the weir length is 20 ft and the depth of flow over the weir is 3 feet.

Using the trapezoidal weir flow formula:

$Q \quad = CLH^{3/2}$

$Q \quad = 3.3(20 \text{ ft})(3 \text{ ft})^{3/2}$

$Q \quad = 343$ cfs

2.15 Flow Measurement Devices

Flow or discharge can be metered using measurement devices such as weirs and flumes. For a discussion concerning weirs as flow measurement devices, please refer to Section 1.13 of this manual. Open channel flow through water treatment and wastewater plants is often measured by using a Parshall Flume(Figure 2.29), a device developed in 1922 by Professor Ralph L. Parshall, a long time professor of civil and environmental engineering at Colorado State University

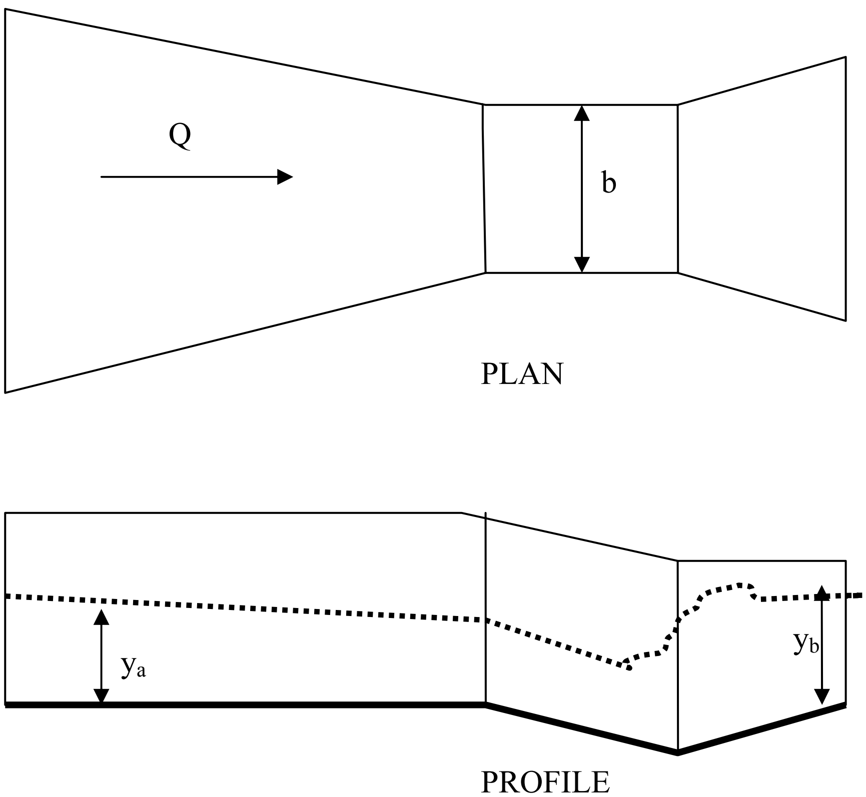


Figure 2.29. Parshall Flume.

The flow equation for the Parshall flume is given by:

$$Q = 4(b)(y_a)^n$$

Where:

Q = flow rate (cfs)

b = flume throat width (ft)

y_a = upstream flow depth (ft)

n $= 1.5(b)^{0.03}$

Example 2.28

Calculate the flow in a 4 feet wide Parshall flume where the upstream flow depth is 2 ft.

Using Parshall Flume equation:

$Q \quad = 4(b)(y_a)^n$

$n \quad = 1.5b^{0.03}$

$n \quad = 1.5(4)^{0.03} = 1.53$

$Q \quad = 4(b)(y_a)^n$

$Q \quad = 4(4 \text{ ft})(2 \text{ ft})^{1.53}$

$\quad = 46.2$ cfs

2.16 Detention/Retention Ponds

Many jurisdictions in the United States have regulations that control the quality and quantity of stormwater from new development by requiring stormwater detention and retention ponds. Detention ponds hold enough volume to store runoff and slowly release the flow to the waterway or storm sewer system primarily through an outlet structure such as a pipe or weir (Figure 2.30). By comparison, retention ponds store the runoff volume and then release the runoff from the pond through infiltration and evaporation, not through an outlet structure.

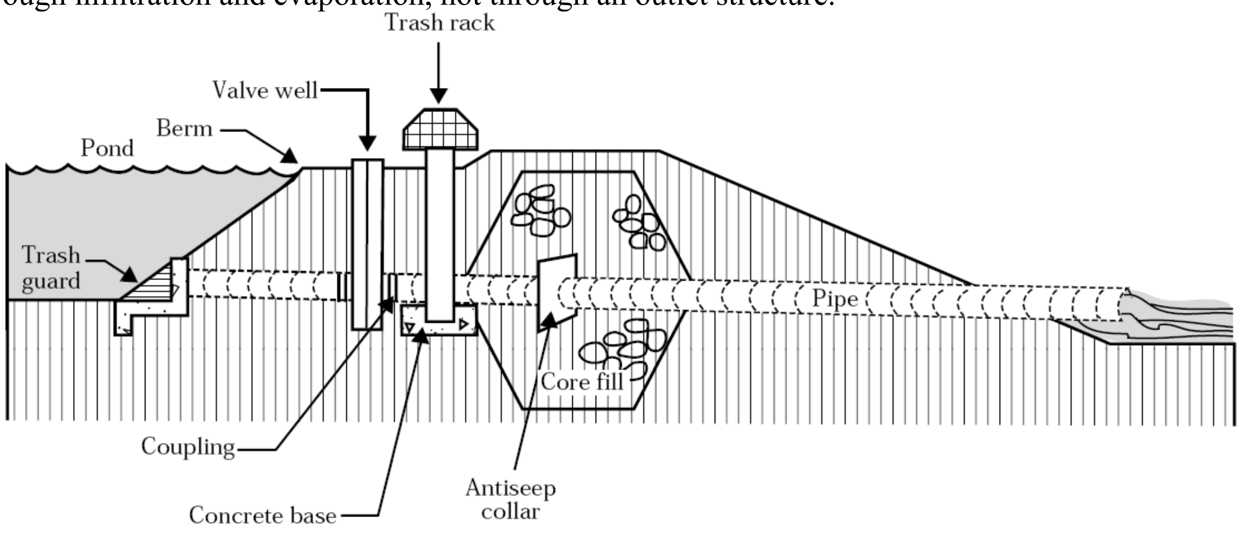


Figure 2.30. Typical stormwater detention pond outlet (source: USDA-NRCS)

Stormwater facilities accomplish quality control by settling out pollutants in runoff and by bio-removal in vegetated basins. Stormwater ponds for quality control are often sized to store the first one or two inches of runoff (the pollutant laden first flush) and for the 1- or 2-year design storm.

Table 2.13 summarizes the pollutant removal efficiencies of various stormwater best management practices including detention and retention ponds.

Table 2.13. Stormwater Best Management Practice Pollutant Removal Efficiencies

Pollutant	Dry Ponds	Wet Ponds	Wetlands	Filters/Bioswales	Infiltration
Bacteria	78%	70%	78%	37%	5%
Total Phosp.	19	51	49	59	70
Nitrate Nitrogen	4	43	67	14	82
TSS	47	80	76	86	95
Cu	26	57	40	49	--
Zn	26	66	44	88	99

Source: Center for Watershed Protection, 2000

Many regulations require stormwater quantity control by specifying that the peak post development stormwater runoff should not exceed the peak predevelopment runoff. This criteria is accomplished by sizing stormwater detention or retention ponds with sufficient volume to store much of the runoff and then slowly release the flow volume though a restricted outlet structure. Stormwater ponds are often sized to release the runoff volume within 48 to 72 hours after the design storm. Depending on the jurisdiction, stormwater ponds for quantity control are often sized for the 100 – year design storm. The maximum ponded depth of stormwater ponds should not exceed 5 feet for safety reasons. The sideslopes should not exceed a ratio of 4 ft horizontal to 1 feet vertical (4 H/1 V) to prevent soil erosion and accommodate mowing and maintenance equipment.

The reservoir routing technique is used to design the storage area and outlet characteristics of stormwater basins and ponds. The reservoir routing technique is defined by the following equation, which is a variation of the conservation of mass, or continuity principle that dictates that inflow must equal outflow:

Δ Storage (S) = Inflow (I) – Outflow (O)

$(S_2-S_1)/t = (I_1+I_2)/2 - (O_1+O_2)/2$

Where:

S = storage of the basin (cf)

I = inflow to the basin (cfs)

O = outflow from the basin (cfs)

t = time step of the hydrograph (sec)

The following example outlines stormwater pond design using the reservoir routing technique.

Example 2.29

Design a stormwater detention pond for the 100-year storm given that the peak post-development discharge shall not exceed the peak pre –development discharge.

1. Determine the 100 – year storm predevelopment and post-development conditions hydrographs as summarized in Table 2.14 and plotted in Figure 2.31.

Table 2.14. Stormwater hydrographs for predevelopment and post- development conditions.

Time (hrs)	Predevelopment (cfs)	Post-development (cfs)
10	0	0
11	2	9
12	10	42
13	25	89
14	38	28
15	23	19
16	17	15
17	14	12
18	10	11
19	8	10
20	6	8
21	4	7
22	2	0
23	1	0
24	0	0

Figure 2.31 Stormwater hydrographs for predevelopment and post- development conditions.

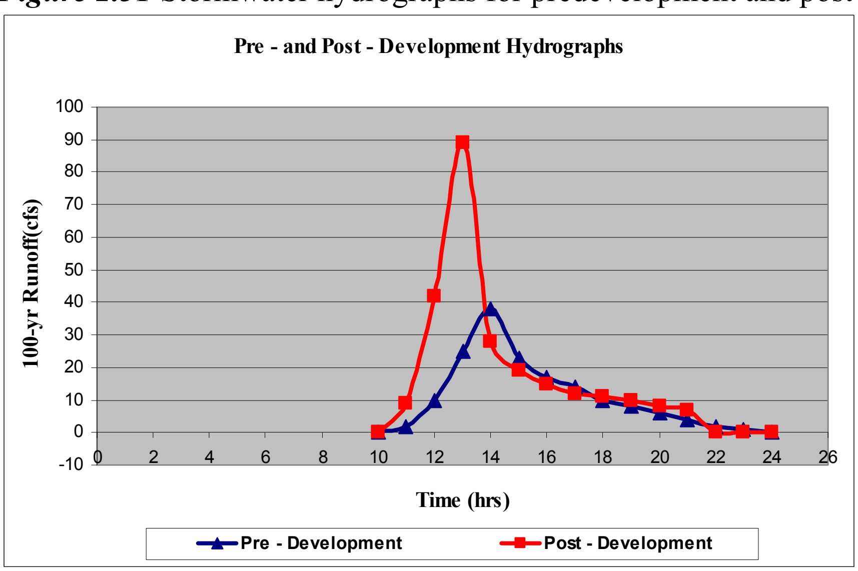


Figure 2.31. Pre and post development hydrographs.

According to the stormwater ordinance, the peak post-development 100-yr discharge should not exceed the peak pre-development conditions discharge. According to the hydrograph illustrated in Figure 2.31, the peak pre-development discharge is 38 cfs. Therefore, a stormwater detention pond should be designed with a maximum 100-year outflow of 38 cfs.

2. Define the outflow rating curve at 0.5 feet intervals of depth for a 1.7 feet wide rectangular weir, C = 3.5. The maximum depth of the storm water pond should not exceed 3 or 4 feet for safety purposes. Refer to Table 2.15 for a summary of the weir outflow rating curve calculations.

Table 2.15. Weir outflow rating curve calculations

Elevation	C	L	H	$H^{3/2}$	$Q=CLH^{3/2}$
0.0	3.5	1.7	0.0	0.00	0
0.5	3.5	1.7	0.5	0.35	2.1
1.0	3.5	1.7	1.0	1.00	6
1.5	3.5	1.7	1.5	1.80	11
2.0	3.5	1.7	2.0	2.80	17
2.5	3.5	1.7	2.5	3.95	24
3.0	3.5	1.7	3.0	5.20	31
3.5	3.5	1.7	3.5	6.50	39 (peak)
4.0	3.5	1.7	4.0	8.00	48

3. Define the surface area-volume relationship for the storm water pond using the average end area method at depth increments of 1 foot as depicted in Table 2.16.

Table 2.16. Surface area – volume relationship for the storm water pond..

Depth H (ft)	Surface Area A (sf)	Average Area $\frac{A_1+A_2}{2}$ (sf)	ΔH (ft)	Incremental. Storage (cu ft)	Cumulative Storage, S cu ft	Weir Outflow (Table 1.9) O cfs	(2S/t)+O cfs
0.0	40,000	--	--	--	--	0	0
1.0	100,000	70,000	1.0	70,000	70,000	6	45
2.0	200,000	150,000	1.0	150,000	220,000	17	139
3.0	300,000	250,000	1.0	250,000	470,000	31	292
4.0	400,000	350,000	1.0	350,000	820,000	48	503
						Δ t = 1 hr = 3600 sec.	

4. Plot a rating curve with weir outflow (O) on the “x” axis versus (2S/t)+O on the ”y” axis (Figure 2.32).

Outflow (cfs)	2S/t + O
0	0
6	46
7	139
31	292
48	503

Figure 2.32. Weir outflow versus 2S/t + O

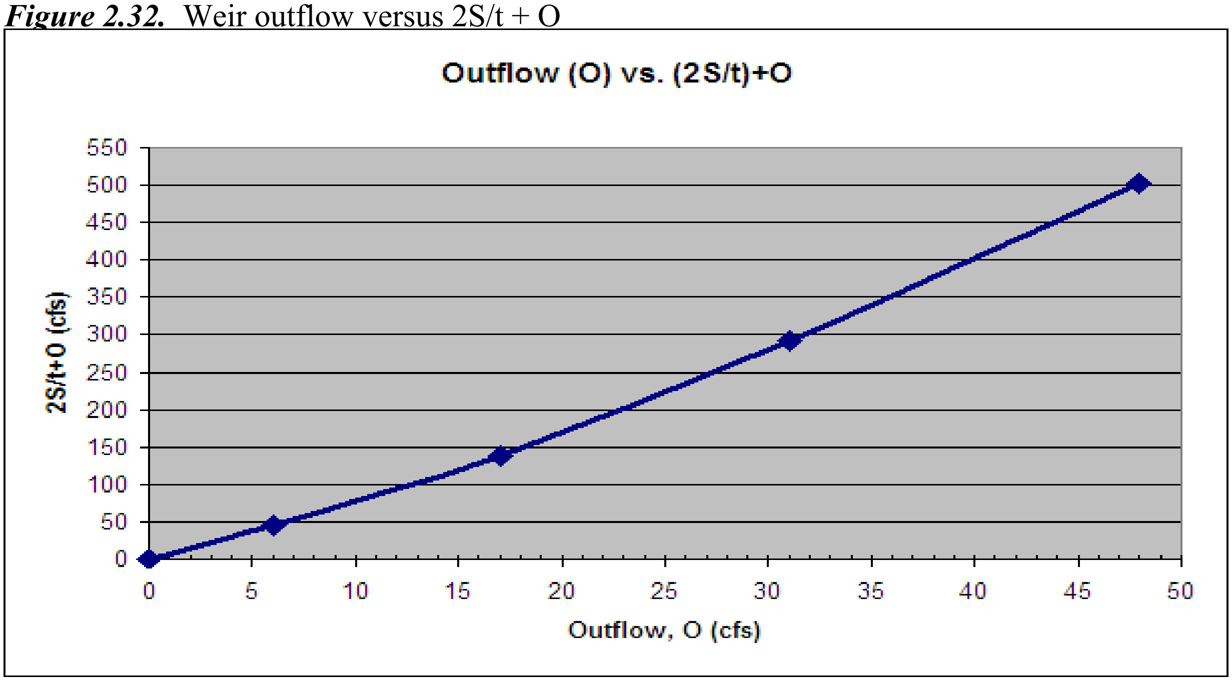


Table 2.17. Stormwater pond routing.

1 Time (hr)	2 Postdevelop Inflow. I (cfs)	3 I_1+I_2 (cfs)	4 (2S/t)-O_1 (cfs)	5 (2S/t)+0_2 (cfs)	6 Outflow O (cfs)
10	0	9	0	--	0
11	9	51	0	--	0
12	42	131	51-2(8)=35 C	51 A	8 B
13	89	117	126	166	20
14	28	47	183	243	30
15	19	34	174	230	28
16	15	27	160	208	24
17	12	23	145	187	21

18	11	21	128	168	20
19	10	18	113	149	18
20	8	15	100	131	15
21	7	7	93	115	11
22	0	0	80	100	10
23	0	0	62	80	9
24	0	0	48	62	7

A: 3+4=5 of next interval
B: Col. 6 obtained from O vs. (2S/t)+O rating curve
C: Col. 4 = Col. 5 - 2(Col. 6)

The maximum surface area is 300,000 sf = 6.9 ac.

