SECTION 7

WATER-SUPPLY AND STORM-WATER SYSTEM DESIGN

WATER-WELL ANALYSIS 7.1
Determining the Drawdown for Gravity Water-Supply Well 7.1
Finding the Drawdown of a Discharging Gravity Well 7.4
Analyzing Drawdown and Recovery for Well Pumped for Extended Period 7.6
Selection of Air-Lift Pump for Water Well 7.9
WATER-SUPPLY AND STORM-WATER SYSTEM DESIGN 7.11
Water-Supply System Flow-Rate and Pressure-Loss Analysis 7.11
Water-Supply System Selection 7.17
Selection of Treatment Method for Water-Supply System 7.21
Storm-Water Runoff Rate and Rainfall Intensity 7.24
Sizing Sewer Pipe for Various Flow Rates 7.25
Sewer-Pipe Earth Load and Bedding Requirements 7.29
Storm-Sewer Inlet Size and Flow Rate 7.33
Storm-Sewer Design 7.34

Water-Well Analysis

DETERMINING THE DRAWDOWN FOR GRAVITY WATER-SUPPLY WELL

Determine the depth of water in a 24-in. (61-cm) gravity well, 300 ft (91-m) deep, without stopping the pumps, while the well is discharging 400 gal/min (25.2 L/s). Tests show that the drawdown in a test borehole 80 ft (24.4 m) away is 4 ft (1.2 m), and in a test borehole 20 ft (6.1 m) away, it is 18 ft (5.5 m). The distance to the static groundwater table is 54 ft (16.5 m).

Calculation Procedure:

1. *Determine the key parameters of the well*

Figure 1 shows a typical gravity well and the parameters associated with it. The Dupuit formula, given in step 2, below, is frequently used in analyzing gravity wells. Thus, from the given data, $Q = 400$ gal/min (25.2 L/s); $h_e = 300 - 54 = 246$ ft (74.9 m); $r_w = 1$ ft (0.3 m) for the well, and 20 and 80 ft (6.1 and 24.4 m), respectively, for the boreholes. For this well, h_w is unknown; in the nearest borehole it is $246 - 18 = 228$ ft (69.5 m); for the farthest borehole it is $246 - 4 = 242$ ft (73.8 m). Thus, the parameters have been assembled.

2. *Solve the Dupuit formula for the well*

Substituting in the Dupuit formula

$$Q = K\frac{h_e^2 - h_w^2}{\log_{10}(r_e/r_w)} = K\frac{(h_e - h_w)(h_e + h_w)}{\log_{10}(r_e/r_w)}$$

we have,

$$300 = K\frac{(246 + 228)(246 - 228)}{\log_{10}(r_e/20)} = K\frac{(246 + 242)(246 - 242)}{\log_{10}(r_e/80)}$$

Solving, $r_e = 120$ and $K = 0.027$. Then, for the well,

$$300 = 0.027\frac{(246 + h_w)(246 - h_w)}{\log_{10}(120/1)}$$

Solving h_w, = 195 ft (59.4 m). The drawdown in the well is $246 - 195 = 51$ ft (15.5 m).

Related Calculations. The graph resulting from plotting the Dupuit formula produces the "base-pressure curve," line ABCD in Fig. 1. It has been found in practice that the approximation in using the Dupuit formula gives results of practical value. The

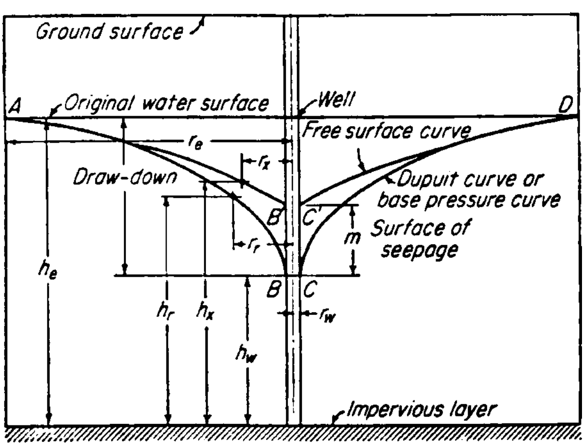


FIGURE 1. Hypothetical conditions of underground flow into a gravity well. (*Babbitt, Doland, and Cleasby.*)

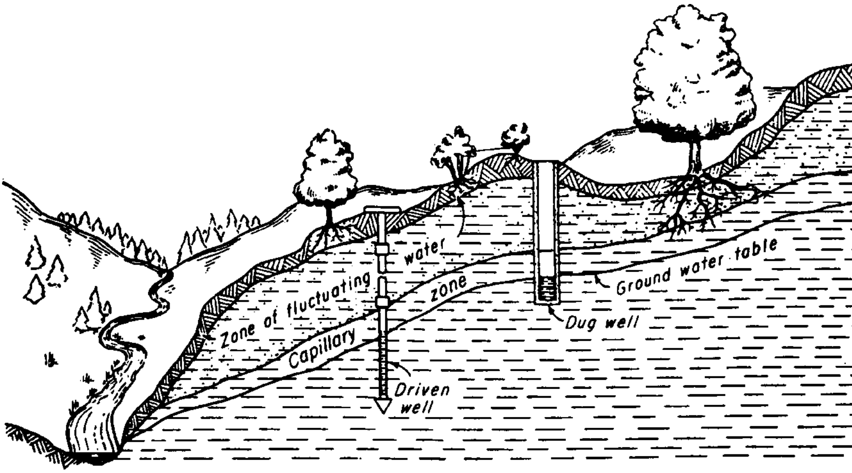


FIGURE 2. Relation between groundwater table and ground surface. (*Babbitt, Doland, and Cleasby.*)

results obtained are most nearly correct when the ratio of drawdown to the depth of water in the well, when not pumping, is low.

Figure 1 is valuable in analyzing both the main gravity well and its associated boreholes. Since gravity wells are, Fig. 2, popular sources of water supply throughout the world, an ability to analyze their flow is an important design skill. Thus, the effect of the percentage of total possible drawdown on the percentage of total possible flow from a well, Fig. 3, is an important design concept which finds wide use in industry today. Gravity wells are highly suitable for supplying typical weekly water demands, Fig. 4, of a moderate-size city. They are also suitable for most industrial plants having modest process-water demand.

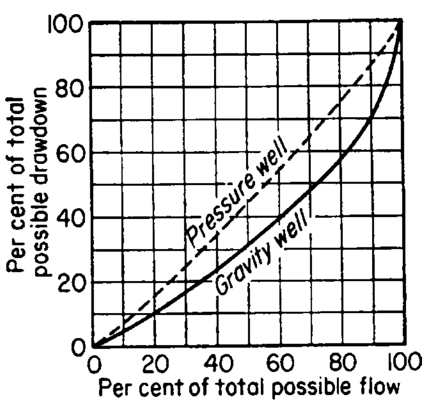


FIGURE 3. The effect of the percentage of total possible drawdown on the percentage of total possible flow from a well. (*Babbitt, Doland, and Cleasby.*)