The cumulative volume = 470,000 cf = 10.8 ac-ft

6. Plot hydrographs delineating the inflow from developed site into stormwater pond and outflow from the weir outlet structure as shown in Table 2.18 and Figure 2.33.

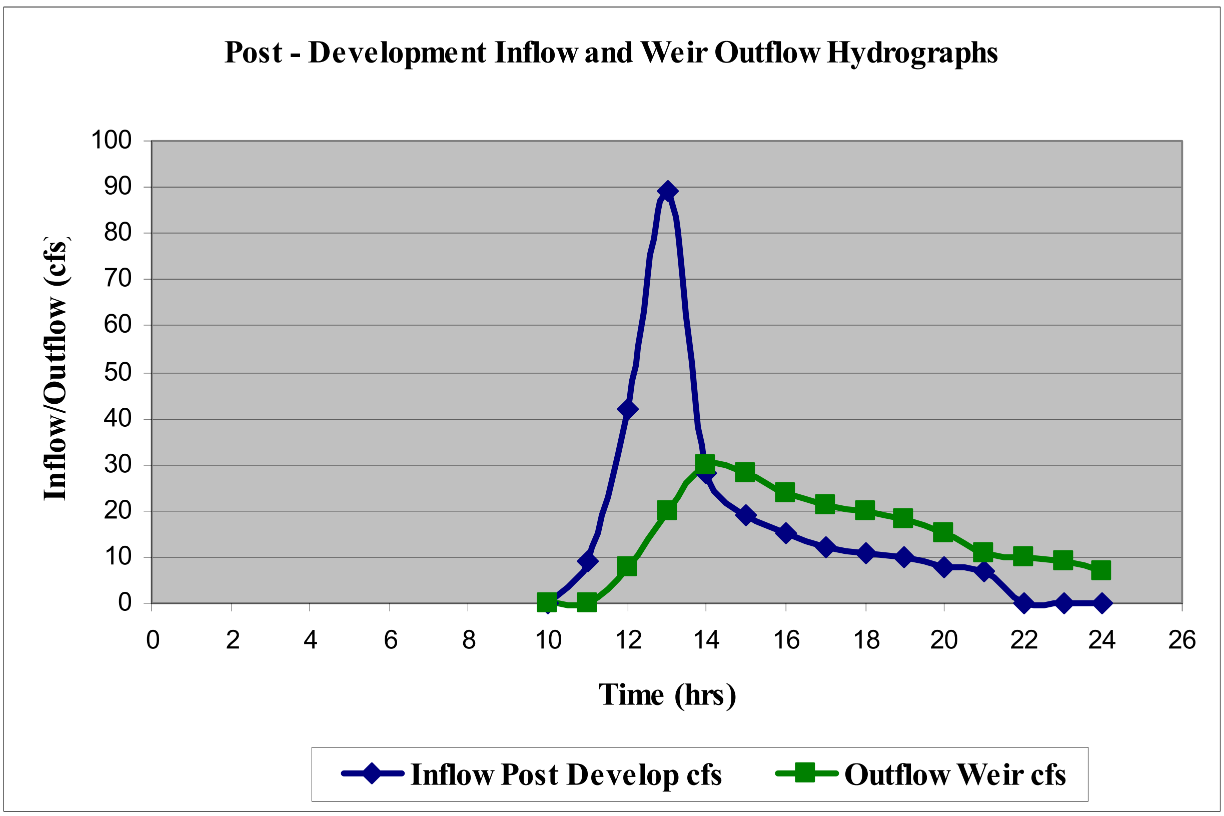


Figure 2.33. Inflow and outflow hydrographs

Table 2.18. Inflow and outflow hydrographs

Time	Inflow	Outflow
	Post Develop	Weir
hrs	cfs	cfs
10	0	0
11	9	0
12	42	8
13	89	20
14	28	30
15	19	28
16	15	24
17	12	21
18	11	20
19	10	18
20	8	15
21	7	11
22	0	10
23	0	9
24	0	7

Summary results:

The peak inflow into the storm water pond = 89 cfs.

The peak outflow from the weir outlet structure = 30 cfs.

The maximum surface area of the stormwater pond at depth of 3.5 feet = 300,000 sf = 6.9 ac.

The volume of the stormwater pond = 470,000 cf = 10.8 ac-ft

2.17 Culvert Design

Culverts are large enclosed channels or pipes that convey flow under roads, railroads, embankments, and other structures. Culverts are usually composed of concrete or corrugated metal structures in standard circular, rectangular, oval, and arch shapes and sizes. Culvert design should incorporate the following hydraulic criteria:

- Culverts under main highways and railroads should be designed to convey the 100 yr flow.
- Culverts under secondary roads, driveways, and bike paths should be conveyed to pass the 50 yr flow.
- The headwater or depth of flow at the entrance to the culvert should not exceed 90 percent of the culvert diameter or rise (0.9D) to allow the culvert to flow partly full to provide

freeboard and allow sufficient capacity for debris to flow through the culvert and avoid clogging.

- At least one feet of freeboard should be included between the design headwater elevation and the roof (underside) of the culvert.

Culvert hydraulics behave under two flow regimes: inlet control and outlet control.

Inlet control occurs where the barrel of the culvert remains unsubmerged by the flow and where the outlet of the culvert flows partly full as depicted in Figure 2.34. Inlet control assumes the hydraulic opening and contraction coefficient at the entrance are the limiting factors in conveying flow through the culvert. Under inlet control the entrance to the culvert may be partly full (if the culvert was designed by the 0.9D criteria) or the inlet may be submerged. The culvert equation for inlet control is:

$Q \quad = C_d A(2gh)^{1/2}$

Where:

Q = discharge or flow (cfs)

C_d = culvert coefficient of contraction which is 0.6 for square edge or unbeveled edge entrance conditions or 1.0 for a rounded or beveled edge entrance condition.

A = cross section area of the culvert (sf)

h = difference (ft) between the headwater depth (HW) upstream from the culvert and the tailwater depth (TW) downstream from the culvert.

,g = acceleration due to gravity = 32.2 ft/sec^2

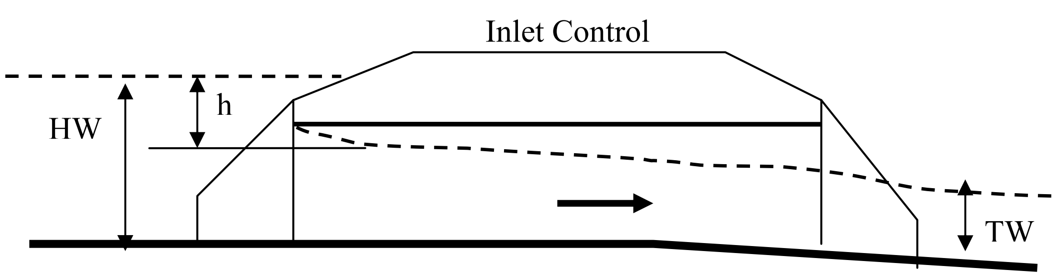


Figure 2.34. Culvert under inlet control.

Outlet control occurs when the barrel and the culvert outlet remains fully submerged (Figure 2.35). The hydraulic opening of the culvert and the friction losses caused by the barrel of the culvert and/or the tailwater downstream from the culvert are the limiting factors in hydraulic capacity of the culvert. The outlet control culvert equation is given by:

$$h = \left(K_e + 1 + \frac{29n^2L}{R^{4/3}}\right)\frac{v^2}{2g}$$

Where:

h = difference (ft) between the headwater depth (HW) and the tailwater depth (TW).

K_e = entrance loss coefficient, 0.5 for a square edged entrance and 0.05 for a rounded entrance.

n = Manning's roughness coefficient of the culvert material.

L = length of culvert barrel (ft)

R = hydraulic radius (ft), culvert cross- section area (A) divided by the wetted perimeter (WP).

v = velocity of flow through the culvert (fps) = Q/A

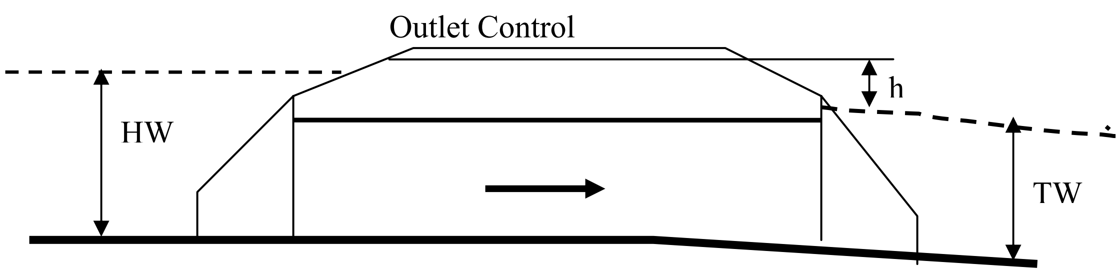


Figure 2.35. Culvert under outlet control.

Example 2.30

Estimate the flow capacity of a 10 ft wide by 5 ft rise concrete box culvert with rounded, beveled edges at the entrance under inlet control and the headwater depth = 6 ft and the tailwater depth = 4 feet. The length of the culvert is 50 ft.

Use the inlet control culvert equation.

$Q = C_dA(2gh)^{1/2}$

Where:

C_d = 1.0 for rounded edge entrances

A = 10 ft x 5 ft = 50 sf

h = 6 ft – 4 ft = 2 ft

g = acceleration due to gravity = 32.2 ft/sec^2

Then the flow capacity of the culvert is

$Q \quad = 1.0(50 \text{ sf})[2(32.2 \text{ ft/sec}^2)(2 \text{ ft})]^{1/2}$

$\quad = 50 \text{ sf } (11.3 \text{ fps})$

$Q \quad = 565 \text{ cfs}$

Example 2.31

Estimate the flow capacity (Q) for 10 ft width by 5 feet rise box culvert under outlet control where headwater depth = 10 ft and the tailwater depth = 9 feet.

Use the outlet control culvert equation:

$$h = \left(K_e + 1 + \frac{29n^2(L)}{(R)^{4/3}}\right)\frac{(v)^2}{2g}$$

$h \quad = 10 \text{ ft} - 9 \text{ ft} = 1 \text{ ft}$

$K_e \quad$ = entrance loss coefficient = 0.05 for a rounded entrance.

$n \quad$ = 0.013 for concrete

$L \quad = 50 \text{ ft}$

$A \quad = 10 \text{ ft}(5 \text{ ft}) = 50 \text{ sf}$

$WP \quad = 10 \text{ ft} + 5 \text{ ft} + 10 \text{ ft} = 25 \text{ ft}$

$R \quad = A/WP = 50 \text{ sf}/25 \text{ ft} = 2 \text{ ft}$

$v \quad = Q/A = Q/50 \text{ sf}$

Substituting into the culvert outlet control equation:

$$1 = \left(0.05 + 1 + \frac{29(0.013)^2(50 \text{ ft})}{(2 \text{ ft})^{4/3}}\right)\frac{(Q/50 \text{ sf})^2}{2(32.2 \text{ ft/sec}^2)}$$

$1 \quad = (0.05 + 1 + 0.10)(Q^2/161{,}000)$

$(Q)^2 \quad = 161{,}000/1.15 = 140{,}000$

$Q \quad = 375 \text{ cfs}$

2.18 Velocity Control

Erosion along open channels can be avoided by utilizing properly sized cross section area, side slopes, and longitudinal slope for a given channel material. The velocity in open channels can be estimated using Manning's equation. Table 2.19 lists the maximum allowable velocities along open channels for a given channel material.

Table 2.19 Maximum allowable velocities along open channels.

Channel Material	Maximum Velocity (fps)
Sand	1.5
Silt	2.0
Pebbles	3.5
Clay	3.0
Gravel	6.0
Stone Cobbles Riprap	7
Concrete	40

Table 2.20 summarizes the minimum sideslopes for a given channel material. The minimum sideslope should be 4 ft horizontal to 1 ft vertical for grass lined channels where mowing equipment will be used to maintain the turf grass.

Table 2.20. Minimum channel side slopes.

Channel Material	Minimum side slope (horizontal/vertical)
Rock	½:1
Firm Soil	1:1
Gravel, loam	1-1/2:1
Sand	3:1
Grass turf, mowed	4:1

Example 3.32

For the trapezoidal channel depicted in Figure 2.36 with a bottom width 10 feet, 3 to 1 sideslopes and depth of flow is 3 feet, define the maximum longitudinal channel slope necessary to prevent erosion with gravel banks,

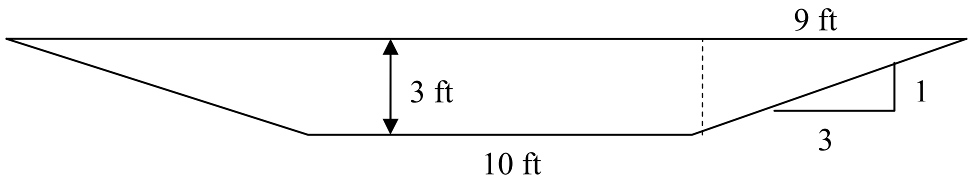


Figure 2.36. Trapezoidal channel

Use Manning's Equation, the minimum channel longitudinal slope can be computed as

$$v = \frac{1.49}{n} R^{2/3} S^{1/2}$$

Where:

n = 0.022 for sand, earth (Table 1.5)

v = 6 fps for gravel (Table 2.19)

A = 10 ft(3 ft) + (3 ft)(9 ft)/2 + (3 ft)(9 ft)/2 = 57 sf

WP = 10 ft + $(9^2 + 3^2)^{1/2}$ + $(9^2 + 3^2)^{1/2}$ = 10 ft + 9.5 ft + 9.5 ft = 29 ft

R = A/WP = 57 sf /29 ft = 2.0 ft

Substitute into Manning's equation and solve for channel slope (S):

$$6 \text{ fps} = \frac{1.49}{0.022} (2.0 \text{ ft})^{2/3} (S)^{1/2}$$

$(S)^{1/2}$ = 0.055

The maximum longitudinal channel slope to prevent erosion should be:

S = 0.003 ft/ft

2.19 Flow Rates (domestic, irrigation, fire)

Table 2.21 summarizes peak daily water demand criteria for residential, commercial, office and industrial uses.

Table 2.21. Water demand criteria

	Residential	Commercial	Office	Industrial
Daily Peak: Lots less than 1 acre	400 gpd / detached du 250 gpd / attached du	0.5 gpd/sf	0.3 gpd/sf	0.5 gpd/sf or actual whichever is more
Daily Peak: Lots more than 1 acre	500 gpd/du	0.5 gpd/sf	0.3 gpd/sf	0.5 gpd/sf or actual whichever is more
Fire Flows (gpm) over a 5 hr period	500-1,000 gpm	1,000 gpm	1,000 gpm	1,500 gpm
Minimum Residual Pressure	20 psi	20 psi	20 psi	20 psi
Minimum Service Pressure	35 psi	35 psi	35 psi	35 psi

Normal residential water demands are estimated at 75 to 100 gallons per capita per day (gpcd).

Peak daily water demands range from 150 to 200 gpcd.

The ratio of peak daily to normal water demands is defined as the peaking factor. The peaking factor can range from 1.5 for older neighborhoods with smaller lots to 2.0 for new neighborhoods with larger lots that require more outdoor watering.

The minimum water pressure at each customer service connection should be 35 psi in accordance with the Recommended Standards for Water Works. Policies for the Review and Approval of Plans and Specifications for Public Water Supplies (Ten-State Standards), 1997

Example 2.33

Estimate the peak daily water demand for a new subdivision with 200 detached residential dwellings (¼ - acre lots) and 50,000 sf of new office space.

Referring to Table 2.21, the peak daily water demand can be estimated as:

Peak demand = 200 du(400 gpd/du) + 50,000 sf(0.3 gpd/sf)

= 80,000 gpd + 15,000 gpd

= 95,000 gpd.

Example 2.34

Estimate normal and peak daily water demands for a city population of 28,000 where the majority of the dwellings are over 50 years old on ¼ acre lots.

Normal demand = 28,000 people (100 gpcd)

= 2,800,000 gpd

= 2.8 mgd

Assume a peaking factor of 1.5 for older neighborhoods with smaller lots (over 50 years old, ¼ acre lots).

Peak daily demand = 2.8 mgd (1.5)

= 4.2 mgd

Finished water tank storage is calculated by the following formula:

Volume of Storage = Volume of demand in excess of 25% of the peak daily demand

+ Fire storage for 5 hour fire at 5,000 gpm

+ Emergency storage for one day at average daily demand.

Example 2.35

Design the water tank storage volume for a city with population of 28,000 people given normal per capita water use is 150 gpcd and the fire flow requirement is 5000 gpm over 5 hours.

Water tank storage volume = Peaking Factor(population)(normal per capita water use)(0.25)

+ Fire flow rate (duration)

+ Emergency storage for 1 day at average daily demand

Water tank storage volume = 2.0(28,000 people)(150 gpcd)(0.25)

+ 5000 gpm(60 min/hr)(5 hr)

+ 28,000 people (150 gpcd)

Water tank storage volume = 2.1 mg + 1.5 mg + 4.2 mg = 7.8 mg

2.20 Hydraulics Sample Problems

Problem 2.1

Calculate the volume (V) in mg from 5 in of rain falling over a 1000 ac watershed.

V	=	5 in (1 ft /12 in) (1000 ac)	= 417 ac-ft
	=	417 ac-ft (43,560 sf / ac)	= 18,164,520 cf
	=	18,164,520 cf (7.48 gal/cf)	= 135,870,460 gal
V	=	136 mg	

Problem 2.2

Estimate the flow (Q) in a creek in mgd if the USGS stream gage estimates the flow is 100 cfs.

Q = 100 cfs (60 sec / min) (60 min / hr) (24 hr / day) (7.48 gal / cf)

= 64,627,200 gpd

Q = 65 mgd

Problem 2.3

A water purveyor estimates the peak water demand is 4 mgd. What is the design flow (Q) for the water distribution system in gpm?

Q = 4 mgd (1,000,000 gal / mg) (1 day/24 hr) (1 hr / 60 min)

Q = 2,800 gpm

Problem 2.4

What is the time (t) in days to drain a 4 billion gallon reservoir with an outlet pipe capacity of 100 cfs?

t = 1 bg (4,000,000,000 gal / bg) (1 cf /7.48 gal) / 100 cfs

= 5,347,592 sec (1 min / 60 sec) (1 hr / 60 min)

= 1,484 hr (1 day / 24 hr)

t = 62 days

Problem 2.5

Calculate the flow through a rectangular weir where the weir length (L) is 10 ft and the depth (H) of flow over the weir is 1 feet.

The rectangular weir flow coefficient (C) is 3.5.

The rectangular weir flow equation is:

$Q \quad = CLH^{3/2}$

$Q \quad = 3.5(10\ ft)(1\ ft)^{3/2}$

$Q \quad = 35$ cfs

Problem 2.6

Calculate the flow through a V- notch weir where the angle is 90 degrees and the flow through the weir is 1 ft deep.

The v-notch weir flow coefficient (C) is 0.6.

The v-notch weir flow formula is:

$Q \quad = C\ (0.5)\ \tan(0/2)\ (2g)^{1/2}\ (H)^{5/2}$

$Q \quad = 0.6\ (0.5)\ \tan\ (90\ deg/2)\ (2(32.2\ ft/sec^2))^{1/2}\ (1\ ft)^{5/2}$

$Q \quad = 2.4$ cfs

Problem 2.7

Calculate the flow through a trapezoidal weir where the weir length is 10 ft and the depth of flow over the weir is 1 feet.

The trapezoidal weir flow coefficient is 3.3.

The trapezoidal weir flow formula is:

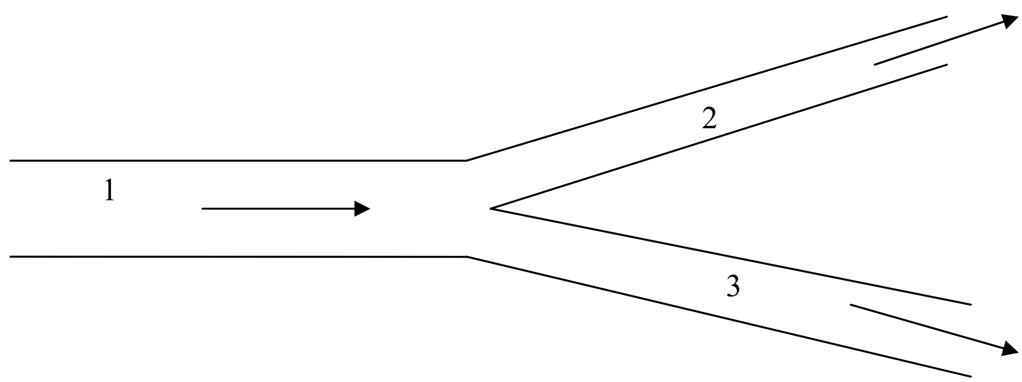
$Q \quad = CLH^{3/2}$

$Q \quad = 3.3(10\ ft)(1\ ft)^{3/2}$

$Q \quad = 33$ cfs

Problem 2.8

A large pipe 1 splits into two smaller pipes 2 and 3. The flow through pipe 1 is 10 cfs and the pipe diameter is 24 in. The flow through pipe 2 is 4 cfs and the diameter is 12 in. The diameter of pipe 3 is 12 in. What is the velocity in pipe 1? What is the flow through pipe 3?



By the continuity equation:

$Q_1 \quad = A_1 v_1 = \pi r^2 (v_1)$

$10 \text{ cfs} = 3.1416((24 \text{ in}/2)(1 \text{ ft}/12 \text{ in}))^2 (v_1)$

$v_1 \quad = 10 / 3.1416$

$v_1 \quad = 3.2 \text{ fps}$

$Q_1 \quad = Q_2 + Q_3$

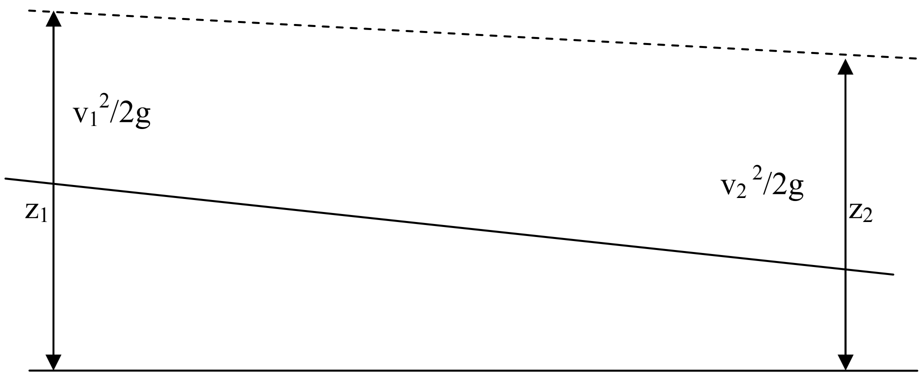
$10 \text{ cfs} = 4 \text{ cfs} + Q_3$

$Q_3 \quad = 10 \text{ cfs} - 4 \text{ cfs}$

$Q_3 \quad = 6 \text{ cfs}$

Problem 2.9

Calculate the velocity at cross – section 2 in an open channel where the velocity at cross - section 1 is 4 fps and the water surface elevation at cross – section 1 is 200 ft and the water surface elevation at cross - section 2 is 199 ft.



Using the energy equation:

$p_1/\gamma + v_1^2/2g + z_1 = p_2/\gamma + v_2^2/2g + z_2$

p = 0 (open channel flow is at atmospheric pressure)

v_1 = 4 fps

z_1 = 200 ft

z_2 = 199 ft

g = acceleration due to gravity = 32.2 ft/sec^2

$0 + (4\text{ fps})^2/2\,(32.2\text{ ft/sec}^2) + (200\text{ ft}) = 0 + v_2^2/2(32.2\text{ ft/sec}^2) + 199\text{ ft}$

$0.25 + 200 = v_2^2/64.4 + 199$

$v_2^2 = 80.5$

$v_2 = 9.0$ cfs

Problem 2.10

Calculate the head loss in a 1000 ft long, 12 in diameter, new ductile iron pipe where the flow rate is 1000 gpm.

Use the Hazen – Williams formula:

$$h_L = \frac{L\,Q^{1.85}}{17{,}076\,(C)^{1.85}(D)^{4.87}}$$

Where:

L = 1000 ft

Q = 1000 gpm

D = 12 in = 1 ft

C = 130 (new, ductile iron pipe from Table 2.1)

$$h_L = \frac{(1000\text{ ft})\,(1000\text{ gpm})^{1.85}}{17{,}076\,(130)^{1.85}(1\text{ ft})^{4.87}}$$

$$h_L = 2.5\text{ ft}$$

Problem 2.11

Compute the flow in an earth trapezoidal channel with a 20 feet bottom width, side slopes at 1 ft horizontal to 1 ft vertical (1:1), depth of flow = 5 feet, and the difference in the channel bottom elevation is 20 feet over 1000 feet in length.

Use Manning's equation:

n = 0.022 (earth from Table 2.3)

A = 20 ft (5 ft) + 5 ft(5 ft)/2 + 5 ft(5 ft)/2 = 125 sf

$WP = ((5\text{ ft})^2 + (5\text{ ft})^2)^{1/2} + ((5\text{ ft})^2 + (5\text{ ft})^2)^{1/2} + 20\text{ ft} = 7.1\text{ ft} + 7.1\text{ ft} + 20\text{ ft} = 34.2\text{ ft}$

R = A/WP = 125 sf/34.2 ft = 3.6 ft

S = 20 ft / 1000 ft = 0.02 ft/ft

$$Q = \frac{1.49}{n} R^{2/3} S^{1/2} (A)$$

$$Q = \frac{1.49}{0.022}(3.6 \text{ ft})^{2/3}(0.02 \text{ ft/ft})^{1/2}(125 \text{ sf})$$

$$Q = 67.7(2.4)(0.14)(125 \text{ sf})$$

$$Q = 2{,}843 \text{ cfs}$$

Problem 2.12

Design a storm sewer system given that the Q10 at station 2+0 is 60 cfs, station 4+0 is 40 cfs, and station 5+0 is 20 cfs. Design for concrete circular pipe. Check for minimum velocity.

Station	Q (cfs)	Invert Elevation (ft)	Δ Inv. El. (ft)	Length (ft)	Slope (ft/ft)	n	Find Diameter (in)	Check Velocity (fps)
0+0	0	90	--	--	--	--	--	--
2+0	60	90.5	0.5	200	0.0025	0.013	48”	4.7
4+0	40	91.0	0.5	200	0.0025	0.013	42”	4.7
5+0	20	91.5	0.5	100	0.005	0.013	30”	4.7

Problem 2.13

Estimate the peak daily water demand for a new subdivision with 5000 detached residential dwellings (¼ acre lots) and 500,000 sf of new office space.

Peak daily demand = 5000 du(400 gpd/du) + 500,000 sf(0.3 gpd/sf)

= 2,000,000 gpd + 150,000 gpd

= 2,150,000 gpd.

Problem 2.14

Estimate normal and peak daily water demands for a city population of 50,000 where the majority of the dwellings are over 50 years old on ¼ acre lots.

Normal demand = 50,000(100 gpcd)

= 5,000,000 gpd

= 5.0 mgd

Assume a peaking factor = 1.5 for older neighborhoods.

Peak daily demand = 5.0 mgd (1.5)

= 7.5 mgd

Problem 2.15

Design the water tank storage volume for a city with population of 10,000 people given normal per capita water use is 150 gpcd and fire flow requirement is 5000 gpm over 5 hours.

Finished water tank storage is calculated by the following formula

Volume of Storage = Volume of demand in excess of or 25% of the peak daily demand

+ Fire storage for 5 hour fire at 5,000 gpm

+ Emergency storage for one day at average daily demand.

Volume of Storage = Peaking factor(population)(normal per capita water use)(0.25)

+ Fire flow rate (duration)

+ emergency storage for 1 day at average daily demand

= 2.0 (10,000 people)(150 gpcd)(0.25)

+ 5000 gpm(60 min/hr)(5 hr)

+ 10,000 people (150 gpcd)

= 0.75 mg + 1.50 mg + 1.5 mg

Volume of Storage = 3.75 mg

Problem 2.16

The allowable net positive suction head for a pump is 50 ft, for water pumped through a 100 ft long pipe of diameter 12 in. The flow rate is 1000 gpm, and C = 130 for its new ductile iron pipe. At what height (z) above the pipe inlet water surface should the pump be located to avoid cavitation? Assume p_{vap} = 0.5 psi for temperature = 80 deg F.

The allowable net positive suction head equation is:

$$NPSH \quad \leq \frac{p_{atm} - p_{vap}}{\gamma} - \Delta z - h_L$$

$$50 \text{ ft} \quad = \frac{(14.7 \text{ psi} - 0.5 \text{ psi})(144 \text{ sq in/sq ft})}{62.4 \text{ pcf}} - \Delta z - \frac{(100 \text{ ft})(1000 \text{ gpm})^{1.85}}{17{,}076(130)^{1.85}(1 \text{ ft})^{4.87}}$$

$$50 \text{ ft} \quad = 33 \text{ ft} - \Delta z - 0.25 \text{ ft}$$

$$\Delta z \quad = 33 \text{ ft} - 50 \text{ ft} - 0.25 \text{ ft} = 17.25 \text{ ft.}$$

Therefore, the pump should be placed no higher than 17.275 ft above the inlet end of the pipe.

Problem 2.17

Calculate the critical depth in a rectangular channel where the discharge (Q) = 100 cfs and the flow width (b) = 10 ft.

The critical depth equation for rectangular channels is:

$$y_c \quad = \frac{(q^2)^{1/3}}{(g)^{1/3}}$$

Where:

$$Q \quad = Q/b$$

$$y_c \quad = \frac{(100 \text{ cfs}/10 \text{ ft})^2)^{1/3}}{(32.2 \text{ ft/sec}^2)^{1/3}}$$

$$y_c \quad = \frac{(25)^{1/3}}{(32.2)^{1/3}} \quad = \frac{4.6}{3.1}$$

The critical depth in the channel is:

$$y_c \quad = 1.5 \text{ ft}$$

Problem 2.18

Compute the depth downstream from a hydraulic jump in a 20 ft wide rectangular channel where the flow is 200 cfs and the depth upstream from the jump is 1 ft. Calculate the upstream velocity and the downstream velocity.

Compute the specific discharge in a rectangular channel:

$q \quad = Q/b = (200 \text{ cfs})/(20 \text{ ft}) = 10 \text{ cfs/ft}$

Compute the Froude number upstream from the hydraulic jump at section 1:

$$F_{r1} = \frac{q}{(g(y)^3)^{1/2}}$$

$$= \frac{(10 \text{ cfs/ft})}{(32.2 \text{ ft/sec}^2)(1 \text{ ft})^3)^{1/2}}$$

$$F_{r1} = \frac{10}{5.7} = 1.7$$

Compute the flow depth downstream from the hydraulic jump:

$$y_2 = \frac{1((1+(8)(F_{r1})^2)^{1/2}}{2} - 1)\ (y_1)$$

$$= \frac{1((1+8(1.7)^2)^{1/2}}{2} - 1)\ (1)$$

$$= 1.5 \text{ ft}$$

The downstream depth (y_2) = 1.5 ft

By the continuity equation:

$Q \quad = A_1v_1 = A_2v_2$

$Q \quad = 100 \text{ cfs} = 10 \text{ ft}(1 \text{ ft})(v_1)$

$v_1 \quad = 100 \text{ cfs}/10 \text{ sf} = 10 \text{ fps}$

The upstream velocity (v_1) = 10 fps

Then, solving for the downstream velocity (v_2):

$Q \quad = 100 \text{ cfs} = A_2\, v_2$
$\quad = (10 \text{ ft})(1.5 \text{ ft})(v_2)$

$v_2 \quad = 100 \text{ cfs}/15 \text{ sf} = 6.7 \text{ fps}$

The downstream velocity (v_2) = 6.7 fps

Problem 2.19

Calculate the flow in a 5 feet wide Parshall flume where the upstream flow depth is 1 ft.

Using Parshall Flume equation:

$n \quad = 1.5b^{0.03}$

$n \quad = 1.5(5)^{0.03} = 1.57$

$Q \quad = 4(b)(y_a)^n$

$Q \quad = 4(5\text{ ft})(1\text{ ft})^{1.57}$

$\quad = 20\text{ cfs}$

Problem 2.20

Estimate the flow capacity for a 12 ft width by 6 ft rise concrete box culvert with rounded, beveled edges at the entrance under inlet control and the headwater depth = 12 ft and the tailwater depth = 10 feet. The length of the culvert is 100 ft.

Use the inlet control culvert equation:

$Q \quad = C_dA(2gh)^{1/2}$

Where:

$C_d \quad = 1.0$ for rounded edge entrance

$A \quad = 12\text{ ft} \times 6\text{ ft} = 72\text{ sf}$

$h \quad = 12\text{ ft} - 10\text{ ft} = 2\text{ ft}$

$Q \quad = 1.0(72\text{ sf})[2(32.2\text{ ft/sec}^2)(2\text{ ft})]^{1/2} = 72\text{ sf}(11.3\text{ fps})$

$Q \quad = 814\text{ cfs}$

3.0 Water Treatment

Water treatment includes the design of water supply treatment and pipeline distribution systems necessary to deliver clean and plentiful drinking water to customers. Professional engineers must be proficient in estimating water demands and designing closed conduit pipe networks using key terms like flow (mgd or gpm), pressure (psi), and velocity (fps). Water treatment design requires knowledge of the chemical and physical processes necessary to purify water to meet Federal and state safe drinking water standards. Other key units in the water treatment discipline include volume (cf or acre-ft) and settling velocity (fps).