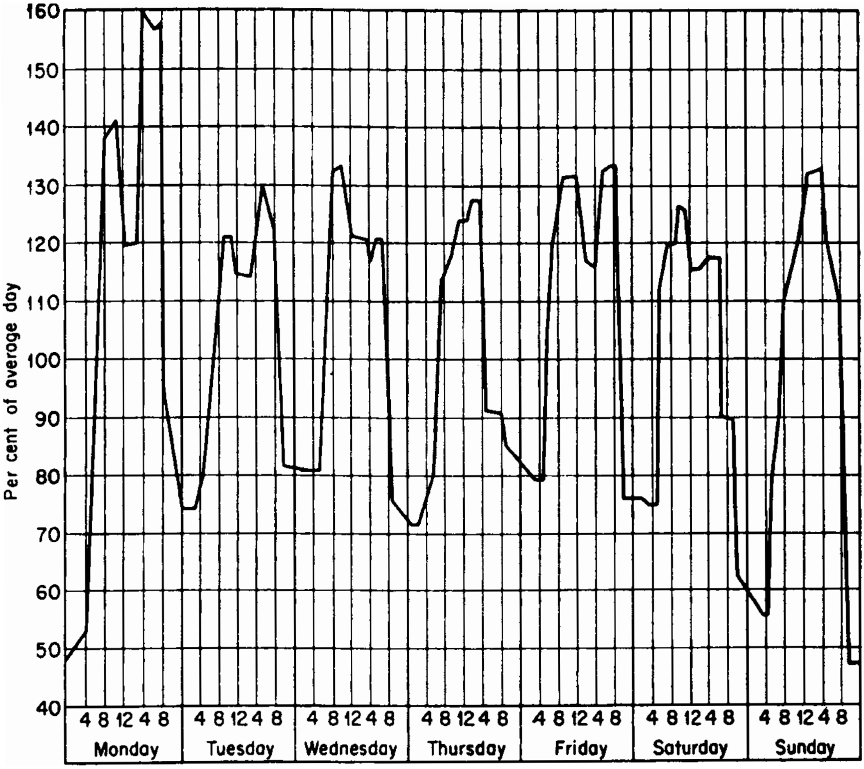


FIGURE 4. Demand curve for a typical week for a city of 100,000 population. (*Babbitt, Doland, and Cleasby*.)

This procedure is the work of Harold E. Babbitt, James J. Doland, and John L. Cleasby, as reported in their book, *Water Supply Engineering,* McGraw-Hill.

FINDING THE DRAWDOWN OF A DISCHARGING GRAVITY WELL

A gravity well 12 in. (30.5 cm) in diameter is discharging 150 gal/min (9.5 L/s), with a drawdown of 10 ft (3 m). It discharges 500 gal/min (31.6 L/s) with a drawdown of 50 ft (15 m). The static depth of the water in the well is 150 ft (45.7 m). What will be the discharge from the well with a drawdown of 20 ft (6 m)?

Calculation Procedure:

1. ***Apply the Dupuit formula to this well***

Using the formula as given in the previous calculation procedure, we see that

$$150 = K\frac{(10)(290)}{\log_{10}(150C/0.5)} \qquad \text{and} \qquad 500 = K\frac{(50)(250)}{\log_{10}(500C/0.5)}$$

Solving for C and K we have

$$C = 0.21 \qquad \text{and} \qquad K = \frac{(500)(\log 210)}{12{,}500} = 0.093;$$

then

$$Q = 0.093\frac{(20)(280)}{\log_{10}(0.210Q/0.5)}$$

2. *Solve for the water flow by trial*

Solving by successive trial using the results in step 1, we find $Q = 257$ gal/min (16.2 L/s).

Related Calculations. If it is assumed, for purposes of convenience in computations, that the radius of the circle of influence, r_e, varies directly as Q for equilibrium conditions, then $r_e = CQ$. Then the Dupuit equation can be rewritten as

$$Q = K\frac{(h_e + h_w)(h_e - h_w)}{\log_{10}(CQ/r_w)}$$

From this rewritten equation it can be seen that where the drawdown $(h_e - h_w)$ is small compared with $(h_e + h_w)$ the value of Q varies approximately as $(h_e - h_w)$. This straight-line relationship between the rate of flow and drawdown leads to the definition of the *specific capacity* of a well as the rate of flow per unit of drawdown, usually expressed in gallons per minute per foot of drawdown (liters per second per meter). Since the relationship is not the same for all drawdowns, it should be determined for one special foot (meter), often the first foot (meter) of drawdown. The relationship is shown graphically in Fig. 3 for both gravity, Fig. 1, and pressure wells, Fig. 5. Note also that since K in

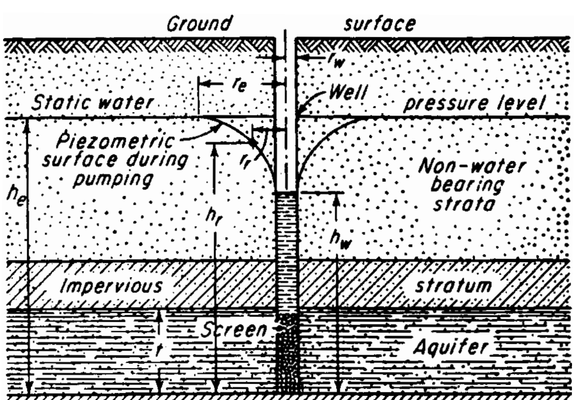


FIGURE 5. Hypothetical conditions for flow into a pressure well. (*Babbitt, Doland, and Cleasby.*)

different aquifers is not the same, the specific capacities of wells in different aquifers are not always comparable.

It is possible, with the use of the equation for Q above, to solve some problems in gravity wells by measuring two or more rates of flow and corresponding drawdowns in the well to be studied. Observations in nearby test holes or boreholes are unnecessary. The steps are outlined in this procedure.

This procedure is the work of Harold E. Babbitt, James J. Doland, and John L. Cleasby, as reported in their book, *Water Supply Engineering*, McGraw-Hill. SI values were added by the handbook editor.

ANALYZING DRAWDOWN AND RECOVERY FOR WELL PUMPED FOR EXTENDED PERIOD

Construct the drawdown-recovery curve for a gravity well pumped for two days at 450 gal/min (28.4 L/s). The following observations have been made during a test of the well under equilibrium conditions: diameter, 2 ft (0.61 m); $h_e = 50$ ft (15.2 m); when $Q = 450$ gal/min (28.4 L/s), drawdown = 8.5 ft (2.6 m); and when $r_x = 60$ ft (18.3 m), $(h_e - h_x) =$ 3 ft (0.91 m). The specific yield of the well is 0.25.

Calculation Procedure:

1. *Determine the value of the constant k*

Use the equation

$$Q = \frac{k(h_e - h_x)h_e}{C_x \log_{10}(r_e/0.1h_e)} \qquad \text{and} \qquad k = \frac{QC_x \log_{10}(r_e/0.1h_e)}{(h_e - h_x)(h_e)}$$

Determine the value of C_x when r_w, is equal to the radius of the well, in this case 1.0. The value of k can be determined by trial. Further, the same value of k must be given when $r_x = r_e$ as when $r_x = 60$ ft (18.3 m). In this procedure, only the correct assumed value of r_e is shown—to save space.

Assume that $r_e = 350$ ft (106.7 m). Then, 1/350 = 0.00286 and, from Fig. 6, $C_x = 0.60$. Then $k = (1)(0.60)(\log 350/5)/(8)(50) = (1)(0.6)(1.843)/400 = 0.00276$, $r_x/r_e = 60/350 = 0.172$, and $C_x = 0.225$. Hence, checking the computed value of k, we have $k = (1)(0.22)(1.843)/150 = 0.0027$, which checks with the earlier computed value.

2. *Compute the head values using k from step 1*

Compute $h_e - (h_e^2 - 1.7\,Q/k)^{0.5} = 50 - (2500 - 1.7/0.0027)^{0.5} = 6.8$.

3. *Find the values of T to develop the assumed values of r_e*

For example, assume that $r_e = 100$. Then $T = (0.184)(100)^2(0.25)(6.8)/1 = 3230$ sec = 0.9 h, using the equation

$$T = \left(h_e - \sqrt{h_e^2 - 1.7\frac{Q}{k}}\right)\frac{0.184 r_e^2 f}{Q}$$

4. *Calculate the radii ratio and d_0*

These computations are: $r_e/r_w = 100/1 = 100$. Then, $d_0 = (6.8)(\log_{10} 100)/2.3 = 5.9$ ft (1.8 m), using the equation

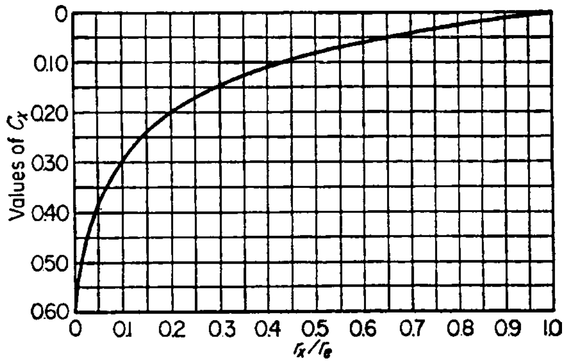


FIGURE 6. Values of C_x for use in calculations of well performance. (*Babbitt, Doland, and Cleasby.*)

$$d_0 = \frac{1}{2.3}\left(h_e - \sqrt{h_e^2 - 1.7\frac{Q}{k}}\right)\log_{10}\frac{r_o}{r_w}$$

5. *Compute other points on the drawdown curve*

Plot the values found in step 4 on the drawdown-recovery curve, Fig. 7. Compute additional values of d_0 and T and plot them on Fig. 7, as shown.