The water treatment process removes impurities pollutants from ground and surface water to make the water safe and healthy to drink in accordance with state and federal drinking water standards. The Source Water Protection Program administered by the U. S. Environmental Protection Agency under the Federal Safe Drinking Water Act classifies the following pollutants of concern:

Nutrients (Nitrogen and Phosphorus)

Pathogens (Coliform, Cryptosporidium, Giardia, etc.)

Petroleum Hydrocarbons (Benzene, Toluene, etc)

Pesticides (Alachlor, Endrin, Lindane, etc)

Polychlorinated Biphenyls (PCBs)

Organics (Chloroform, PCE, TCE, etc)

Metals (Copper, Iron, Zinc, etc)

Inorganics (Chloride, Fluoride, Radon, etc)

Typical steps in the water treatment process as depicted in Figure 3.1 include:

- Sources – Raw water supply from ground and surface water.
- Storage – On stream or off stream reservoir.
- Screening – Removes floating debris, solids and fine impurities.
- Rapid Mixing – Mixes chemical used in treatment.
- Flocculation – Addition of agents to promote clumping and settling of fine particles.
- Sedimentation – Removes silt and sand particles.
- Filtration – Removes remaining particles by sand filters leaving water with good clarity.

- Disinfection – Application of chlorine, ultraviolet light, and or ozone to remove bacteria and pathogens.

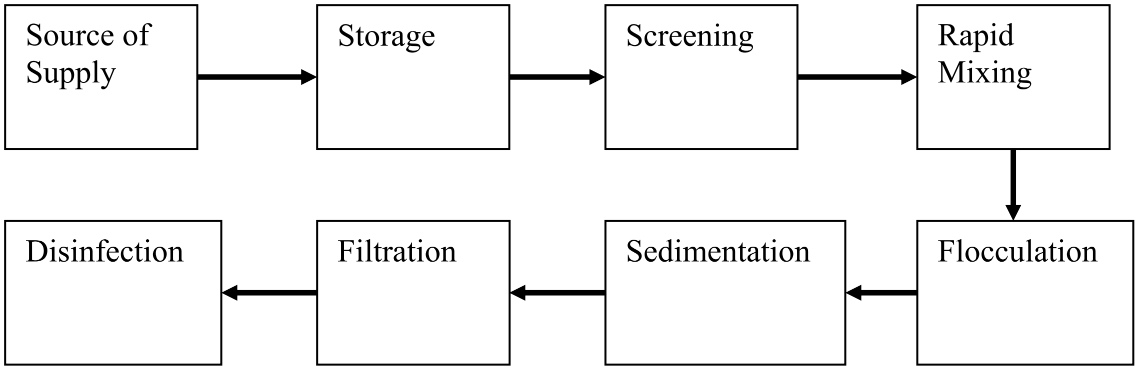


Figure 3.1. Typical water treatment process

3.1 Water Demands

Table 3.1 summarizes peak daily demand criteria for residential, commercial, office and industrial uses.

Table 3.1 Peak daily demand criteria

	Residential	**Commercial**	**Office**	**Industrial**
Daily Peak: Lots less than 1 acre	400 gpd / detached du 250 gpd / attached du	0.5 gpd/sf	0.3 gpd/sf	0.5 gpd/sf or actual whichever is more
Daily Peak: Lots more than 1 acre	500 gpd/du	0.5 gpd/sf	0.3 gpd/sf	0.5 gpd/sf or actual whichever is more
Fire Flows (gpm) over a 5 hr period	500-1,000 gpm	1,000 gpm	1,000 gpm	1,500 gpm
Minimum Residual Pressure	20 psi	20 psi	20 psi	20 psi
Minimum Service Pressure	35 psi	35 psi	35 psi	35 psi

Normal residential water demands are estimated at 75 to 100 gpcd. Peak daily water demands range from 150 to 200 gpcd. The ratio of peak daily to normal daily water demands is defined as the peaking factor. The peaking factor can range from 1.5, for older neighborhoods with smaller lots, to 2.0, for new neighborhoods with larger lots that require more water.

Water distribution systems are designed in accordance with the 10 State Standards that require minimum water pressure of 35 psi at each service connection.

Example 3.1

Estimate the peak daily water demand for a new subdivision with 2000 detached residential dwellings (¼ acre lots) and 500,000 sf of new office space.

Peak daily demand = 2000 du (400 gpd/du) + 500,000 sf (0.3 gpd /sf)

= 800.000 gpd+ 150,000 gpd

= 950,000 gpd.

Example 3.2

Estimate normal and peak daily water demand for a city population of 50,000 where the majority of the dwellings are over 50 years old on ¼ acre lots.

Normal demand = 50,000 (100 gpcd)

= 500,000 gpd

= 5.0 mgd

If the peaking factor = 1.5, then,

Peak daily demand = 5.0 mgd (1.5)

= 7.5 mgd

3.2 Storage (Raw and Treated Water)

Raw (untreated) water storage is usually provided by reservoirs either on-stream or off-stream. On-stream reservoirs are situated along rivers or streams and are filled by the flow of the water into them by gravity. The volume and yield of on-stream reservoir depends on the size of the drainage area of the waterway that flows into the reservoir.

Off-stream reservoirs are also known as pumped storage facilities. At such facilities, water is pumped from a waterway via pipeline into the reservoir, which is usually situated at a high elevation. The water is released from the reservoir back to the water supply system usually via gravity flow through a pipeline.

Example 3.5

Calculate the number of days of storage for a 300 mg reservoir with a water supply yield of 3 mgd.

Days of storage = 300 mg/3 mgd

= 100 days.

Example 3.6

The volume of a 4 feet deep reservoir can be calculated where the elevation – surface area relationship is depicted as follows. Elevation – surface area - volume relationships for reservoirs can be calculated using the average end method.

Depth (ft)	Difference (ft)	Area (sf)	Average Area (sf)	Incr.Volume (cf)	Cum.Volume (cf)
0	0	100,000	0	0	0
1	1	200,000	150,000	150,000	150,000
2	1	300,000	250,000	250,000	400,000
3	1	400,000	350,000	350,000	750,000
4	1	500,000	450,000	450,000	1,200,000

The total volume of the reservoir is 1,200,000 cf or 27 ac – ft.

Treated water storage is usually provided by covered water tanks situated at the highest elevation in a water supply system. The treated water volume is usually sized for 1 of 2 days of reserve supply. Water is released from the elevated water tank back into the system via gravity to generate sufficient water pressure for the customer.

According to the 10 States Standards for water supply systems, the minimum water pressure at the service connection is 35 psi. Normal working pressures in the water distribution network should be 60 to 65 psi.

Water pressure provided by an elevated water tank is calculated by the following formula:

$p \quad = \gamma h$

Where:

$P \quad$ = water pressure (psi)

$\gamma \quad$ = specific weight of water = 62.4 pcf at sea level.

$h \quad$ = difference in elevation from the tank water level to the customer connection.

Example 3.7

What is the water pressure at a service connection to a home (elevation 100 ft msl) if the water tank water level is at elevation 200 ft msl?

$p \quad = \gamma h$

$p \quad$ = 62.4 pcf(200 ft -100 ft)

= 6,250 psf/(144 in/sf)

= 43 psi

The volume of treated water storage tanks are sized according to the following formula:

$V_t \quad = V_d + V_f + V_e$

Where:

V_t = water tank storage volume

V_d = volume of demand in excess (usually 25 %) of maximum daily demand
= peaking factor(population)(normal per capita water demand)(0.25)

V_f = fire storage volume = fire flow rate (duration of fire)

V_e = emergency storage for one day at average daily demand
= population (normal per capita water use)

Example 3.8

Calculate the water tank storage volume needed for a city of 2,800 people where the normal per capita water use is 150 gpcd and the fire flow requirement is 5000 gpm over 5 hours.

V_t = 2.0(2,800 people)(150 gpcd)(0.25) + 5000 gpm(60 min/hr)(5 hr) + 2,800(150 gpcd)

V_t = 210,000 gal + 1,500,000 gal + 420,000 gal

Vt = 2,130,000 gal

= 2.1 mg

3.3 Hydraulic Loading

Hydraulic loading rates are estimated to design various stages in the water treatment process such as mixing, flocculation, sedimentation, and filtration. Hydraulic loading rates are calculated by the following formula:

HLR = Q/A

Where:

HLR = hydraulic loading rate (gpd/sf)

Q = flow rate (gpd)

A = cross section surface area (sf)

Example 3.9

Compute the hydraulic loading rate for a sand filter where the flow rate is 50 gpm and the diameter of the filter is 24 inches or 2 ft.

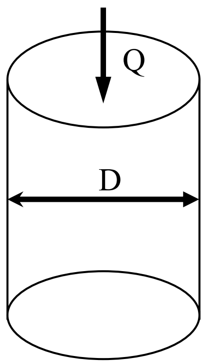


Figure 3.3. Sand filter

Using the hydraulic loading rate formula:

HLR $= Q/A = Q/\pi\ (d/2)^2$

HLR $= 50$ gpm(60 min/hr)(24 hr/day)/ π (2 ft/2)^2

= 72,000 gpd /3.14 sf

= 23,000 gpd/sf

Water treatment processes also require calculation of detention time by the formula:

DT = V/Q

Where:

DT = detention time (hr)

V = volume (cf) = A (H)

A = surface area of tank (sf)

H = depth of tank (ft)

Q = flow rate (gpm, units must be converted)

Example 3.10

Calculate the detention time of a flocculation tank (20 ft diameter by 10 feet deep) where the flow rate is 50 gpm.

DT = V/Q = A(H)/Q

DT = π $(r)^2$(H)/Q

DT = π $(20/2)^2$ (10 ft)/50 gpm(1 cf /7.48 gal)(60 min/hr)

= 3,140 cf / 401 cf/hr

The detention time is:

DT = 7.8 hr.

3.4 Rapid Mixing

Water treatment chemicals are blended with raw water through a process called rapid mixing where the detention time is low, usually 10 to 30 seconds. To promote rapid mixing, the concrete tank volume is small and square in cross section, usually no greater than 300 cf. A vertical shaft impeller is fitted into the square tank.

Tank volume and mixing time can be estimated by the following equation.

$$G(T_d) = \frac{1(PV)^{1/2}}{Q(\mu)^{1/2}}$$

Where:

G = mean velocity gradient, sec^{-1} usually 900 to 1000 for a mixing time of 10 to 30 sec.

T_d = detention time (sec)

P = power requirement (ft-lb/sec)

μ = dynamic viscosity of water approximately 0.000023 lb-sec/sf.

V = rapid mixer volume (cf)

Q = flow rate (cfs)

Example 3.11

Calculate the volume and dimensions of a rapid mixing tank with flow rate of 5 cfs, a mixing time of 10 sec, and power requirement of 200 ft-lb/sec.

The rapid mixing equation is:

$$G(T_d) = \frac{1(PV)^{1/2}}{Q(\mu)^{1/2}}$$

Where:

G = 1000 sec^{-1} for a mixing time of 10 to 30 sec.

T_d = 10 sec

P = 200 ft-lb/sec

μ = 0.000023 lb-sec/sf.

V = rapid mixer volume (cf)

Q = 5 cfs

$$1000\ sec^{-1}\ (10\ sec) = \frac{1(200\ ft\text{-}lb/sec)^{1/2}\ (V)^{1/2}}{5\ cfs(0.000023\ lb\text{-}sec/sf)^{1/2}}$$

$$50{,}000 = \frac{14(V)^{1/2}}{0.0048}$$

$$\underline{(V)^{1/2}} = 17.1$$

$$V = 293\ cf$$

The mixing tank is usually square in cross section. Therefore, design a 6 ft deep x 7 ft wide x 7 ft long square, concrete tank to provide 294 cf volume.

3.5 Flocculation

Coagulating chemicals are added to the turbid raw water to remove particulates through a process called flocculation. After rapid mixing, the particles are gently agitated for a 20 to 30 minute period. During this period, the particles combine to form larger particles (floc). Since the larger particles have greater size and density, they can be removed more easily through settling.

Slowly rotating paddles are used to achieve flocculation. Depending on the turbidity of the water, the G and GT_o values for flocculation are expressed as:

Turbidity	G, sec^{-1}	GT_o
Low	20 -70	60,000 to 200,000
High	10 – 150	90,000 to 180,000

Example 3.12

Determine the volume and dimensions of a circular flocculation tank needed for high turbidity water with average flow of 1000 gpm.

For high turbidity water, select a G = 100 sec-1 and GT_o = 120,000.

T_o = GT_o/G

T_o = 120,000/100

= 1200 sec(1 min/60 sec)

= 20 min

The volume of the basin is then:

V = Q (T_o)

= (1000 gpm)(20 min)(1 cf/7.48 gal)

= 2670 cf

Assuming a 10 feet depth, the surface area (A) is 2670 cf /10 ft = 270 sf.

A = $\pi (r)^2$

270 sf = $\pi (r)^2$

r = $(270/ \pi)^{0.5}$

r = $(86)^{0.5}$

The radius of the tank is then:

r = 9.3 ft, rounded to 9 ft

The diameter (d) is:

d = 2r = 2(9 ft) = 18 ft

The dimensions of the tank are then

18 ft diameter by 10 ft deep.

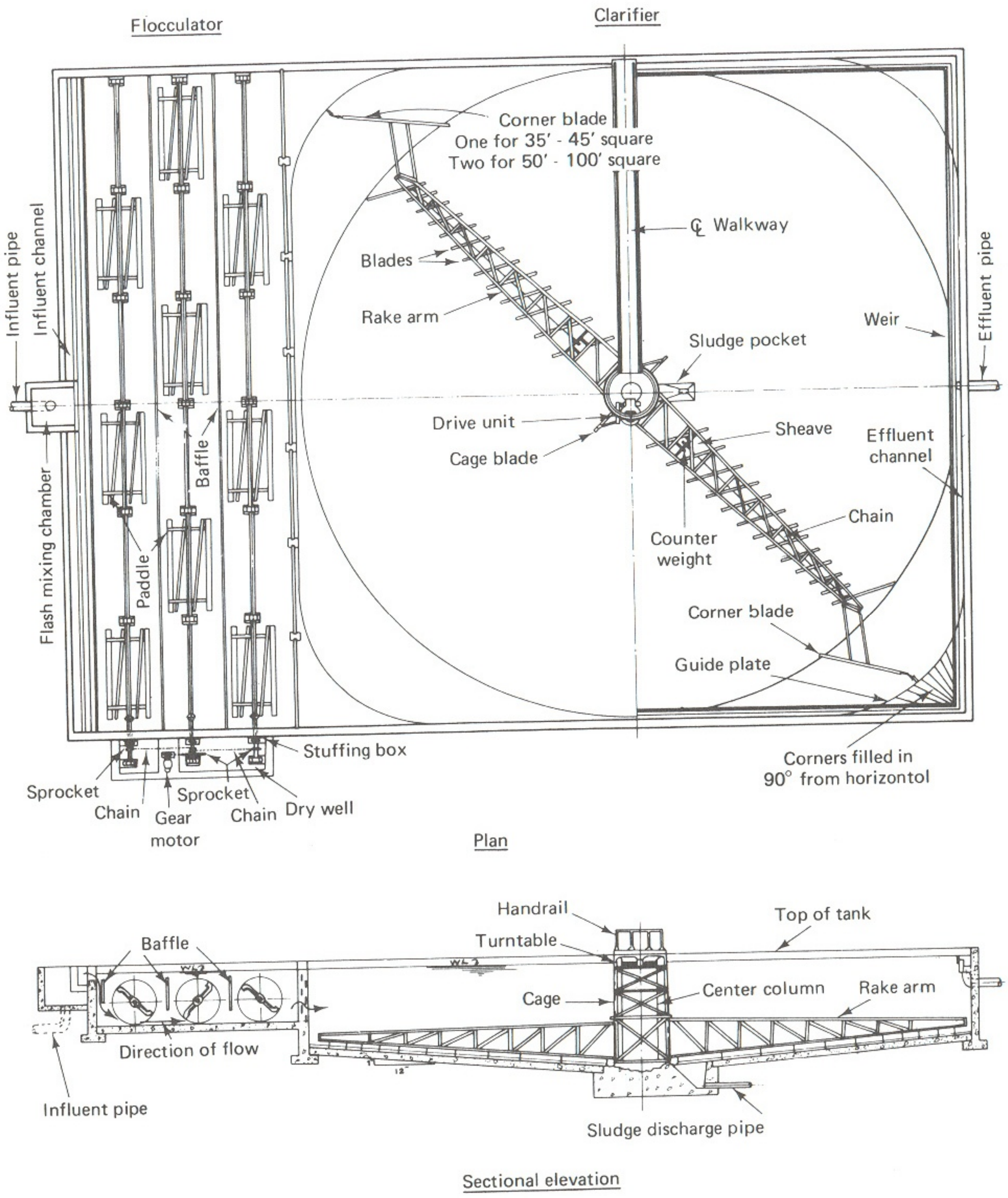


Figure 3.4. Typical sedimentation and flocculation tank (source: Water Resources Engineering, 1992)

3.6 Sedimentation

Sedimentation is used to remove particles from sediment-laden water. The settling rate is dictated by the water density and viscosity and the shape, dimensions, and specific gravity of the sediment. The typical sedimentation parameters are:

Water:	Sediment in cold water will settle less quickly than in warm water.
Particle specific gravity:	Ranges from 1.0 for mud particles to 1.4 for organic matter to 2.65 for sand.
Particle settling velocities:	Refer to Figure 3.5, settling velocity curves.
Detention period:	1 to 10 hrs
Surface overflow rates:	600 to 1200 gal/sf-day
Flow through velocity:	Max. = 1 ft/min
Tank depth:	10 to 15 ft
Tank width:	30 ft
Tank length:	100 to 200 ft
Tank aspect ratio:	3:1 length to width
Tank material:	Concrete
Tank shape:	Rectangular or circular

The minimum settling time is related by:

$T_{settling} = h/v_s$

Where:

$T_{settling}$ = settling time (sec)

h = tank depth (ft)

v_s = settling velocity (fps) from Figure 3.5

The detention time is calculated as:

$T_d = A/Q$

Where:

T_d = detention time

A = cross section area of tank (sf) = length (l) x width (w)

Q = flow rate

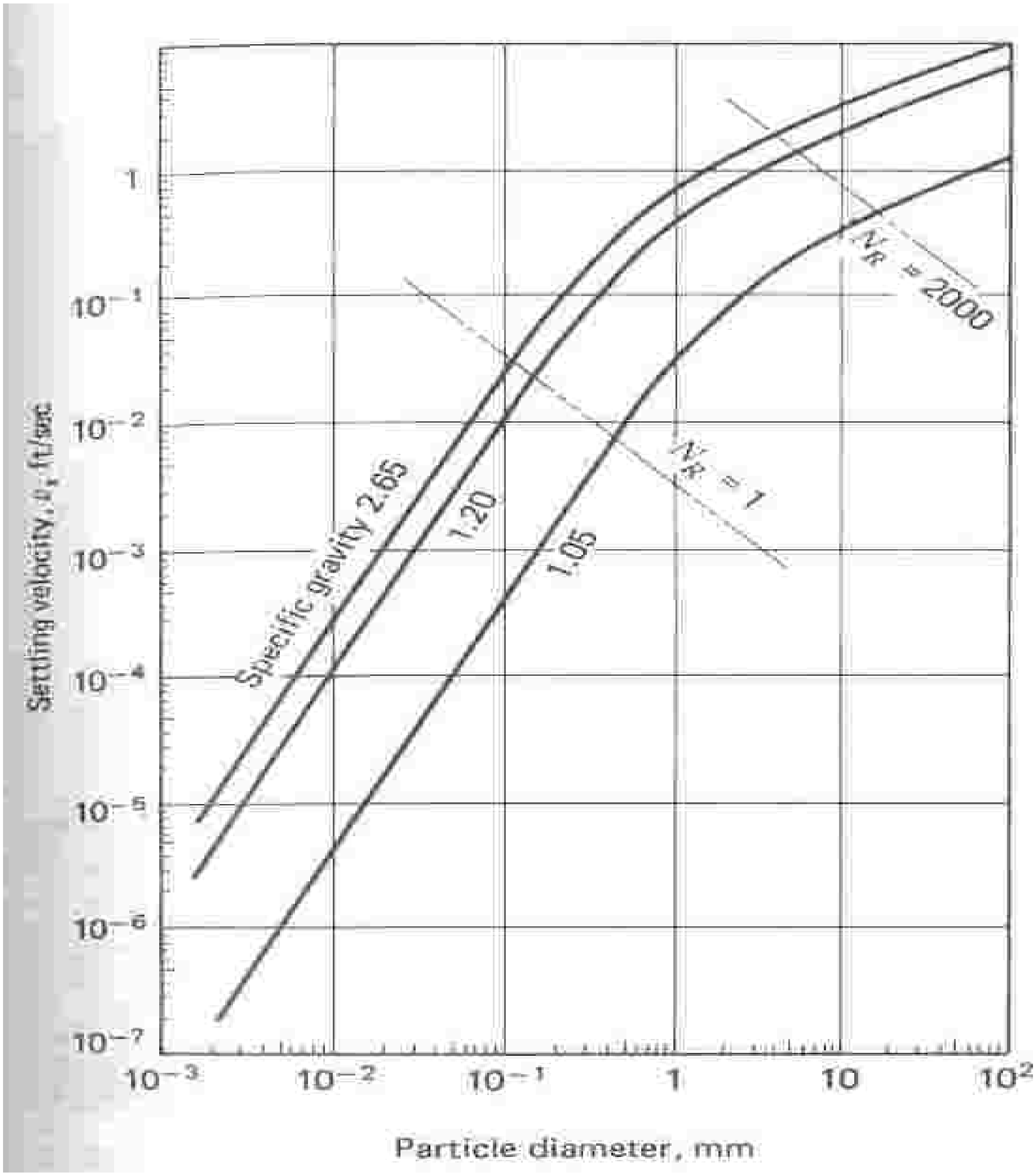


Figure 3.5 Settling velocity of particles

Example 3.13

Determine the rectangular tank dimensions for sedimentation. Assume a sand particle diameter of 0.1 mm and a flow rate of 2,000,000 gal/day. Check for settling time.

Use a surface overflow rate of 600 gal/sf-day.

The surface area of the tank is then:

$A \quad = 2{,}000{,}000 \text{ gal/day}/600 \text{ gal/sf-day}$

$\quad\quad = 3{,}333 \text{ sf}$

For a rectangular tank with length to width aspect ratio of 3:1:

$A \quad = \text{length (l) x width (w)}$

$l \quad = 3w$

Then:

$A \quad = 3w(w) = 3{,}333 \text{ sf}$

$w^2 \quad = 1{,}111 \text{ sf}$

$w \quad = 33 \text{ ft}$

And:

$l \quad = 3(w)$

$\quad\quad = 3(33 \text{ ft})$

$l \quad = 100 \text{ ft}$

Select tank depth:

$h \quad = 15 \text{ ft}$

The tank volume is then:

$V \quad = (15 \text{ ft})\ (3{,}333 \text{ sf}) = 50{,}000 \text{ cf}$

Check detention time:

$T_d \quad = 50{,}000 \text{ cf } (7.48 \text{ gal/cf})/2{,}000{,}000 \text{ gal/day}$

= 0.18 day(24 hr/day)

= 4.5 hr within detention time of 1 to 10 hrs (OK)

3.7 Filtration

Filtration removes algae, particles, floc and other materials remaining in the water after sedimentation. Filters are square in cross section. Water treatment plants usually have 3 to 6 filters so at least one filter can be taken out of service to be backwashed and cleaned. Table 3.2 lists the different filter types and characteristics.

Table 3.2 Filter Types and Characteristics

Filter Type	Bed Materials	Media Thickness (in)	Loading rate (gpm/sf)
Slow sand filter	Sand and gravel	24 to 30 in	1 to 2
Rapid sand filter	Sand and gravel	24 to 30 in	2 to 3
Dual media filter	Sand and coal	12 in sand, 18 in coal	4 to 6
Multi media filter	Sand, coal, and granular activated carbon	12 in sand, 18 in coal, 12 in coal	5 to 10

The filter loading rate (LR) is given as:

LR = Q/A (gpm/sf)

Where:

Q = flow rate (gpm)

A = cross sectional area (sf) of the filter

Example 3.14

Calculate the filter cross section area and number of filters needed for a rapid sand filter with a flow rate of 1000 gpm.

From Table 3.2, select a loading rate of 3 gpm/sf for a rapid sand filter.

LR = Q/A

A = Q/LR

= 1000 gpm/3 gpm/sf

= 333 sf

Assume 6 filters are needed. Each filter cross section area = 333 sf/6 = 55 sf or 7.5 ft by 7.5 ft square.

Specify 6 filters, each 7.5 ft by 7.5 ft for a total of 333 sf of filter cross section filter area.

3.8 Disinfection

Disinfection is necessary to remove bacteria and other pathogens during the last step in the water treatment process. The different types of disinfection include the use of chlorine, ozone, and ultra violet radiation. The most common form of disinfection in the water treatment field is chlorination, a significant advance that has virtually eliminated cholera and other water borne epidemics.

Chlorine can be added as a liquid or gas. A free chlorine residual of 0.2 mg/l to 0.5 mg/l within 10 min after chlorination is needed to achieve disinfection.

When chlorine gas is added to water, hydrolysis occurs to form hypochlorous acid and hydrochloric acid as expressed by the following equation:

$$Cl_2 + H_2O \longleftrightarrow HOCl + Cl^- + H^+$$

Then ionization occurs where the hypochlorous acid dissociates into hypochlorite ion as expressed by:

$$HOCl \longleftrightarrow OCl^- + H^+$$

When calcium hypochlorite as a solid is added to water, it dissociates to form hypochlorite ions as expressed by:

$$Ca(OCl)_2 \longleftrightarrow Ca^{+2} + 2OCl^-$$

Table 3.3 summarizes the atomic weights of the various elements involved in chlorination.

Table 3.3 Atomic weight of the elements involved in chlorination

Element	Weight
Hydrogen	1.0
Calcium	40.1
Oxygen	16.0
Chloride	35.5

The percent available chlorine is calculated as:

$$\% \text{ Available Chlorine} = \frac{\text{Equivalent Weight of Chlorine}}{\text{Equivalent Weight of Compound}}$$

The standard dose equation is given as:

$$F = \frac{D(Q)(8.34)}{Py(Cl)}$$

Where:

F = weight of chemical needed to treat water (lbm/day)

D = dose of substance (mg/l)

Q = flow rate (mgd)

P_Y = purity of the compound (%)

Cl = available chlorine (%)

Example 3.15

How many pounds of calcium hypochlorite are needed to treat water given the flow through the treatment plant is 5 mgd, the hypochlorite ion dose is 25 mg/l, and the purity of calcium hypochlorite is 98 %.

Calculate the molecular weight of calcium hypochlorite $Ca(OCl)_2$:

Calcium	=	40.1
Oxygen 2(16)	=	32.0
Chloride 2(35.5)	=	70.0
Total	=	142.1

The available chlorine (Cl) from hypochlorite ion is:

$$G = \frac{32 + 70}{142.1} = 0.718$$

The weight (F) of calcium hypochlorite needed to treat the water by the dosage equation is then:

$$F = \frac{25 \text{ mg/l } (5 \text{ mgd}) (8.345)}{0.98 (0.718)}$$

F = 1,482 lbm/day

3.9 Applicable Standards

Most water treatment standards in North America are derived from the *Recommended Ten State Standards for Water Treatment* as contained in the *2003 Edition of the Policies for the Review and Approval of Plans and Specifications for Public Water Supplies, A Report of the Water Supply Committee of the Great Lakes--Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers.*

The Ten State Standards were assembled by the following Great Lakes member states and province:

- Illinois
- Indiana
- Iowa
- Michigan
- Minnesota
- Missouri
- New York
- Ohio
- Ontario
- Pennsylvania
- Wisconsin

The Ten State Standards are available by ordering from

Health Research Inc., Health Education Services Division, P.O. Box 7126, Albany, NY 12224 (518) 439-7286 or refer to www.hes.org.

The Ten State Standards are also available on line from

http://www.leafocean.com/test/10statepreface.html.

Pertinent water treatment standards as recommended by the 10 State Standards include:

- PART 1 - SUBMISSION OF PLANS
- PART 2 - GENERAL DESIGN CONSIDERATIONS
- PART 3 - SOURCE DEVELOPMENT
- PART 4 - TREATMENT
- PART 5 - CHEMICAL APPLICATION
- PART 6 - PUMPING FACILITIES
- PART 7 - FINISHED WATER STORAGE

- PART 8 - DISTRIBUTION SYSTEM PIPING AND APPURTENANCES
- PART 9 - WASTE RESIDUALS

3.1.1 Quantity. The quantity of water at the source shall be adequate to meet the maximum projected water demand of the service area as shown by calculations based on a one in fifty year drought or the extreme drought of record, and should include consideration of multiple year droughts.

4.1.1 Presedimentation. Detention time - Three hours detention is the minimum period recommended; greater detention may be required.

4.1.2 Rapid mix. Rapid mix shall mean the rapid dispersion of chemicals throughout the water to be treated, usually by violent agitation. The engineer shall submit the design basis for the velocity gradient (G value) selected, considering the water temperature, color, the chemicals to be added and other related water quality parameters.

Mixing - The detention period should be not more than thirty seconds.

4.1.3 Flocculation. Flocculation shall mean the agitation of water at low velocities for long periods of time.

Detention - The flow-through velocity should be neither less than 0.5 nor greater than 1.5 feet per minute with a detention time for floc formation of at least 30 minutes.

Equipment - Agitators shall be driven by variable speed drives with the peripheral speed of paddles ranging from 0.5 to 3.0 feet per second.

Piping - Flocculation and sedimentation basins shall be as close together as possible. The velocity of flocculated water through pipes or conduits to settling basins shall be neither less than 0.5 nor greater than 1.5 feet per second. Allowances must be made to minimize turbulence at bends and changes in direction.

4.1.4 Sedimentation. Sedimentation shall follow flocculation. The detention time for effective clarification is dependent upon a number of factors related to basin design and the nature of the raw water. The following criteria apply to conventional sedimentation units:

Detention time - shall provide a minimum of four hours of settling time. This may be reduced to two hours for lime-soda softening facilities treating only groundwater. Reduced sedimentation time may also be approved when equivalent effective settling is demonstrated or when overflow rate is not more than 0.5 gpm per square foot (1.2 m/hr).

Outlet weirs and submerged orifices shall be designed as follows:

The rate of flow over the outlet weirs or through the submerged orifices shall not exceed 20,000 gallons per day per foot (250 m^3/day/m) of the outlet launder.

Submerged orifices should not be located lower than three (3) feet below the flow line.

The entrance velocity through the submerged orifices shall not exceed 0.5 feet per second.

Velocity - The velocity through settling basins should not exceed 0.5 feet per minute. The basins must be designed to minimize short-circuiting. Fixed or adjustable baffles must be provided as necessary to achieve the maximum potential for clarification.

Detention period. The detention time shall be established on the basis of the raw water characteristics and other local conditions that affect the operation of the unit. Based on design flow rates, the detention time should be

two to four hours for suspended solids contact clarifiers and softeners treating surface water, and

one to two hours for the suspended solids contact softeners treating only groundwater.

Weirs or orifices. The units should be equipped with either overflow weirs or orifices constructed so that water at the surface of the unit does not travel over 10 feet horizontally to the collection trough.

Weirs shall be adjustable, and at least equivalent in length to the perimeter of the tank.

Weir loading shall not exceed 0 gpm per foot of weir length (120 L/min/m) for units used for clarifiers, 20 gpm per foot of weir length (240 L/min/m) for units used for softeners.

Filtration. Acceptable filters shall include, upon the discretion of the reviewing authority, the following types:

a. rapid rate gravity filters (4.2.1),
b. rapid rate pressure filters (4.2.2),
c. diatomaceous earth filtration (4.2.3),
d. slow sand filtration (4.2.4),
e. direct filtration (4.2.5),
f. deep bed rapid rate gravity filters (4.2.6),
g. biologically active filters (4.2.7),

Rapid rate gravity filter.

Filter material

The media shall be clean silica sand or other natural or synthetic media free from detrimental chemical or bacterial contaminants, approved by the reviewing authority, and having the following characteristics:

a. a total depth of not less than 24 inches and generally not more than 30 inches,
b. an effective size range of the smallest material no greater than 0.45 mm to 0.55 mm,
c. a uniformity coefficient of the smallest material not greater than 1.65,
d. a minimum of 12 inches of media with an effective size range no greater than 0.45 mm to 0.55 mm, and a specific gravity greater than other filtering materials within the filter.

Types of filter media:

1. Anthracite - Clean crushed anthracite, or a combination of anthracite and other media may be considered on the basis of experimental data specific to the project, and shall have
a. effective size of 0.45 mm - 0.55 mm with uniformity coefficient not greater than 1.65 when used alone,
b. effective size of 0.8 mm - 1.2 mm with a uniformity coefficient not greater than 1.85 when used as a cap,
c. effective size for anthracite used as a single media on potable groundwater for iron and manganese removal only shall be a maximum of 0.8 mm (effective sizes greater than 0.8 mm may be approved based upon onsite pilot plant studies or other demonstration acceptable to the reviewing authority).

2. Sand - sand shall have
a. effective size of 0.45 mm to 0.55 mm,
b. uniformity coefficient of not greater than 1.65.

3. Granular activated carbon (GAC) - Granular activated carbon as a single media may be considered for filtration only after pilot or full scale testing and with prior approval of the reviewing authority. The design shall include the following:

\6. Gravel - Gravel, when used as the supporting medium, shall consist of cleaned and washed, hard, durable, rounded silica particles and shall not include flat or elongated particles. The coarsest gravel shall be 2 2 inches in size when the gravel rests directly on a lateral system, and must extend above the top of the perforated laterals. Not less than four layers of gravel shall be provided in accordance with the following size and depth distribution:

Size	Depth
2 1/2 to 1 1/2 inches	5 to 8 inches
1 1/2 to 3/4 inches	3 to 5 inches
3/4 to 1/2 inches	3 to 5 inches
1/2 to 3/16 inches	2 to 3 inches
3/16 to 3/32 inches	2 to 3 inches

Filtration. Rate of filtration - The recommended nominal rate is 1.0 gallon per minute per square foot of filter area (2.4 m/hr) with a recommended maximum of 1.5 gallons per minute per square foot (3.7 m/hr).