6. *Make the recovery-curve computations*

The recovery-curve, Fig. 7, computations are based the assumption that by imposing a negative discharge on the positive discharge from the well there will be in effect zero flow from the well, provided the negative discharge equals the positive discharge. Then, the sum of the drawdowns due to the two discharges at any time T after adding the negative discharge will be the drawdown to the recovery curve, Fig. 7.

Assume some time after the pump has stopped, such as 6 h, and compute r_e, with $Q, f, k,$ and h_e as in step 3, above. Then $r_e = [(6 \times 3600 \times 1)/(0.184 \times 0.25 \times 6.8)]^{0.5} = 263$ ft (80.2 m). Then, $r_e/r_w = 263$; check.

7. *Find the value of d_0 corresponding to r_e in step 6*

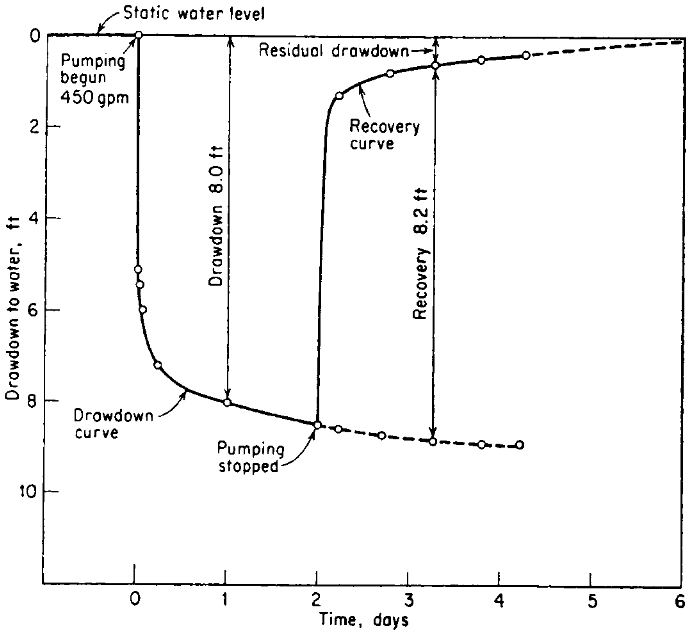
Computing, we have $d_0 = (6.8)(\log_{10})/2.3 = 7.15$ ft (2.2 m). Tabulate the computed values as shown in Table 1 where the value 7.15 is rounded off to 7.2.

Compute the value of r_e using the total time since pumping started. In this case it is $48 + 6 = 54$ h. Then $r_e = [(54 \times 3600 \times 1)/(0.184 \times 0.25 \times 6.8)]^{0.5} = 790$ ft (240.8 m). The d_0 corresponding to the preceding value of $r_e = 790$ ft (240.8 m) is $d_0 = (6.8)(\log_{10} 790)/2.3 = 8.55$ ft (2.6 m).

8. *Find the recovery value*

The recovery value, $d_r = 8.55 - 7.15 = 1.4$ ft (0.43 m). Coordinates of other points on the recovery curve are computed in a similar fashion. Note that the recovery curve does not attain the original groundwater table because water has been removed from the aquifer and it has not been restored.

Related Calculations. If water is entering the area of a well at a rate q and is being pumped out at the rate Q' with Q' greater than q, then the value of Q to be used in computing the drawdown recovery is $Q' - q$. If this difference is of appreciable magnitude, a correction must be made because of the effect of the inflow from the aquifer into



SI Values			
ft	m		
2	0.6		
4	1.2		
6	1.8		
8	2.4	8.2 ft	2.5 m
10	3.0		

gpm	L/s
450	28.4

FIGURE 7. Drawdown-recovery curves for a gravity well. (*Babbitt, Doland, and Cleasby.*)

TABLE 1. Coordinates for the Drawdown-Recovery Curve of a Gravity Well

(1) Time after pump starts, hr	(2) $\frac{r_e}{r_x} = r'_e$	(3) $2.95 \times \log_{10} \frac{r_e}{r_w} = d_0$	(4) Time after pump starts, hr	(5) $\frac{r_e}{r_x} = r'_e$	(6) $2.95 \times \log_{10} \frac{r_e}{r_w} = d_0$	(7) Time after pump stops, hr	(8) $\frac{r_e}{r_x} = r'_e$	(9) $2.95 \times \log_{10} \frac{r_e}{r_w} = d_0$	(10) Col 6 minus col 9 = d_r
0.25	54	5.10	54	784	8.5	6	263	7.2	1.3
0.50	76	5.45	66	872	8.7	18	455	7.9	0.8
1.00	107	6.0	78	950	8.8	30	587	8.2	0.6
6	263	7.2	90	1,020	8.9	42	694	8.4	0.5
24	526	8.0	102	1,085	8.9	54	784	8.5	0.4
48	745	8.5							

Conditions: $r_w = 1.0$ ft; $h_e = 50$ ft. When $Q = 1$ ft^3/s and $r_x = 1.0$ ft $(h_e - h_x) = 8.0$ ft. When $Q = 1$ ft^3/s and $r_x = 60$ ft, $(h_e - h_x) = 3.0$ ft. Specific yield = 0.25; k, as determined in step 1 of example, = 0.0027; and $h_e - (h_e^2 - 1.79Q/k)^{0.5} = 6.8$.

the cone of depression so the groundwater table will ultimately be restored, the recovery curve becoming asymptotic to the table.

This procedure is the work of Harold E. Babbitt, James J. Doland, and John L. Cleasby, as reported in their book, *Water Supply Engineering*, McGraw-Hill. SI values were added by the handbook editor.

SELECTION OF AIR-LIFT PUMP FOR WATER WELL

Select the overall features of an air-lift pump, Fig. 8, to lift 350 gal/min (22.1 L/s) into a reservoir at the ground surface. The distance to groundwater surface is 50 ft (15.2 m). It is expected that the specific gravity of the well is 14 gal/min/ft (2.89 L/s/m).

Calculation Procedure:

1. *Find the well drawdown, static lift, and depth of this well*

The drawdown at 350 gal/min is $d = 350/14 = 25$ ft (7.6 m). The static lift, h, is the sum of the distance from the groundwater surface plus the drawdown, or $h = 50 + 25 = 75$ ft (22.9 m).

Interpolating in Table 2 gives a submergence percentage of $s = 0.61$. Then, the depth of the well, D ft is related to the submergence percentage thus: $s = D/(D + h)$. Or, $0.61 = D/(D + 75)$; $D = 117$ ft (35.8 m). The depth of the well is, therefore, $75 + 117 = 192$ ft (58.5 m).