Disinfection. Chlorine is historically the preferred disinfecting agent. Disinfection may be accomplished with gas and liquid chlorine, calcium or sodium hypochlorites, chlorine dioxide, ozone, or ultraviolet light. Disinfection with chloramines is not recommended for primary disinfection. The required amount of primary disinfection needed shall be specified by the reviewing authority.

The chlorinator capacity shall be such that a free chlorine residual of at least 2 mg/L can be maintained in the water once all demands are met after contact time of at least 30 minutes when maximum flow rate coincides with anticipated maximum chlorine demand. The equipment shall be of such design that it will operate accurately over the desired feeding range.

Residual chlorine. Minimum free chlorine residual in a water distribution system should be 0.2 mg/L. Minimum chloramine residuals, where chloramination is practiced, should be 1.0 mg/L at distant points in the distribution system.

Distribution System Storage. The maximum variation between high and low levels in storage structures providing pressure to a distribution system should not exceed 30 feet. The minimum working pressure in the distribution system should be 35 psi (240 kPa) and the normal working pressure should be approximately 60 to 80 psi (410 - 550 kPa). When static pressures exceed 100 psi (690 kPa), pressure reducing devices should be provided on mains in the distribution system.

Pressure. All water mains, including those not designed to provide fire protection, shall be sized after a hydraulic analysis based on flow demands and pressure requirements. The system shall be designed to maintain a minimum pressure of 20 psi (140 kPa) at ground level at all points in the distribution system under all conditions of flow. The normal working pressure in the distribution system should be approximately 60 to 80 psi (410 - 550 kPa) and not less than 35 psi (240 kPa).

3.11 Public Drinking Water Standards

The United States Environmental Protection Agency sets drinking water standards in accordance with the Federal Safe Drinking Water Act of 1996 with amendments (Table 3.3). Water Treatment systems are designed to remove impurities in drinking waster to ensure that water is safe to drink.

Table 3.3. USEPA Public Drinking Water Standards.

Contaminant	**MCL [1] (mg/L)[2]**	**Potential Health Effects from Ingestion of Water**	**Sources of Contaminant in Drinking Water**
Cryptosporidium	TT [3]	Gastrointestinal illness (e.g., diarrhea, vomiting, cramps)	Human and fecal animal waste
Giardia lamblia	TT[3]	Gastrointestinal illness (e.g., diarrhea, vomiting, cramps)	Human and animal fecal waste
Heterotrophic plate count	TT[3]	HPC has no health effects; it is an analytic method used to measure the variety of bacteria that are common in water.	HPC measures a range of bacteria that are naturally present in the environment
Legionella	TT[3]	Legionnaire's Disease, a type of pneumonia	Found naturally in water; multiplies in heating systems
Total Coliforms (including fecal	5.0%[4]	Not a health threat in itself; it is used to indicate whether other potentially	Coliforms are naturally present in the environment;

coliform and *E. Coli*)		harmful bacteria may be present[5]	as well as feces; fecal coliforms and *E. coli* only come from human and animal fecal waste.
Turbidity	TT[3]	Turbidity is a measure of the cloudiness of water. It is used to indicate water quality and filtration effectiveness (e.g., whether disease-causing organisms are present). Higher turbidity levels are often associated with higher levels of disease-causing microorganisms such as viruses, parasites and some bacteria. These organisms can cause symptoms such as nausea, cramps, diarrhea, and associated headaches.	Soil runoff
Viruses (enteric)	TT[3]	Gastrointestinal illness (e.g., diarrhea, vomiting, cramps)	Human and animal fecal waste

Contaminant	MCL[1] (mg/L)[2]	Potential Health Effects from Ingestion of Water	Sources of Contaminant in Drinking Water
Bromate	0.010	Increased risk of cancer	Byproduct of drinking water disinfection
Chlorite	1.0	Anemia; infants & young children: nervous system effects	Byproduct of drinking water disinfection
Haloacetic acids (HAA5)	0.060	Increased risk of cancer	Byproduct of drinking water disinfection
Total Trihalomethanes (TTHMs)	0.10 ---------- 0.080	Liver, kidney or central nervous system problems; increased risk of cancer	Byproduct of drinking water disinfection

Contaminant	MRDL[1] (mg/L)[2]	Potential Health Effects from Ingestion of Water	Sources of Contaminant in Drinking Water
Chloramines (as Cl_2)	MRDL= 4.0[1]	Eye/nose irritation; stomach discomfort, anemia	Water additive used to control microbes
Chlorine (as Cl_2)	MRDL= 4.0[1]	Eye/nose irritation; stomach discomfort	Water additive used to control microbes
Chlorine dioxide (as ClO_2)	MRDL= 0.8[1]	Anemia; infants & young children: nervous system effects	Water additive used to control microbes

Contaminant	MCL[1] (mg/L)[2]	Potential Health Effects from Ingestion of Water	Sources of Contaminant in Drinking Water
Antimony	0.006	Increase in blood cholesterol; decrease in blood sugar	Discharge from petroleum refineries; fire retardants; ceramics; electronics; solder
Arsenic	0.010	Skin damage or problems with circulatory systems, and may have increased risk of getting cancer	Erosion of natural deposits; runoff from orchards, runoff from glass & electronicsproduction wastes
Asbestos (fiber >10 micrometers)	7 MFL	Increased risk of developing benign intestinal polyps	Decay of asbestos cement in water mains; erosion of natural deposits
Barium	2	Increase in blood pressure	Discharge of drilling wastes; discharge from metal refineries; erosion of natural deposits
Beryllium	0.004	Intestinal lesions	Discharge from metal refineries and coal-burning factories;

			discharge from electrical, aerospace, and defense industries
Cadmium	0.005	Kidney damage	Corrosion of galvanized pipes; erosion of natural deposits; discharge from metal refineries; runoff from waste batteries and paints
Chromium (total)	0.1	Allergic dermatitis	Discharge from steel and pulp mills; erosion of natural deposits
Copper	TT[8]; Action Level=1.3	Short term exposure: Gastrointestinal distress Long term exposure: Liver or kidney damage People with Wilson's Disease should consult their personal doctor if the amount of copper in their water exceeds the action level	Corrosion of household plumbing systems; erosion of natural deposits
Cyanide (as free cyanide)	0.2	Nerve damage or thyroid problems	Discharge from steel/metal factories; discharge from plastic and fertilizer factories
Fluoride	4.0	Bone disease (pain and tenderness of the bones); Children may get mottled teeth	Water additive which promotes strong teeth; erosion of natural deposits; discharge from fertilizer and aluminum factories
Lead	TT[8]; Action Level=0.015	Infants and children: Delays in physical or mental development; children could show slight deficits in attention span and learning abilities Adults: Kidney problems; high blood	Corrosion of household plumbing systems; erosion of natural deposits

		pressure	
Mercury (inorganic)	0.002	Kidney damage	Erosion of natural deposits; discharge from refineries and factories; runoff from landfills and croplands
Nitrate (measured as Nitrogen)	10	Infants below the age of six months who drink water containing nitrate in excess of the MCL could become seriously ill and, if untreated, may die. Symptoms include shortness of breath and blue-baby syndrome.	Runoff from fertilizer use; leaching from septic tanks, sewage; erosion of natural deposits
Nitrite (measured as Nitrogen)	1	Infants below the age of six months who drink water containing nitrite in excess of the MCL could become seriously ill and, if untreated, may die. Symptoms include shortness of breath and blue-baby syndrome.	Runoff from fertilizer use; leaching from septic tanks, sewage; erosion of natural deposits
Selenium	0.05	Hair or fingernail loss; numbness in fingers or toes; circulatory problems	Discharge from petroleum refineries; erosion of natural deposits; discharge from mines
Thallium	0.002	Hair loss; changes in blood; kidney, intestine, or liver problems	Leaching from ore-processing sites; discharge from electronics, glass, and drug factories

Contaminant	**MCL or TT[1] (mg/L)[2]**	**Potential Health Effects from Ingestion of Water**	**Sources of Contaminant in Drinking Water**
Acrylamide	TT[9]	Nervous system or blood problems; increased risk of cancer	Added to water during sewage/wastewater

			treatment
Alachlor	0.002	Eye, liver, kidney or spleen problems; anemia; increased risk of cancer	Runoff from herbicide used on row crops
Atrazine	0.003	Cardiovascular system or reproductive problems	Runoff from herbicide used on row crops
Benzene	0.005	Anemia; decrease in blood platelets; increased risk of cancer	Discharge from factories; leaching from gas storage tanks and landfills
Benzo(a)pyrene (PAHs)	0.0002	Reproductive difficulties; increased risk of cancer	Leaching from linings of water storage tanks and distribution lines
Carbofuran	0.04	Problems with blood, nervous system, or reproductive system	Leaching of soil fumigant used on rice and alfalfa
Carbon tetrachloride	0.005	Liver problems; increased risk of cancer	Discharge from chemical plants and other industrial activities
Chlordane	0.002	Liver or nervous system problems; increased risk of cancer	Residue of banned termiticide
Chlorobenzene	0.1	Liver or kidney problems	Discharge from chemical and agricultural chemical factories
2,4-D	0.07	Kidney, liver, or adrenal gland problems	Runoff from herbicide used on row crops
Dalapon	0.2	Minor kidney changes	Runoff from herbicide used on rights of way
1,2-Dibromo-3-chloropropane (DBCP)	0.0002	Reproductive difficulties; increased risk of cancer	Runoff/leaching from soil fumigant used on soybeans, cotton,

			pineapples, and orchards
o-Dichlorobenzene	0.6	Liver, kidney, or circulatory system problems	Discharge from industrial chemical factories
p-Dichlorobenzene	0.075	Anemia; liver, kidney or spleen damage; changes in blood	Discharge from industrial chemical factories
1,2-Dichloroethane	0.005	Increased risk of cancer	Discharge from industrial chemical factories
1,1-Dichloroethylene	0.007	Liver problems	Discharge from industrial chemical factories
cis-1,2-Dichloroethylene	0.07	Liver problems	Discharge from industrial chemical factories
trans-1,2-Dichloroethylene	0.1	Liver problems	Discharge from industrial chemical factories
Dichloromethane	0.005	Liver problems; increased risk of cancer	Discharge from drug and chemical factories
1,2-Dichloropropane	0.005	Increased risk of cancer	Discharge from industrial chemical factories
Di(2-ethylhexyl) adipate	0.4	Weight loss, liver problems, or possible reproductive difficulties.	Discharge from chemical factories
Di(2-ethylhexyl) phthalate	0.006	Reproductive difficulties; liver problems; increased risk of cancer	Discharge from rubber and chemical factories
Dinoseb	0.007	Reproductive difficulties	Runoff from herbicide used on soybeans and vegetables
Dioxin (2,3,7,8-TCDD)	0.00000003	Reproductive difficulties; increased risk of cancer	Emissions from waste incineration and other

			combustion; discharge from chemical factories
Diquat	0.02	Cataracts	Runoff from herbicide use
Endothall	0.1	Stomach and intestinal problems	Runoff from herbicide use
Endrin	0.002	Liver problems	Residue of banned insecticide
Epichlorohydrin	TT[9]	Increased cancer risk, and over a long period of time, stomach problems	Discharge from industrial chemical factories; an impurity of some water treatment chemicals
Ethylbenzene	0.7	Liver or kidneys problems	Discharge from petroleum refineries
Ethylene dibromide	0.00005	Problems with liver, stomach, reproductive system, or kidneys; increased risk of cancer	Discharge from petroleum refineries
Glyphosate	0.7	Kidney problems; reproductive difficulties	Runoff from herbicide use
Heptachlor	0.0004	Liver damage; increased risk of cancer	Residue of banned termiticide
Heptachlor epoxide	0.0002	Liver damage; increased risk of cancer	Breakdown of heptachlor
Hexachlorobenzene	0.001	Liver or kidney problems; reproductive difficulties; increased risk of cancer	Discharge from metal refineries and agricultural chemical factories
Hexachlorocyclopentadiene	0.05	Kidney or stomach problems	Discharge from chemical factories

Lindane	0.0002	Liver or kidney problems	Runoff/leaching from insecticide used on cattle, lumber, gardens
Methoxychlor	0.04	Reproductive difficulties	Runoff/leaching from insecticide used on fruits, vegetables, alfalfa, livestock
Oxamyl (Vydate)	0.2	Slight nervous system effects	Runoff/leaching from insecticide used on apples, potatoes, and tomatoes
Polychlorinated biphenyls (PCBs)	0.0005	Skin changes; thymus gland problems; immune deficiencies; reproductive or nervous system difficulties; increased risk of cancer	Runoff from landfills; discharge of waste chemicals
Pentachlorophenol	0.001	Liver or kidney problems; increased cancer risk	Discharge from wood preserving factories
Picloram	0.5	Liver problems	Herbicide runoff
Simazine	0.004	Problems with blood	Herbicide runoff
Styrene	0.1	Liver, kidney, or circulatory system problems	Discharge from rubber and plastic factories; leaching from landfills
Tetrachloroethylene	0.005	Liver problems; increased risk of cancer	Discharge from factories and dry cleaners
Toluene	1	Nervous system, kidney, or liver problems	Discharge from petroleum factories
Toxaphene	0.003	Kidney, liver, or thyroid problems; increased risk of cancer	Runoff/leaching from insecticide used on cotton and cattle

2,4,5-TP (Silvex)	0.05	Liver problems	Residue of banned herbicide
1,2,4-Trichlorobenzene	0.07	Changes in adrenal glands	Discharge from textile finishing factories
1,1,1-Trichloroethane	0.2	Liver, nervous system, or circulatory problems	Discharge from metal degreasing sites and other factories
1,1,2-Trichloroethane	0.005	Liver, kidney, or immune system problems	Discharge from industrial chemical factories
Trichloroethylene	0.005	Liver problems; increased risk of cancer	Discharge from metal degreasing sites and other factories
Vinyl chloride	0.002	Increased risk of cancer	Leaching from PVC pipes; discharge from plastic factories
Xylenes (total)	10	Nervous system damage	Discharge from petroleum factories; discharge from chemical factories

Contaminant	MCL[1] (mg/L)[2]	Potential Health Effects from Ingestion of Water	Sources of Contaminant in Drinking Water
Alpha particles	15 picocuries per Liter (pCi/L)	Increased risk of cancer	Erosion of natural deposits of certain minerals that are radioactive and may emit a form of radiation known as alpha radiation
Beta particles and photon emitters	4 millirems per year	Increased risk of cancer	Decay of natural and man-made deposits of certain minerals that are radioactive and may emit forms of radiation known

			as photons and beta radiation
Radium 226 and Radium 228 (combined)	5 pCi/L	Increased risk of cancer	Erosion of natural deposits
Uranium	30 ug/L as of 12/08/03	Increased risk of cancer, kidney toxicity	Erosion of natural deposits

[1] Definitions:
Maximum Contaminant Level (MCL) - The highest level of a contaminant that is allowed in drinking water.
Maximum Residual Disinfectant Level (MRDL) - The highest level of a disinfectant allowed in drinking water. .
Treatment Technique (TT)- A required process intended to reduce the level of a contaminant in drinking water.

[2] Units are in milligrams per liter (mg/L) unless otherwise noted. Milligrams per liter are equivalent to parts per million.

[3] EPA's surface water treatment rules require systems using surface water or ground water under the direct influence of surface water to (1) disinfect their water, and (2) filter their water or meet criteria for avoiding filtration so that the following contaminants are controlled at the following levels:

- Cryptosporidium: (as of1/1/02 for systems serving >10,000 and 1/14/05 for systems serving <10,000) 99% removal.
- Giardia lamblia: 99.9% removal/inactivation
- Viruses: 99.99% removal/inactivation
- Legionella: No limit, but EPA believes that if *Giardia* and viruses are removed/inactivated, *Legionella* will also be controlled.
- Turbidity: At no time can turbidity (cloudiness of water) go above 5 nephelolometric turbidity units (NTU); systems that filter must ensure that the turbidity go no higher than 1 NTU (0.5 NTU for conventional or direct filtration) in at least 95% of the daily samples in any month. As of January 1, 2002, turbidity may never exceed 1 NTU, and must not exceed 0.3 NTU in 95% of daily samples in any month.
- HPC: No more than 500 bacterial colonies per milliliter.
- Long Term 1 Enhanced Surface Water Treatment (Effective Date: January 14, 2005); Surface water systems or (GWUDI) systems serving fewer than 10,000 people must comply with the applicable Long Term 1 Enhanced Surface Water Treatment Rule provisions (e.g. turbidity standards, individual filter monitoring, Cryptosporidium removal requirements, updated watershed control requirements for unfiltered systems).
- Filter Backwash Recycling; The Filter Backwash Recycling Rule requires systems that recycle to return specific recycle flows through all processes of the system's existing conventional or direct filtration system or at an alternate location approved by the state.

[4] more than 5.0% samples total coliform-positive in a month. (For water systems that collect fewer than 40 routine samples per month, no more than one sample can be total coliform-positive per month.) Every sample that has total coliform must be analyzed for either fecal coliforms or E. coli if two consecutive TC-positive samples, and one is also positive for E.coli fecal coliforms, system has an acute MCL violation.

[5] Fecal coliform and E. coli are bacteria whose presence indicates that the water may be contaminated with human or animal wastes. Disease-causing microbes (pathogens) in these wastes can cause diarrhea, cramps, nausea, headaches, or other symptoms. These pathogens may pose a special health risk for infants, young children, and people with severely compromised immune systems.

[6] Although there is no collective MCLG for this contaminant group, there are individual MCLGs for some of the individual contaminants:

- Trihalomethanes: bromodichloromethane (zero); bromoform (zero); dibromochloromethane (0.06 mg/L). Chloroform is regulated with this group but has no MCLG.
- Haloacetic acids: dichloroacetic acid (zero); trichloroacetic acid (0.3 mg/L). Monochloroacetic acid, bromoacetic acid, and dibromoacetic acid are regulated with this group but have no MCLGs.

[7] MCLGs were not established before the 1986 Amendments to the Safe Drinking Water Act. Therefore, there is no MCLG for this contaminant.

[8] Lead and copper are regulated by a Treatment Technique that requires systems to control the corrosiveness of their water. If more than 10% of tap water samples exceed the action level, water systems must take additional steps. For copper, the action level is 1.3 mg/L, and for lead is 0.015 mg/L.

[9] Each water system must certify, in writing, to the state (using third-party or manufacturer's certification) that when acrylamide and epichlorohydrin are used in drinking water systems, the combination (or product) of dose and monomer level does not exceed the levels specified, as follows:

- Acrylamide = 0.05% dosed at 1 mg/L (or equivalent)
- Epichlorohydrin = 0.01% dosed at 20 mg/L (or equivalent)

National Secondary Drinking Water Regulations

National Secondary Drinking Water Regulations (NSDWRs or secondary standards) are non-enforceable guidelines regulating contaminants that may cause cosmetic effects (such as skin or tooth discoloration) or aesthetic effects (such as taste, odor, or color) in drinking water. EPA recommends secondary standards to water systems but does not require systems to comply. However, states may choose to adopt them as enforceable standards.

- National Secondary Drinking Water Regulations **-** The complete regulations regarding these contaminants availible from the Code of Federal Regulations Web Site.

- For more information, read Secondary Drinking Water Regulations: Guidance for Nuisance Chemicals.

List of National Secondary Drinking Water Regulations

Contaminant	Secondary Standard
Aluminum	0.05 to 0.2 mg/L
Chloride	250 mg/L
Color	15 (color units)
Copper	1.0 mg/L
Corrosivity	noncorrosive
Fluoride	2.0 mg/L
Foaming Agents	0.5 mg/L
Iron	0.3 mg/L
Manganese	0.05 mg/L
Odor	3 threshold odor number
pH	6.5-8.5
Silver	0.10 mg/L
Sulfate	250 mg/L
Total Dissolved Solids	500 mg/L
Zinc	5 mg/L

3.10 Water Treatment Sample Problems

Problem 3.1

Compute the hydraulic loading rate for a sand filter where the flow rate is 100 gpm and the diameter of the filter is 18 in (1.5 ft)..

Using the hydraulic loading rate formula:

HLR $= Q/A = Q/\pi\,(d/2)^2$

HLR $= 100$ gpm(60 min/hr)(24 hr/day)/ π (1.5 ft/2)^2

= 144,000 gpd /1.8 sf

= 80,000 gpd/sf

Problem 3.2

Calculate the detention time of a flocculation tank (24 ft diameter by 12 feet deep) where the flow rate is 100 gpm.

DT $= V/Q = A(H)/Q$

DT $= \pi\,(r)^2(H)/Q$

DT $= \pi\,(24/2)^2$ (12 ft)/100 gpm(1 cf /7.48 gal)(60 min/hr)

= 5,426 cf / 802 cf/hr

The detention time is:

DT = 6.8 hr.

Problem 3.3

Calculate the number of days of storage for a 500 mg reservoir with a water supply yield of 5 mgd.

Days of storage = 500 mg/5 mgd = 100 days.

Problem 3.4

Using the average end area method, calculate the volume of a 4 feet deep reservoir where the elevation – surface area relationship is depicted as follows:

Elevation (ft)	Δ Elev. (ft)	Area (sf)	Average Area (sf)	Incr.Volume (cf)	Cum.Volume (cf)
100	0	50,000	0	0	0
101	1	100,000	75,000	75,000	75,000
102	1	150,000	125,000	125,000	200,000
103	1	200,000	175,000	175,000	375,000

104	1	250,000	225,000	225,000	600,000

The total volume of the reservoir is 600,000 cf or 14 ac – ft.

Problem 3.5

What is the water pressure at a service connection to a home (elevation 200 ft msl) if the water tank water level is at elevation 350 ft msl?

$p \quad = \gamma h$

$p \quad = 62.4 \text{ pcf}(350 \text{ ft} - 200 \text{ ft})$

$= 9{,}360 \text{ psf}/(144 \text{ in/sf})$

$= 65 \text{ psi}$

Problem 3.6

Calculate the water tank storage volume needed for a city of 5,000 people where the normal per capita water use is 150 gpcd and the fire flow requirement is 5000 gpm over 5 hours.

The volume of treated water storage tanks are sized according to the following formula:

$V_t \quad = V_d + V_f + V_e$

Where:

V_t = water tank storage volume

V_d = volume of demand in excess (usually 25 %) of maximum daily demand
= peaking factor(population)(normal per capita water demand)(0.25)

V_f = fire storage volume = fire flow rate (duration of fire)

V_e = emergency storage for one day at average daily demand
= population (normal per capita water use)

$V_t \quad = 2.0(5{,}000 \text{ people})(150 \text{ gpcd})(0.25) + 5000 \text{ gpm}(60 \text{ min/hr})(5 \text{ hr}) + 5{,}000(150 \text{ gpcd})$

$Vt \quad = 375{,}000 \text{ gal} + 1{,}500{,}000 \text{ gal} + 750{,}000 \text{ gal}$

$= 2{,}625{,}000 \text{ gal}$

$= 2.6 \text{ mg}$

Problem 3.8

Calculate the volume and dimensions of a rapid mixing tank with flow rate of 6 cfs, a mixing time of 15 sec and power requirement of 300 ft-lb/sec.

The rapid mixing equation is:

$$G(T_d) = \frac{1(PV)^{1/2}}{Q(\mu)^{1/2}}$$

Where:

G	= 1000 sec^{-1} for a mixing time of 10 to 30 sec.
T_d	= 15 sec
P	= 300 ft-lb/sec
μ	= 0.000023 lb-sec/sf.
V	= rapid mixer volume (cf)
Q	= 6 cfs

$$1000\ sec^{-1}\ (15\ sec) = \frac{1(300\ \text{ft-lb/sec})^{1/2}\ (V)^{1/2}}{6\ \text{cfs}(0.000023\ \text{lb-sec/sf})^{1/2}}$$

$$90{,}000 = \frac{17(V)^{1/2}}{0.0048}$$

$$\underline{(V)^{1/2}} = 25$$

$$V = 625\ cf$$

The mixing tank is usually square in cross section. Therefore, design a 6.2 ft deep x 10 ft wide x 10 ft long square, concrete tank to provide 625 cf volume.

Problem 3.9

Determine the volume and dimensions of a circular flocculation tank needed for low turbidity water with average flow of 2000 gpm.

Slowly rotating paddles are used to achieve flocculation. Depending on the turbidity of the water, the G and GT_o values for flocculation are expressed as:

Turbidity	G, sec^{-1}	GT_o
Low	20 -70	60,000 to 200,000
High	100	100,000

For low turbidity water, select a $G = 50\ sec^{-1}$ and $GT_o = 150{,}000$.

$T_o \quad = G(T_o)/G$

$T_o \quad = 150{,}000/50\ \underline{sec^{-1}}$

$= 3{,}000$ sec

$= 50$ min

The volume of the basin is then:

$V \quad = Q\ (To)$

$= (2000\ gpm)(50\ min)(1\ cf/7.48\ gal)$

$= 13{,}400$ cf

Assuming a 10 feet depth, the surface area (A) is 13,400 cf /10 ft = 1,340 sf.

$A \quad = \pi\ (r)^2$

$1{,}340\ sf = \pi\ (r)^2$

$r \quad = (1{,}340/\ \pi)^{0.5}$

$r \quad = (427)^{\ 0.5}$

The radius of the tank is then:

$r \quad = 21$ ft

The diameter (d) is:

$d \quad = 2r\ = 2(21\ ft) = 42$ ft

The dimensions of the tank are then

42 ft diameter by 10 ft deep.

Problem 3.10

Determine the rectangular tank dimensions for sedimentation. Assume a sand particle diameter of 0.1 and a flow rate of 1,000,000 gal/day. Check for settling time.

Use a surface overflow rate of 600 gal/sf-day.

The surface area of the tank is then:

$A \quad = 1{,}000{,}000 \text{ gal/day}/600 \text{ gal/sf-day}$

$\quad\quad = 1{,}667 \text{ sf}$

For a rectangular tank with length to width aspect ratio of 3:1:

$A \quad = \text{length (l) x width (w)}$

$l \quad = 3w$

Then:

$A \quad = 3w(w) = 1{,}667 \text{ sf}$

$w^2 \quad = 556 \text{ sf}$

$w \quad = 24 \text{ ft}$

And:

$l \quad = 3(w)$

$\quad\quad = 3(24 \text{ ft})$

$l \quad = 72 \text{ ft}$

Select tank depth:

$h \quad = 10 \text{ ft}$

The tank volume is then:

$V \quad = (10 \text{ ft}) (1{,}667 \text{ sf}) = 16{,}670 \text{ cf}$

Check detention time:

$T_d \quad = 16{,}670 \text{ cf } (7.48 \text{ gal/cf})/1{,}000{,}000 \text{ gal/day}$

= 0.12 day (24 hr/day)

= 3.0 hr within detention time of 1 to 10 hrs (OK)

Problem 3.11

Calculate the filter cross section area and number of filters needed for a rapid sand filter with a flow rate of 10,000 gpm.

Select a loading rate (LR) of 3 gpm/sf for a rapid sand filter.

A = Q/LR

= 10,000 gpm/3 gpm/sf

= 3,333 sf

Assume 10 filters are needed. Each filter cross section area = 3,333 sf/10 = 333 sf or 18 ft by 18 ft square.

Specify 10 filters, each 18 ft by 18 ft for a total of 3,333 sf of filter cross section filter area.

Problem 3.12

How many pounds of calcium hypochlorite is needed to treat water given the flow through the treatment plant is 1 mgd, the hypochlorite ion dose is 25 mg/l, and the purity of calcium hypochlorite is 98 %.

Calculate the molecular weight of calcium hypochlorite $Ca(OCl)_2$:

Calcium	=	40.1
Oxygen 2(16)	=	32.0
Chloride 2(35.5)	=	<u>70.0</u>
Total	=	142.1

The available chlorine (G) from hypochlorite ion is:

$$G = \frac{32 + 70}{142.1}$$

$$G = 0.718$$

Q = 1 mgd

D = 25 mg/l

Py = 0.98

The weight (F) of calcium hypochlorite needed to treat the water by the dosage equation is then:

$$F = \frac{D(Q)(8.34)}{Py(Cl)}$$

$$F = \frac{25 \text{ mg/l } (1 \text{ mgd})\ (8.345)}{0.98\ (0.718)}$$

F = 296 lbm/day

Reference Manual for the Water Resources P.M. Depth Portion of the Civil P.E. Exam

G. J. Kauffman **Sep 15, 2006, rev Dec 23, 2006**

Multiple Choice Problems

1.0 Hydraulics

1.1 What is the flow (cfs) through a 24 in pipe where the velocity is measured at 5 fps?
a. 62.8 b. **15.7 (correct)** c. 31.4 d. 125.6 e. 5.0

1.2 If the headloss is 0.5 ft, what is the flow rate (gpm) through a 2 ft diameter, 2000 feet long new ductile iron pipe?
a. 1,000 b. 1,500 c. **1,700 (correct)** d. 3,400 e. 2,100

1.3 What is the velocity (fps) through a 10 feet wide concrete rectangular channel flowing at 5 feet deep with a slope of 2%?
a. 29.7 **(correct)** b. 13.9 c. 4.6 d. 40.1 e. 20.9

1.4 What diameter concrete pipe (in) can convey 200 cfs when full at a slope of 0.02 ft/ft?

a. 36 b. 42 c. 24 d. **48 (correct)** e. 54

1.5 If the critical depth is 1.1 ft through a 10 ft wide concrete channel, what is the discharge (cfs)?
a. 25 b. **65 (correct)** c. 75 d. 100 e. 50

1.6 What is the flow (cfs) through a 10 ft long rectangular weir at depth of 1 feet?
a. **35 (correct)** b. 25 c. 6 d. 40 e. 111

1.7 Estimate the flow capacity of a 12 ft wide by 5 ft rise concrete box culvert with beveled edges under inlet control and headwater depth = 6 ft and tailwater depth = 4 ft.
a. 565 b. **678 (correct)** c. 1179 d. 963 e. 113

1.8 The maximum allowable velocity (fps) for gravel in an open channel is
a. 7.0 b**. 6.0 (correct)** c. 3.0 d. 3.5 e. 1.5

2.0 Hydrology

2.1 If the annual precipitation is 40 in and the runoff is 10 in and evapotranspiration is 22 in, what is the infiltration (in)?
a. 72 in **b. 8 in (correct)** c. 28 in d. 52 in e. 10 in

2.2 What is the 100 yr, 24 hr precipitation depth for Atlanta, Georgia?
a. 6 in b. 7 in **c. 8 in (correct)** d. 10 in e. 9 in

2.3 What is the maximum incremental intensity (in/hr) of a storm based on the following precipitation data?

Hour of Storm	Incremental Precip. Depth (in)
0.0	0.0
0.2	0.1
0.4	0.2
0.6	0.5
0.8	0.2
1.0	0.1

a. 0.5 b. 1.0 **c. 1.5 (correct)** d. 2.0 e. 2.5

2.4 What is the probability of a 50 – yr storm occurring once in any given year?
a. 20% b. 50% c. 200% **d. 2 % (correct)** e. 0.5%

2.5 What is the 100-yr flow (cfs) from a 200 acre watershed in a downtown where the time of concentration is 0.5 hr min?
a. 800 cfs (correct) b. 600 cfs c. 80 cfs d. 1000 cfs e. 4800

2.6 In Delaware, what is the time of concentration (hr) for a 300 ft long sheet flow component over dense grass where the slope is 0.07 ft/ft?
a. 0.15 b. 0.45 ***c. 0.35 (correct)*** d. 0.7 e. 1.35

2.7 What is the runoff curve number for a 1000 acre watershed that is 50% single family residential (½ acre lots) and 50% open space (grass cover exceeds 75%) assuming Hydrologic Soil Group C soils?
a. 74 b. 75 c. 76 **d. 77 (correct)** e. 78

2.8. What is the 10 –yr peak discharge (cfs) along a stream where the watershed area is 88 sq. mi and the 10 –yr discharge is 550 cfs measured at the stream gage that drains 52 sq mi?
a. 931 (correct) b. 325 c. 8.3 d. 650 e. 1931

3.0 Water Treatment

3.1. Which of the following is not a step in a water treatment process?
a. Aerobic Digester (correct) b. Sedimentation c. Screening d. Filtration e. Disinfection

3.2. What is the peak water demand (mgd) for a city population of 5,000?
a. 5.0 b. 6.5 **c. 0.75 (correct)** d. 0.3 e. 0.65

3.3 A house at 1,100 ft above mean sea level has a water tank at 1,200 ft msl? What is the water pressure (psi) at the connection to the house?
a. 6250 b. 12,500 c. 86.6 **d. 43.4 (correct)** e. 62.5

3.4 What is the hydraulic loading rate (gpd/sf) for a filter where the flow is 100 gpm and the diameter of the filter is 24 in?
a. 46,000 (correct) b. 31.8 c. 7.9 11,459 e. 4.2

3.5 Calculate the detention time (hr) of a flocculation tank (20 ft diameter by 10 ft deep where the flow rate is 100 gpm.
a. **3.9 (correct)** b. 3.1 c. 125 d. 7.8 e. 31.4

3.6 Calculate the volume (cf) of a rapid mixing tank with flow rate 5 cfs, a mixing time of 10 sec and power requirement 200 ft-lb/sec.

a. 290 (correct) b. 580 c. 240 d. 480 e. 190

3.7 What is the volume (cf) of a sedimentation tank needed for a flow rate of 1 mgd assuming sand particle diameter of 0.1?

a. 25,000 (correct) b. 50,000 c. 75,000 d. 2,500 e. 5,000

3.8 What is the loading rate (gpm/sf) for a rapid sand filter?

a. 1 to 2 **b. 2 to 3 (correct)** c. 4 to 6 d. 7 to 8 e. 9 to 10

3.9 How many pounds of calcium hypochlorite are needed to treat water given the flow is 1 mgd assuming the hypochlorite ion dose is 25 mg/l and the purity of hypochlorite is 98%.

a. 300 (correct) b. 400 c. 500 d. 3000 e. 5000

3.10 What is the normal minimum working water pressure (psi) in a water supply distribution system ?

a. 20 b. 25 **c. 35 (correct)** d. 45 e. 40

CEWR Solutions to multiple choice problems G. J. Kauffman Dec 23, 2006

1.0 Hydraulics

1.1. **What is the flow (cfs) through a 24 in diameter pipe where the velocity is measured at 5 fps?**

Using the continuity equation:

$$Q = v\,A$$

Where:

$$v = 5 \text{ fps}$$

$$A = \pi (r)^2 = \pi \frac{(d)^2}{4} = \pi \frac{(24 \text{ in}/12 \text{ in/ft})^2}{4} = \pi (1 \text{ sf})^2 = 3.14 \text{ sf}$$

$$Q = 5 \text{ fps } (3.14 \text{ sf})$$

$$Q = 15.7 \text{ cfs}$$

_1.2._ If the headloss is 0.5 ft, what is the flow rate (gpm) through a 2 - ft diameter, 2000 feet long, new ductile iron pipe?