2. *Determine the required capacity of the air compressor*

The rate of water flow in cubic feet per second, Q_w is given by Q_w = gal/min/(60 min/s)(7.5 ft^3/gal) = 350/(60)(7.5) = 0.78 ft^3/s (0.022 m^3/s). Then the volume of free air required by the air-lift pump is given by

$$Q_a = \frac{Q_w(h + h_1)}{75E \log r}$$

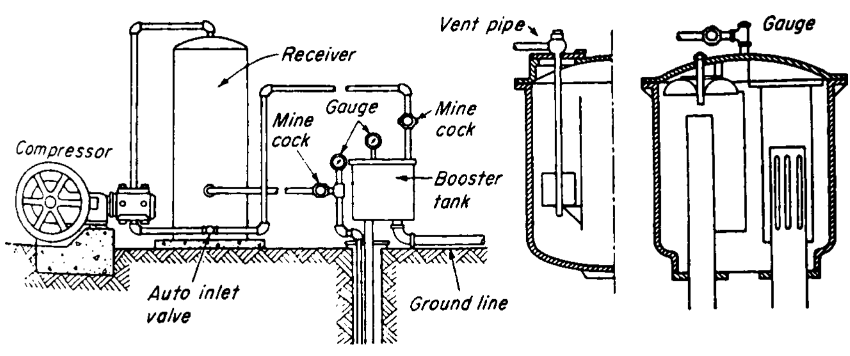


FIGURE 8. Sullivan air-lift booster. (*Babbitt, Doland, and Cleasby.*)

TABLE 2. Some Recommended Submergence Percentages for Air Lifts

Lift, ft	Up to 50	50–100	100–200	200–300	300–400	400–500
Lift, m	Up to 15	15–30	30–61	61–91	91–122	122–152
Submergence percentage	70–66	66–55	55–50	50–43	43–40	40–33

where Q_a = volume of free air required, ft^3/min (m^3/min); h_1 = velocity head at discharge, usually taken as 6 ft (1.8 m) for deep wells, down to 1 ft (0.3 m) for shallow wells; E = efficiency of pump, approximated from Table 3; r = ratio of compression = $(D + 34)/34$. Substituting, using 6 ft (1.8 m) since this is a deep well, we have, $Q_a = (0.779 \times 81)/(75 \times 0.35 \times 0.646) = 3.72$ ft^3/s (0.11 m^3/s).

3. *Size the air pipe and determine the operating pressures*

The cross-sectional area of the pipe = Q'_a/V. At the bottom of the well, $Q'_a = 3.72\,(34/151) = 0.83$ ft^3/s (0.023 m^3/s). With a flow velocity of the air typically at 2000 ft/min (610 m/min), or 33.3 ft/s (10 m/s), the area of the air pipe is 0.83/33.3 = 0.025 sq.ft., and the diameter is $[(0.025 \times 4)/\pi]^{0.5} = 0.178$ ft or 2.1 in. (53.3 mm); use 2-in. (50.8 mm) pipe.

The pressure at the start is 142 ft (43 m); operating pressure is 117 ft (35.7 m).

4. *Size the eductor pipe*

At the well bottom, $A = Q/V$. $Q = Q_w + Q'_a = 0.78 + 0.83 = 1.612$ ft^3/s (0.45 m^3/s). The velocity at the entrance to the eductor pipe is 4.9 ft/s (1.9 m/s) from a table of eductor entrance velocities, available from air-lift pump manufacturers. Then, the pipe area, $A = Q/V = 1.61/4.9 = 0.33$. Hence, $d = [(4 \times 0.33)/\pi)]05 - 0.646$ ft, or 7.9 in. Use 8-in. (203 mm) pipe.

If the eductor pipe is the same size from top to bottom, then V at top $= (Q_a + Q_w)/A = (3.72 + 0.78)(4)/(\pi \times 0.667^2) = 13$ ft/s (3.96 m/s). This is comfortably within the permissible maximum limit of 20 ft/s (6.1 m/s). Hence, 8-in. pipe is suitable for this eductor pipe.

Related Calculations. In an air-lift pump serving a water well, compressed air is released through an air diffuser (also called a foot piece) at the bottom of the eductor pipe. Rising as small bubbles, a mixture of air and water is created that has a lower specific gravity than that of water alone. The rising air bubbles, if sufficiently large, create an upward water flow in the well, to deliver liquid at the ground level.

Air lifts have many unique features not possessed by other types of well pumps. They are the simplest and the most foolproof type of pump. In operation, the airlift pump gives the least trouble because there are no remote or submerged moving parts. Air lifts can be operated successfully in holes of any practicable size. They can be used in crooked holes

TABLE 3. Effect of Submergence on Efficiencies of Air Lift*

Ratio D/h	8.70	5.46	3.86	2.91	2.25
Submergence ratio, $D/(D + h)$	0.896	0.845	0.795	0.745	0.693
Percentage efficiency	26.5	31.0	35.0	36.6	37.7
Ratio D/h		1.86	1.45	1.19	0.96
Submergence ratio, $D/(D + (h)$		0.650	0.592	0.544	0.490
Percentage efficiency		36.8	34.5	31.0	26.5

*At Hattiesburg MS.

not suited to any other type of pump. An air-lift pump can draw more water from a well, with sufficient capacity to deliver it, than any other type of pump that can be installed in a well. A number of wells in a group can be operated from a central control station where the air compressor is located.

The principal disadvantages of air lifts are the necessity for making the well deeper than is required for other types of well pumps, the intermittent nature of the flow from the well, and the relatively low efficiencies obtained. Little is known of the efficiency of the average air-lift installation in small waterworks. Tests show efficiencies in the neighborhood of 45 percent for depths of 50 ft (15 m) down to 20 percent for depths of 600 ft (183 m). Changes in efficiencies resulting from different submergence ratios are shown in Table 3. Some submergence percentages recommended for various lifts are shown in Table 2.

This procedure is the work of Harold E. Babbitt, James J. Doland, and John L. Cleasby, as reported in their book, *Water Supply Engineering,* McGraw-Hill. SI values were added by the handbook editor.

Water-Supply and Storm-Water System Design

WATER-SUPPLY SYSTEM FLOW-RATE AND PRESSURE-LOSS ANALYSIS

A water-supply system will serve a city of 100,000 population. Two water mains arranged in a parallel configuration (Fig. 9*a*) will supply this city. Determine the flow rate, size, and head loss of each pipe in this system. If the configuration in Fig. 9*a* were replaced by the single pipe shown in Fig. 9*b*, what would the total head loss be if

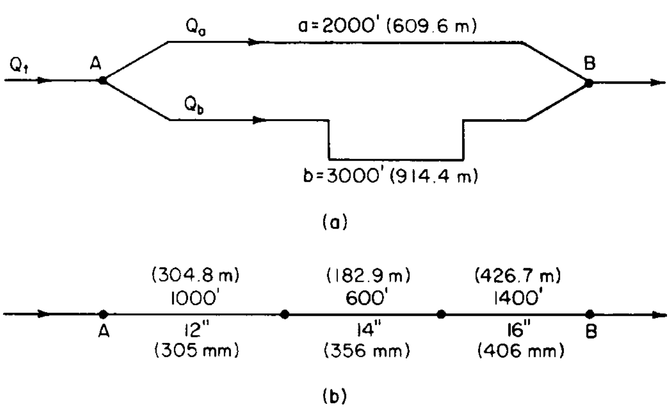


FIGURE 9. (*a*) Parallel water distribution system; (*b*) single-pipe distribution system.

$C = 100$ and the flow rate were reduced to 2000 gal/min (126.2 L/s)? Explain how the Hardy Cross method is applied to the water-supply piping system in Fig. 11.

Calculation Procedure:

1. *Compute the domestic water flow rate in the system*

Use an average annual domestic water consumption of 150 gal/day (0.0066 L/s) per capita. Hence, domestic water consumption = (150 gal per capita per day)(100,000 persons) = 15,000,000 gal/day (657.1 L/s). To this domestic flow, the flow required for fire protection must be added to determine the total flow required.

2. *Compute the required flow rate for fire protection*

Use the relation $Q_f = 1020(P)^{0.5}\,[1 - 0.01(P)^{0.5}]$, where Q_f = fire flow, gal/min; P = population in thousands. Substituting gives $Q_f = 1020(100)^{0.5}\,[1 - 0.01(100)^{0.5}] = 9180$, say 9200 gal/min (580.3 L/s).

3. *Apply a load factor to the domestic consumption*

To provide for unusual water demands, many design engineers apply a 200 to 250 percent load factor to the average hourly consumption that is determined from the average annual consumption. Thus, the average daily total consumption determined in step 1 is based on an average annual daily demand. Convert the average daily total consumption in step 1 to an average hourly consumption by dividing by 24 h or 15,000,000/24 = 625,000 gal/h (657.1 L/s). Next, apply a 200 percent load factor. Or, design hourly demand = 2.00(625,000) = 1,250,000 gal/h (1314.1 L/s), or 1,250,000/60 min/h = 20,850, say 20,900 gal/min (1318.6 L/s).

4. *Compute the total water flow required*

The total water flow required = domestic flow, gal/min + fire flow, gal/min = 20,900 + 9200 = 30,100 gal/min (1899.0 L/s). If this system were required to supply water to one or more industrial plants in addition to the domestic and fire flows, the quantity needed by the industrial plants would be added to the total flow computed above.

5. *Select the flow rate for each pipe*

The flow rate is not known for either pipe in Fig. 9*a*. Assume that the shorter pipe *a* has a flow rate Q_a of 12,100 gal/min (763.3 L/s), and the longer pipe *b* a flow rate Q_b of 18,000 gal/min (1135.6 L/s). Thus, $Q_a + Q_b = Q_t = 12{,}100 + 18{,}000 = 30{,}100$ gal/min (1899.0 L/s), where Q = flow, gal/min, in the pipe identified by the subscript *a* or *b*; Q_t = total flow in the system, gal/min.

6. *Select the sizes of the pipes in the system*

Since neither pipe size is known, some assumptions must be made about the system. First, assume that a friction-head loss of 10 ft of water per 1000 ft (3.0 m per 304.8 m) of pipe is suitable for this system. This is a typical allowable friction-head loss for water-supply systems.

Second, assume that the pipe is sized by using the Hazen-Williams equation with the coefficient $C = 100$. Most water-supply systems are designed with this equation and this value of C.

Enter Fig. 10 with the assumed friction-head loss of 10 ft/1000 ft (3.0 m/304.8 m) of pipe on the right-hand scale, and project through the assumed Hazen-Williams coefficient $C = 100$. Extend this straight line until it intersects the pivot axis. Next, enter Fig. 10 on the left-hand scale at the flow rate in pipe *a*, 12,100 gal/min (763.3 L/s), and project to the previously found intersection on the pivot axis. At the intersection with the pipe-diameter scale, read the required pipe size as 27-in. (686-mm) diameter.

Note that if the required pipe size falls between two plotted sizes, the next *larger* size is used.

Now in any parallel piping system, the friction-head loss through any branch connecting two common points equals the friction-head loss in any other branch connecting the same two points. Using Fig. 10 for a 27-in. (686-mm) pipe, find the actual friction-head loss at 8 ft/1000 ft (2.4 m/304.8 m) of pipe. Hence, the total friction-head loss in pipe *a* is

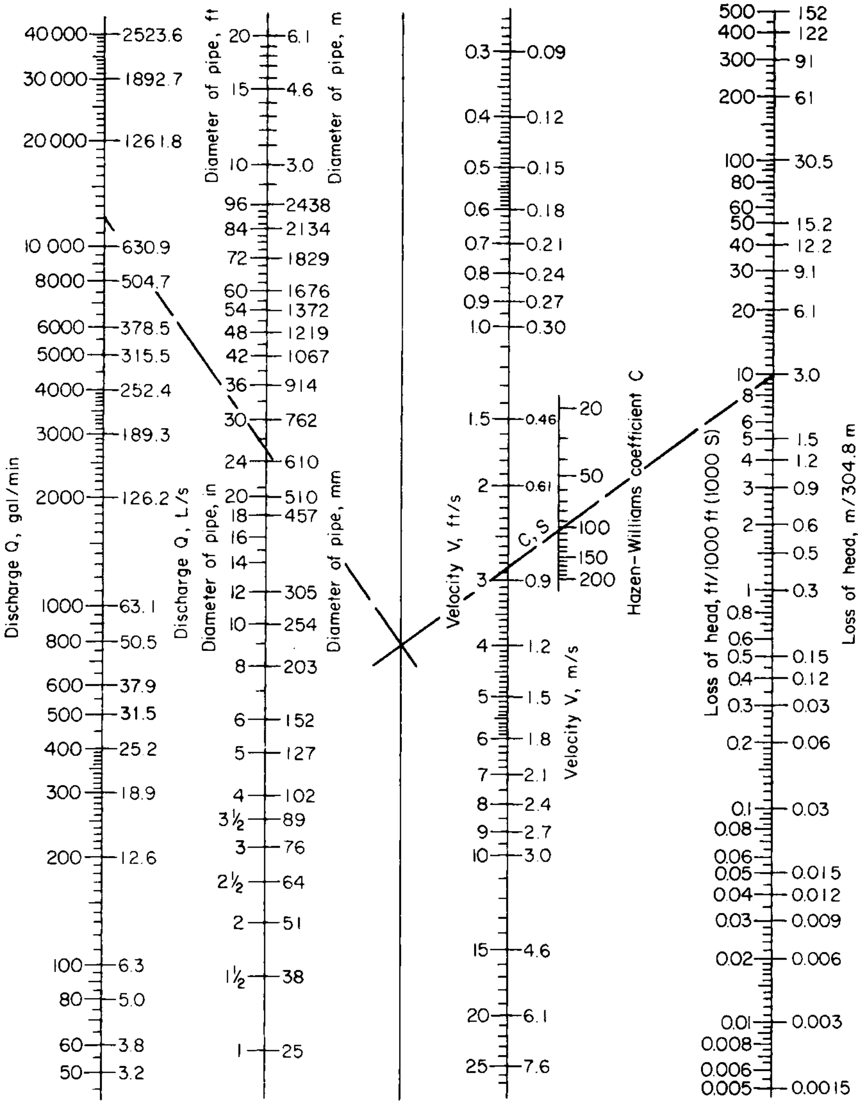


FIGURE 10. Nomogram for solution of the Hazen-Williams equation for pipes flowing full.

(2000 ft long)(8 ft/1000 ft) = 16 ft (4.9 m) of water. This is also the friction-head loss in pipe *b*.

Since pipe *b* is 3000 ft (914.4 m) long, the friction-head loss per 1000 ft (304.8 m) is total head loss, ft/length of pipe, thousands of ft = 16/3 = 5.33 ft/1000 ft (1.6 m/304.8 m). Enter Fig. 10 at this friction-head loss and $C = 100$. Project in the same manner as described for pipe *a*, and find the required size of pipe *b* as 33 in. (838.2 mm).

If the district being supplied by either pipe required a specific flow rate, this flow would be used instead of assuming a flow rate. Then the pipe would be sized in the same manner as described above.

7. *Compute the single-pipe equivalent length*

When we deal with several different sizes of pipe having the same flow rate, it is often convenient to convert each pipe to an *equivalent length* of a common-size pipe. Many design engineers use 8-in. (203-mm) pipe as the common size. Table 4 shows the equivalent length of 8-in. (203-mm) pipe for various other sizes of pipe with $C = 90$, 100, and 110 in the Hazen-Williams equation.

From Table 4, for 12-in. (305-mm) pipe, the equivalent length of 8-in. (203-mm) pipe is 0.14 ft/ft when $C = 100$. Thus, total equivalent length of 8-in. (203-mm) pipe = (1000 ft of 12-in. pipe)(0.14 ft/ft) = 140 ft (42.7 m) of 8-in. (203-mm) pipe. For the 14-in. (356-mm) pipe, total equivalent length = (600)(0.066) = 39.6 ft (12.1 m), using similar data from Table 4. For the 16-in. (406-mm) pipe, total equivalent length = (1400)(0.034) = 47.6 ft (14.5 m). Hence, total equivalent length of 8-in. (203-mm) pipe = 140 + 39.6 + 47.6 = 227.2 ft (69.3 m).