Using the Hazen – Williams formula:

$$h_L = \frac{L\,Q^{1.85}}{17{,}076\,(C)^{1.85}(D)^{4.87}}$$

Where:

h_L = 0.5 ft

L = 2,000 ft

C = 130 (from Table 1.1, new ductile iron pipe)

D = 2 ft

Q = ?

Solving for flow (Q) by the Hazen - Williams formula:

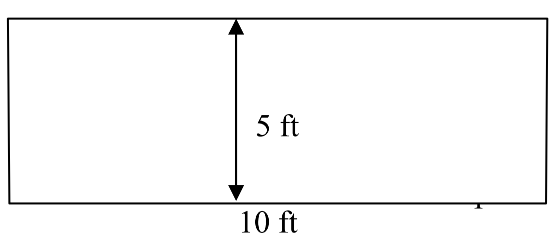
$$0.5\text{ ft} = \frac{2{,}000\text{ ft}\,(Q)^{1.85}}{17{,}076\,(130)^{1.85}(2\text{ ft})^{4.87}}$$

$$(Q)^{1.85} = \frac{(0.5\text{ ft})(17{,}076)(8{,}143)(29)}{2000\text{ ft}}$$

$$(Q)^{1.85} = 1{,}000{,}000$$

$$Q = 1{,}738\text{ gpm}$$

_1.3._ What is the velocity (fps) through a 10 feet wide concrete rectangular channel flowing at 5 feet deep with a slope of 2%?



Using Manning's equation:

$$v = \frac{1.49}{n} R^{2/3} S^{1/2}$$

n = 0.013 (concrete, from Table 1.4)

A = 10 ft (5 ft) = 50 sf

WP = 10 ft + 5 ft + 5 ft = 20 ft

R = A/WP = 50 sf/20 ft = 2.5 ft

S = 0.02 ft/ft

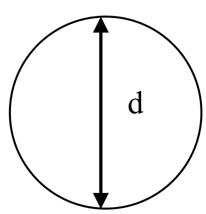
Then the velocity is:

$$v = \frac{1.49}{0.013}(2.5\text{ ft})^{2/3}(0.02\text{ ft/ft})^{1/2}$$

v = 114.61(1.85)(0.14)

v = 29.7 cfs

1.4. **What diameter concrete pipe (in) can convey 200 cfs when full at a slope of 0.02 ft/ft?**



Using Manning's equation:

$$Q = \frac{1.49}{n} R^{2/3} S^{1/2} (A)$$

Where:

n = 0.013 (concrete pipe, from Table 1.4)

$A = \pi (r)^2 = \pi (d)^2/4$

d = ? in

$$WP = 2\,\pi\, r = \frac{2\,\pi\, d}{2} = \pi\, d$$

$$R = A/WP = \frac{\pi\,(d)^2/4}{\pi\, d} = d/4$$

$$S = 0.02 \text{ ft/ft}$$

Using Manning's equation:

$$Q = \frac{1.49}{n} R^{2/3} S^{1/2} (A)$$

Try a 48 in or 4 ft diameter pipe:

$$Q = \frac{1.49}{0.013}(4 \text{ ft}/4)^{2/3}(0.02 \text{ ft/ft})^{1/2}(\pi\,(4 \text{ ft})^2/4)$$

$$Q = 115(1)(0.14)(12.56)$$

$Q = 202$ cfs ≈ 200 cfs (OK) A 48 in diameter pipe will convey 200 cfs when full.

***1.5.* If the critical depth is 1.1 ft through a 10 ft wide concrete channel, what is the discharge (cfs)?**

Using the critical depth equation for rectangular channels:

$$y_c = \frac{(q^2)^{1/3}}{(g)^{1/3}}$$

Where:

y_c = 1.1 ft

q = specific discharge in a rectangular channel (cfs/ft) = Q/b = Q/10 ft

Q = discharge (cfs)

b = 10 ft

g = acceleration due to gravity = 32.2 ft/sec^2.

$$y_c = \frac{(q^2)^{1/3}}{(g)^{1/3}}$$

$$1.1\ ft = \frac{(Q^2/10^2\ ft)^{1/3}}{(32.2\ ft/sec^2)^{1/3}}$$

$3.5 = (Q^2/100)^{1/3}$

$42.9 = Q^2/100$

$Q^2 = 4{,}290$

$Q = 65$ cfs

1.6. **What is the flow (cfs) through a 10 ft long rectangular weir at depth of 1 feet?**

Using the rectangular weir flow formula:

$Q = CLH^{3/2}$

$Q = 3.5(10\ ft)(1\ ft)^{3/2}$

$Q = 35$ cfs

1.7. **Estimate the flow capacity of a 12 ft wide by 5 ft rise concrete box culvert with beveled edges under inlet control and headwater depth = 6 ft and tailwater depth = 4 ft.**

Use the inlet control culvert equation.

$Q = C_dA(2gh)^{1/2}$

Where:

C_d = 1.0 for rounded edge entrances

A = 12 ft x 5 ft = 60 sf

h = 6 ft – 4 ft = 2 ft

g = acceleration due to gravity = 32.2 ft/sec^2

Then the flow capacity of the culvert is

$Q = 1.0(60\ sf)[2(32.2\ ft/sec^2)(2\ ft)]^{1/2}$

$= 60\ sf\ (11.3\ fps)$

$Q = 678$ cfs

1.8. **What is the maximum allowable velocity (fps) for gravel in an open channel?**

From Table 1.13 Maximum allowable velocities in open channels, the maximum velocity for a gravel – lined channel is 6.0 fps.

2.0 Hydrology

2.1 **If the annual precipitation is 40 in, runoff is 10 in, and evapotranspiration is 22 in, what is the infiltration (in)?**

Using the water budget equation, solve for runoff (R):

P = R + I + ET – Δ S

40 in = R + 10 in + 22 in – 0

R = 40 – 10 – 22

R = 8 in

2.2 **What is the 100 yr, 24 hr precipitation depth for Atlanta, Georgia?**

According to Figure 2.1, typical precipitation depth and duration curves from the TR – 55 manual, the 100 yr, 24 hr precipitation depth for Atlanta is 8 in.

a. 6 in b. 7 in **c. 8 in (correct)** d. 10 in e. 9 in

***2.3* What is the maximum incremental intensity (in/hr) of a storm based on the following precipitation data?**

Hour of Storm	Incremental Precip. Depth (in)
1.0	0.0
0.3	0.1
0.4	0.2
0.6	0.5
0.8	0.2
1.0	0.1

The maximum incremental intensity occurs between hours 0.4 and 0.6:

$$i = \frac{(0.5\text{ in} - 0.2\text{ in})}{(0.6\text{ hr} - 0.4\text{ hr})}$$

$$= \frac{0.3\text{ in}}{0.2\text{ hr}}$$

i = 1.5 in/hr is the maximum rainfall intensity.

***2.4* What is the probability of a 50 – yr storm occurring once in any given year?**

The probability is related to the recurrence interval by:

$$p = 1/T$$

$$= 1/50\text{ yr}$$

$$p = 0.02 = 2\ \%$$

***2.5* What is the 100-yr flow (cfs) from a 200 acre watershed in a downtown where the time of concentration is 0.5 hr min?**

Using the Rational method,

$$Q_{100} = c\ i\ A$$

Determine runoff coefficient (c):

Refer to Table 2.7, for downtown area land use,

c = 0.8.

Determine rainfall intensity (i):

T_c = 0.5 hr = 30 min and for a 100 yr storm:

i = 5.0 in/hr from Rainfall intensity – duration – frequency (IDF) curve Figure 2.6.

A = 200 ac

Q_{100} = c I A

= 0.8 (5.0 in/hr) (200 acres)

Q_{100} = 800 cfs

2.6 **In Delaware, what is the time of concentration (hr) for the 300 ft long sheet flow component over dense grass where the slope is 0.07 ft/ft?**

Given for sheet flow:

s = 0.07 ft/ft

n = 0.24, dense grass (Table 2.?? In TR – 55 manual)

L = 300 ft.

P_2 = 3.5 in (for Delaware, from Table 2.??, TR – 55 manual)

Determine time of concentration (T_c) for sheet flow:

$$T_{sf} = \frac{0.007\,(nL)^{0.8}}{(P_2)^{0.5}(s)^{0.4}}$$

$$T_{sf} = \frac{0.007(0.24(300\text{ ft}))^{0.8}}{(3.5\text{ in})^{0.5}\,(0.07\text{ ft/ft})^{0.4}}$$

T_{sf} = 0.35 hr.

2.7 **What is the runoff curve number for a 1000 acre watershed that is 50% single family residential (½ acre lots) and 50% open space (grass cover exceeds 75%) assuming Hydrologic Soil Group C soils?**

Using Worksheet 2 and selecting CN from Table 2-2a of the TR – 55 manual, compute the composite CN:

Soil Name/HSG	**Land Cover**	**CN (Table 2-2)**	**Area (ac)**	**Product (CN X Area)**
C	SF Residential 1/2 ac	80	500	40,000
C	Open space, good condition	74	500	37,000
		Total	***1000 ac***	***77,000***

Composite CN = 77,000 /1000 ac = 77.

2.8 **What is the 10 –yr peak discharge (cfs) along a stream where the watershed area is 88 sq. mi and the 10 –yr discharge is 550 cfs measured at the stream gage that drains 52 sq mi?**

Using the ratio of drainage areas equation:

$$\frac{Q_s}{A_s} = \frac{Q_g}{A_g}$$

The 10 year flow at the site is expressed as:

$$Q_s = \frac{Q_g(A_s)}{(A_g)}$$

$$= \frac{550 \text{ cfs}(88 \text{ sq mi})}{(52 \text{ sq mi})}$$

$$Q_s = 931 \text{ cfs}$$

3.0 Water Treatment

3.1. Which of the following is not a step in a water treatment process?

The aerobic digester is part of a wastewater treatment process, not a water treatment process.

a. Aerobic Digester b. Sedimentation c. Screening d. Filtration e. Disinfection

3.2. What is the peak water demand (mgd) for a city population of 5,000?

Use:

Population = 5,000

Per capita water use = 100 gpcd

Peaking factor = 1.5

Peak water demand = 5,000 people (100 gpcd)(1.5)

= 750,000 gpd = 0.75 mgd

3.3 What is the water pressure (psi) at the connection to a house at 1,100 ft above mean sea level when the level in the water tank is at 1,200 ft msl?

$p \quad = \gamma\ h$

$p \quad = 62.4$ pcf(1,200 ft – 1,100 ft)

$= 62.4$ pcf(100 ft)

$= 6{,}250$ psf/(144 in/sf)

$= 43.4$ psi

3.4 What is the hydraulic loading rate (gpd/sf) for a filter where the flow is 100 gpm and the diameter of the filter is 24 in?

The hydraulic loading rate equation is:

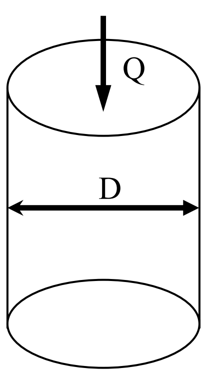
HLR $= Q/A$

Where:

HLR = hydraulic loading rate (gpd/sf)

Q = flow rate (gpd)

A = cross section surface area (sf)



Sand filter

Using the hydraulic loading rate formula:

HLR $= Q/A = Q/\pi\ (d/2)^2$

HLR $= 100$ gpm(60 min/hr)(24 hr/day)/ π (24 in/12 in/ft/2)^2

$= 144{,}000$ gpd /3.14 sf

$= 46{,}000$ gpd/sf

3.5 Calculate the detention time (hr) of a flocculation tank (20 ft diameter by 10 ft deep where the flow rate is 100 gpm.

The detention time is calculated as:

DT $= V/Q = A(H)/Q$

DT $= \pi\,(r)^2(H)/Q$

DT $= \pi\,(20/2)^2$ (10 ft)/100 gpm(1 cf /7.48 gal)(60 min/hr)

$= 3{,}140$ cf / 802 cf/hr

DT $= 3.9$ hr.

3.6 Calculate the volume (cf) of a rapid mixing tank with flow rate 10 cfs, a mixing time of 10 sec and power requirement 200 ft-lb/sec.

The rapid mixing equation is:

$$G(T_d) = \frac{1(PV)^{1/2}}{Q(\mu)^{1/2}}$$

Where:

G $= 1000\ \text{sec}^{-1}$ for a mixing time of 10 to 30 sec.

T_d = 10 sec

P = 200 ft-lb/sec

μ = 0.000023 lb-sec/sf.

V = rapid mixer volume (cf)

Q = 5 cfs

$$1000\ \text{sec}^{-1}\ (10\ \text{sec}) = \frac{1(200\ \text{ft-lb/sec})^{1/2}\ (V)^{1/2}}{5\ \text{cfs}(0.000023\ \text{lb-sec/sf})^{1/2}}$$

$$50{,}000 = \frac{14(V)^{1/2}}{0.0048}$$

$$(V)^{1/2} = 17.1$$

$$V = 290 \text{ cf}$$

3.7 What is the volume (cf) of a sedimentation tank needed for a flow rate of 1 mgd assuming sand particle diameter of 0.1 mm?

Use a surface overflow rate of 600 gal/sf-day.

The surface area of the tank is then:

A = 1,000,000 gal/day/600 gal/sf-day

= 1,667 sf

For a rectangular tank with length to width aspect ratio of 3:1:

A = length (l) x width (w)

l = 3w

Then:

A = 3w(w) = 1,667 sf

w^2 = 555 sf

w = 24 ft

And:

l = 3(w)

= 3(24 ft)

l = 72 ft

Select tank depth:

h = 15 ft

The tank volume is then:

$V = (15 \text{ ft}) (1{,}667 \text{ sf}) = 25{,}000 \text{ cf}$

Check detention time:

$T_d = 25{,}000 \text{ cf } (7.48 \text{ gal/cf})/1{,}000{,}000 \text{ gal/day}$

$= 0.19 \text{ day}(24 \text{ hr/day})$

$= 4.5$ hr within detention time of 1 to 10 hrs (OK)

3.8 What is the loading rate (gpm/sf) for a rapid sand filter?

The loading rate is 2 to 3 gpm/sf according to Table 3.3, Filter Types and Characteristics.

3.9 How many pounds of calcium hypochlorite are needed to treat water given the flow is 1 mgd assuming the hypochlorite ion dose is 25 mg/l and the purity of hypochlorite is 98%.

Calculate the molecular weight of calcium hypochlorite $Ca(OCl)_2$:

Calcium	=	40.1
Oxygen 2(16)	=	32.0
Chloride 2(35.5)	=	70.0
Total	=	142.1

The available chlorine (G) from hypochlorite ion is:

$$G = \frac{32 + 70}{142.1} = 0.718$$

$Q = 1$ mgd

$D = 25$ mg/l

$Py = 0.98$

The weight (F) of calcium hypochlorite needed to treat the water by the dosage equation is then:

$$F = \frac{D(Q)(8.34)}{Py(Cl)}$$

$$F = \frac{25 \text{ mg/l } (1 \text{ mgd}) (8.345)}{0.98 (0.718)}$$

$$F = 300 \text{ lbm/day}$$

3.10 What is the normal minimum working water pressure (psi) in a water supply distribution system ?

According to the 10 – State standards, the normal working pressure in the distribution system should be not less than 35 psi.