8. *Determine the friction-head loss in the pipe*

Enter Fig. 10 at the flow rate of 2000 gal/min (126.2 L/s), and project through 8-in. (203-mm) diameter to the pivot axis. From this intersection, project through $C = 100$ to read the friction-head loss as 100 ft/1000 ft (30.5 m/304.8 m), due to the friction of the water in the pipe. Since the equivalent length of the pipe is 227.2 ft (69.3 m), the friction-head loss in the compound pipe is (227.2/1000)(110) = 25 ft (7.6 m) of water.

TABLE 4. Equivalent Length of 8-in. (203-mm) Pipe for $C = 100$

Pipe diameter				
in.	mm	$C = 90$	$C = 100$	$C = 110$
2	51	1012	851	712
4	102	34	29	24.3
6	152	4.8	4.06	3.4
8	203	1.19	1.00	0.84
10	254	0.40	0.34	0.285
12	305	0.17	0.14	0.117
14	356	0.078	0.066	0.055
16	406	0.040	0.034	0.029
18	457	0.023	0.019	0.016
20	508	0.0137	0.0115	0.0096
24	610	0.0056	0.0047	0.0039
30	762	0.0019	0.0016	0.0013
36	914	0.00078	0.00066	0.00055

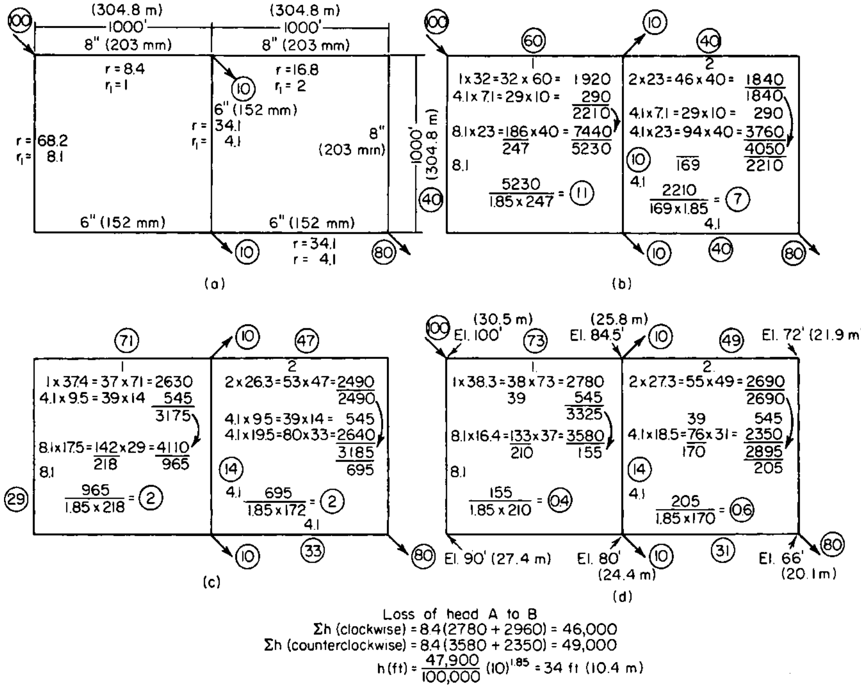


FIGURE 11. Application of the Hardy Cross method to a water distribution system.

Related Calculations. Two pipes, two piping systems, or a single pipe and a system of pipes are said to be *equivalent* when the losses of head due to friction for equal rates of flow in the pipes are equal.

To determine the flow rates and friction-head losses in complex waterworks distribution systems, the Hardy Cross method of network analysis is often used. This method[1] uses trial and error to obtain successively more accurate approximations of the flow rate through a piping system. To apply the Hardy Cross method: (1) Sketch the piping system layout as in Fig. 11. (2) Assume a flow quantity, in terms of percentage of total flow, for each part of the piping system. In assuming a flow quantity note that (*a*) the loss of head due to friction between any two points of a closed circuit must be the same by any path by which the water may flow, and (*b*) the rate of inflow into any section of the piping system must equal the outflow. (3) Compute the loss of head due to friction between two points in each part of the system, based on the assumed flow in (*a*) the clockwise direction and (*b*) the counterclockwise direction. A difference in the calculated friction-head losses in the two directions indicates an error in the assumed direction of flow. (4) Compute a counterflow correction by dividing the difference in head, Δh ft, by $n(Q)^{n-1}$, where $n = 1.85$ and Q = flow, gal/min. Indicate the direction of this counterflow in the pipe by an arrow starting at the right side of the smaller value of h and curving toward the larger value, Fig. 11. (5) Add or subtract the counterflow

[1]O'Rourke—*General Engineering Handbook,* McGraw-Hill.

TABLE 5. Values of r for 1000 ft (304.8 m) of Pipe Based on the Hazen-Williams Formula*

d,in.(mm)	C = 90	C = 100	C = 110	C = 120	C = 130	C = 140
4(102)	340	246	206	176	151	135
6(152)	47.1	34.1	28.6	24.3	21.0	18.7
8(203)	11.1	8.4	7.0	6.0	5.2	4.6
10 (254)	3.7	2.8	2.3	2.0	1.7	1.5
12 (305)	1.6	1.2	1.0	0.85	0.74	0.65
14 (356)	0.72	0.55	0.46	0.39	0.34	0.30
16 (406)	0.38	0.29	0.24	0.21	0.18	0.15
18 (457)	0.21	0.16	0.13	0.11	0.10	0.09
20 (508)	0.13	0.10	0.08	0.07	0.06	0.05
24 (610)	0.052	0.04	0.03	0.03	0.02	0.02
30 (762)	0.017	0.013	0.011	0.009	0.008	0.007

Example: r for 12-in. (305-mm) pipe 4000 ft (1219 m) long, with $C = 100$, is $1.2 \times 4.0 = 4.8$.
*Head loss in ft (m) $= r \times 10^{-5} \times Q^{1.85}$ per 1000 ft (304.8 m), Q representing gal/min (L/s).

to or from the assumed flow, depending on whether its direction is the same or opposite. (6) Repeat this process on each circuit in the system until a satisfactory balance of flow is obtained.

To compute the loss of head due to friction, step 3 of the Hardy Cross method, use any standard formula, such as the Hazen-Williams, that can be reduced to the form $h = rQ^nL$, where h = head loss due to friction, ft of water; r = a coefficient depending on the diameter and roughness of the pipe; Q = flow rate, gal/min; $n = 1.85$; L = length of pipe, ft. Table 5 gives values of r for 1000-ft (304.8-in.) lengths of various sizes of pipe and for different values of the Hazen-Williams coefficient C. When the percentage of total flow is used for computing Σh in Fig. 11, the loss of head due to friction in ft between any two points for any flow in gal/min is computed from h = [Σh (by percentage of flow)/ 100,000] (gal/min/ 100)$^{0.85}$. Figure 11 shows the details of the solution using the Hardy Cross method. The circled numbers represent the flow quantities. Table 6 lists values of numbers between 0 and 100 to the 0.85 power.

TABLE 6. Value of the 0.85 Power of Numbers

N	0	1	2	3	4	5	6	7	8	9
0	0	1.0	1.8	2.5	3.2	3.9	4.6	5.2	5.9	6.5
10	7.1	7.7	8.3	8.9	9.5	10.0	10.6	11.1	11.6	12.2
20	12.8	13.3	13.8	14.4	14.9	15.4	15.9	16.4	16.9	17.5
30	18.0	18.5	19.0	19.5	20.0	20.5	21.0	21.5	22.0	22.5
40	23.0	23.4	23.9	24.3	24.8	25.3	25.8	26.3	26.8	27.3
50	27.8	28.2	28.7	29.1	29.6	30.0	30.5	31.0	31.4	31.9
60	32.4	32.9	33.3	33.8	34.2	34.7	35.1	35.6	36.0	36.5
70	37.0	37.4	37.9	38.3	38.7	39.1	39.6	40.0	40.5	41.0
80	41.5	42.0	42.4	42.8	43.3	43.7	44.1	44.5	45.0	45.4
90	45.8	46.3	46.7	47.1	47.6	48.0	48.4	48.8	49.2	49.6

WATER-SUPPLY SYSTEM SELECTION

Choose the type of water-supply system for a city having a population of 100,000 persons. Indicate which type of system would be suitable for such a city today and 20 years hence. The city is located in an area of numerous lakes.

Calculation Procedure:

1. *Compute the domestic water flow rate in the system*
Use an average annual domestic water consumption of 150 gal per capita day (gcd) (6.6 mL/s). Hence, domestic water consumption = (150 gal per capita day)(100,000 persons) = 15,000,000 gal/day (657.1 L/s). To this domestic flow, the flow required for fire protection must be added to determine the total flow required.

2. *Compute the required flow rate for fire protection*
Use the relation $Q_f = 1020(P)^{0.5}\,[1 - 0.01(P)^{0.5}]$, where Q_f = fire flow, gal/min; P = population in thousands. So $Q_f = 1020(100)^{0.5}\,[1 - 0.01 \times (100)^{0.5}] - 9180$, say 9200 gal/min (580.3 L/s).

3. *Apply a load factor to the domestic consumption*
To provide for unusual water demands, many design engineers apply a 200 to 250 percent load factor to the average hourly consumption that is determined from the average annual consumption. Thus, the average daily total consumption determined in step 1 is based on an average annual daily demand. Convert the average daily total consumption in step 1 to an average hourly consumption by dividing by 24 h, or 15,000,000/24 = 625,000 gal/h (657.1 L/s). Next, apply a 200 percent load factor. Or, design hourly demand = 2.00(625,000) = 1,250,000 gal/h (1314.1 L/s), or 1,250,000/(60 min/h) = 20,850, say 20,900 gal/min (1318.4 L/ s).

4. *Compute the total water flow required*
The total water flow required = domestic flow, gal/min + fire flow, gal/min = 20,900 + 9200 = 30,100 gal/min (1899.0 L/s). If this system were required to supply water to one or more industrial plants in addition to the domestic and fire flows, the quantity needed by the industrial plants would be added to the total flow computed above.

5. *Study the water supplies available*
Table 7 lists the principal sources of domestic water supplies. Wells that are fed by groundwater are popular in areas having sandy or porous soils. To determine whether a

TABLE 7. Typical Municipal Water Sources

Source	Collection method	Remarks
Groundwater	Wells (artesian, ordinary, galleries)	30 to 40 percent of an area's rainfall becomes groundwater
Surface freshwater (lakes, rivers, streams, impounding reservoirs)	Pumping or gravity flow from submerged intakes, tower intakes, or surface intakes	Surface supplies are important in many areas
Surface saltwater	Desalting	Wide-scale application under study at present

well is suitable for supplying water in sufficient quantity, its specific capacity (i.e., the yield in gal/min per foot of drawdown) must be determined.

Wells for municipal water sources may be dug, driven, or drilled. Dug wells seldom exceed 60 ft (18.3 m) deep. Each such well should be protected from surface-water leakage by being lined with impervious concrete to a depth of 15 ft (4.6 m).

Driven wells seldom are more than 40 ft (12.2 m) deep or more than 2 in. (51 mm) in diameter when used for small water supplies. Bigger driven wells are constructed by driving large-diameter casings into the ground.

Drilled wells can be several thousand feet deep, if required. The yield of a driven well is usually greater than any other type of well because the well can be sunk to a depth where sufficient groundwater is available. Almost all wells require a pump of some kind to lift the water from its subsurface location and discharge it to the water-supply system.

Surface freshwater can be collected from lakes, rivers, streams, or reservoirs by submerged-, tower-, or crib-type intakes. The intake leads to one or more pumps that discharge the water to the distribution system or intermediate pumping stations. Locate intakes as far below the water surface as possible. Where an intake is placed less than 20 ft (6.1 m) below the surface of the water, it may become clogged by sand, mud, or ice.

Choose the source of water for this system after studying the local area to determine the most economical source today and 20 years hence. With a rapidly expanding population, the future water demand may dictate the type of water source chosen. Since this city is in an area of many lakes, a surface supply would probably be most economical, if the water table is not falling rapidly.

6. *Select the type of pipe to use*

Four types of pipes are popular for municipal water-supply systems: cast iron, asbestos cement, steel, and concrete. Wood-stave pipe was once popular, but it is now obsolete. Some communities also use copper or lead pipes. However, the use of both types is extremely small when compared with the other types. The same is true of plastic pipe, although this type is slowly gaining some acceptance.

In general, cast-iron pipe proves dependable and long-lasting in water-supply systems that are not subject to galvanic or acidic soil conditions.

Steel pipe is generally used for long, large-diameter lines. Thus, the typical steel pipe used in water-supply systems is 36 or 48 in. (914 or 1219 mm) in diameter. Use steel pipe for river crossings, on bridges, and for similar installations where light weight and high strength are required. Steel pipe may last 50 years or more under favorable soil conditions. Where unfavorable soil conditions exist, the lift of steel pipe may be about 20 years.

Concrete-pipe use is generally confined to large, long lines, such as aqueducts. Concrete pipe is suitable for conveying relatively pure water through neutral soil. However, corrosion may occur when the soil contains an alkali or an acid.

Asbestos-cement pipe has a number of important advantages over other types. However, it does not flex readily, it can be easily punctured, and it may corrode in acidic soils.

Select the pipe to use after a study of the local soil conditions, length of runs required, and the quantity of water that must be conveyed. Usual water velocities in municipal water systems are in the 5-ft/s (1.5-m/s) range. However, the velocities in aqueducts range from 10 to 20 ft/s (3.0 to 6.1 m/s). Earthen canals have much lower velocities—l to 3 ft/s (0.3 to 0.9 m/s). Rock- and concrete-lined canals have velocities of 8 to 15 ft/s (2.4 to 4.6 m/s).

In cold northern areas, keep in mind the occasional need to thaw frozen pipes during the winter. Nonmetallic pipes—concrete, plastic, etc., as well as nonconducting metals—cannot be thawed by electrical means. Since electrical thawing is probably the most practical method available today, pipes that prevent its use may put the water system at a disadvantage if subfreezing temperatures are common in the area served.

7. *Select the method for pressurizing the water system*

Water-supply systems can be pressurized in three different ways: by gravity or natural elevation head, by pumps that produce a pressure head, and by a combination of the first two ways.

Gravity systems are suitable where the water storage reservoir or receiver is high enough above the distribution system to produce the needed pressure at the farthest outlet. The operating cost of a gravity system is lower than that of a pumped system, but the first cost of the former is usually higher. However, the reliability of the gravity system is usually higher because there are fewer parts that may fail.

Pumping systems generally use centrifugal pumps that discharge either directly to the water main or to an elevated tank, a reservoir, or a standpipe. The water then flows from the storage chamber to the distribution system. In general, most sanitary engineers prefer to use a reservoir or storage tank between the pumps and distribution mains because this arrangement provides greater reliability and fewer pressure surges.

Surface reservoirs should store at least a 1-day water supply. Most surface reservoirs are designed to store a supply for 30 days or longer. Elevated tanks should have a capacity of at least 25 gal (94.6 L) of water per person served, *plus* a reserve for fire protection. The capacity of typical elevated tanks ranges from a low of 40,000 gal (151 kL) for a 20-ft (6.1-m) diameter tank to a high of 2,000,000 gal (7.5 ML) for an 80-ft (24.4-m) diameter tank.

Choose the type of distribution system after studying the topography, water demand, and area served. In general, a pumped system is preferred today. To ensure continuity of service, duplicate pumps are generally used.

8. *Choose the system operating pressure*

In domestic water supply, the minimum pressure required at the highest fixture in a building is usually assumed to be 15 lb/sq.in. (103.4 kPa). The maximum pressure allowed at a fixture in a domestic water system is usually 65 lb/sq.in. (448.2 kPa). High-rise buildings (i.e., those above six stories) are generally required to furnish the pressure increase needed to supply water to the upper stories. A pump and overhead storage tank are usually installed in such buildings to provide the needed pressure.

Commercial and industrial buildings require a minimum water pressure of 75 lb/sq.in. (517.1 kPa) at the street level for fire hydrant service. This hydrant should deliver at least 250 gal/min (15.8 L/s) of water for fire-fighting purposes.

Most water-supply systems served by centrifugal pumps in a central pumping station operate in the 100-lb/sq.in. (689.5-kPa) pressure range. In areas of one- and two-story structures, a lower pressure, say 65 lb/sq.in. (448.2 kPa), is permissible. Where the pressure in a system falls too low, auxiliary or booster pumps may be used. These pumps increase the pressure in the main to the desired level.

Choose the system pressure based on the terrain served, quantity of water required, allowable pressure loss, and size of pipe used in the system. Usual pressures required will be in the ranges cited above, although small systems serving one-story residences may operate at pressures as low as 30 lb/sq.in. (206.8 kPa). Pressures over 100 lb/sq.in. (689.5 kPa) are seldom used because heavier piping is required. As a rule, distribution pressures of 50 to 75 lb/sq.in. (344.7 to 517.1 kPa) are acceptable.

9. *Determine the number of hydrants for fire protection*

Table 8 shows the required fire flow, number of standard hose streams of 250 gal/min (15.8 L/s) discharged through a $1^1/_8$-in. (28.6-mm) diameter smooth nozzle, and the average area served by a hydrant in a high-value district. A standard hydrant may have two or three outlets.

Table 8 indicates that a city of 100,000 persons requires 36 standard hose streams. This means that 36 single-outlet or 18 dual-outlet hydrants are required. More, of course,

TABLE 8. Required Fire Flow and Hydrant Spacing*

Population	Required fire flow, gal/min (L/s)	Number of standard hose streams	Average area served per hydrant, sq.ft. (m^2)†	
			Direct streams	Engine streams
22,000	4,500 (284)	18	55,000 (5,110)	90,000 (8,361)
28,000	5,000 (315)	20	40,000 (3,716)	85,000 (7,897)
40,000	6,000 (379)	24	40,000 (3,716)	80,000 (7,432)
60,000	7,000 (442)	28	40,000 (3,716)	70,000 (6,503)
80,000	8,000 (505)	32	40,000 (3,716)	60,000 (5,574)
100,000	9,000 (568)	36	40,000 (3,716)	55,000 (5,110)
125,000	10,000 (631)	40	40,000 (3,716)	48,000 (4,459)
150,000	11,000 (694)	44	40,000 (3,716)	43,000 (3,995)
200,000	12,000 (757)	48	40,000 (3,716)	40,000 (3,716)

*National Board of Fire Underwriters.
†High-value districts.

could be used if better protection were desired in the area. Note that the required fire flow listed in Table 8 agrees closely with that computed in step 2 above.

Related Calculations. Use this general method for any water-supply system, municipal or industrial. Note, however, that the required fire-protection quantities vary from one type of municipal area to another and among different industrial exposures. Refer to *NFPA Handbook of Fire Protection*, available from NFPA, 60 Batterymarch Street, Boston, Massachusetts 02110, for specific fire-protection requirements for a variety of industries. In choosing a water-supply system, the wise designer looks ahead for at least 10 years when the water demand will usually exceed the present demand. Hence, the system may be designed so it is oversized for the present population but just adequate for the future population. The American Society for Testing and Materials (ASTM) publishes comprehensive data giving the usual water requirements for a variety of industries. Table 9 shows a few typical water needs for selected industries.

TABLE 9. Selected Industrial Water and Steam Requirements*

	Water	Steam
Air conditioning	6000 to 15,000 gal (22,700 to 57,000 L) per person per season	. . .
Aluminum	1,920,000 gal/ton (8.0 ML/t)	. . .
Cement, portland	750 gal/ton cement (3129 L/t)	. . .
Coal, by-product coke	1430 to 2800 gal/ton coke (5967 to 11,683 L/t)	570 to 860 lb/ton (382 to 427 kg/t)
Rubber (automotive tire)	. . .	120 lb (54 kg) per tire
Electricity	80 gal/kW (302 L/kW) of electricity	. . .

*Courtesy of American Society for Testing and Materials.

To determine the storage capacity required at present, proceed as follows: (1) Compute the flow needed to meet 50 percent of the present domestic daily (that is, 24-h) demand. (2) Compute the 4-h fire demand. (3) Find the sum of (1) and (2).

For this city, procedure (1) = (20,900 gal/min)(60 min/h)(24 h/day)(0.5) = 15,048,000 gal (57.2 ML) with the data computed in step 3. Also procedure (2) = (4 h)(60 min/h)(9200 gal/min) = 2,208,000 gal (8.4 ML), using the data computed in step 2, above. Then, total storage capacity required = 15,048,000 + 2,208,000 = 17,256,000 gal (65.3 ML). Where one or more reliable wells will produce a significant flow for 4 h or longer, the storage capacity can be reduced by the 4-h productive capacity of the wells.

SELECTION OF TREATMENT METHOD FOR WATER-SUPPLY SYSTEM

Choose a treatment method for a water-supply system for a city having a population of 100,000 persons. The water must be filtered, disinfected, and softened to make it suitable for domestic use.

Calculation Procedure:

1. *Compute the domestic water flow rate in the system*

When water is treated for domestic consumption, only the drinking water passes through the filtration plant. Fire-protection water is seldom treated unless it is so turbid that it will clog fire pumps or hoses. Assuming that the fire-protection water is acceptable for use without treatment, we consider only the drinking water here.

Use the same method as in steps 1 and 3 of the previous calculation procedure to determine the required domestic water flow of 20,900 gal/min (1318.6 L/s) for this city.

2. *Select the type of water-treatment system to use*

Water supplies are treated by a number of methods including sedimentation, coagulation, filtration, softening, and disinfection. Other treatments include disinfection, taste and odor control, and miscellaneous methods.

Since the water must be filtered, disinfected, and softened, each of these steps must be considered separately.

3. *Choose the type of filtration to use*

Slow sand filters operate at an average rate of 3 million gal/(acre·day) [2806.2 L/(m^2·day)]. This type of filter removes about 99 percent of the bacterial content of the water and most tastes and odors.

Rapid sand filters operate at an average rate of 150 million gal/(acre·day) [1.6 L/(m^2·s)]. But the raw water must be treated before it enters the rapid sand filter. This preliminary treatment often includes chemical coagulation and sedimentation. A high percentage of bacterial content—up to 99.98 percent—is removed by the preliminary treatment and the filtration. But color and turbidity removal is not as dependable as with slow sand filters. Table 10 lists the typical limits for certain impurities in water supplies.

The daily water flow rate for this city is, from step 1, (20,900 gal/min)(24 h/day)(60 min/h) = 30,096,000 gal/day (1318.6 L/s). If a slow sand filter were used, the required area would be (30.096 million gal/day)/[3 million gal/(acre·day)] = 10+ acres (40,460 m^2).

TABLE 10. Typical Limits for Impurities in Water Supplies

Impurity	Limit, ppm	Impurity	Limit, ppm
Turbidity	10	Iron plus manganese	0.3
Color	20	Magnesium	125
Lead	0.1	Total solids	500
Fluoride	1.0	Total hardness	100
Copper	3.0	Ca + Mg salts	

A rapid sand filter would require 30.096/150 = 0.2 acre (809.4 m^2). Hence, if space were scarce in this city—and it usually is—a rapid sand filter would be used. With this choice of filtration, chemical coagulation and sedimentation are almost a necessity. Hence, these two additional steps would be included in the treatment process.

Table 11 gives pertinent data on both slow and rapid sand filters. These data are useful in filter selection.

4. *Select the softening process to use*

The principal water-softening processes use: (*a*) lime and sodium carbonate followed by sedimentation or filtration, or both, to remove the precipitates and (*b*) zeolites of the sodium type in a pressure filter. Zeolite softening is popular and is widely used in municipal water-supply systems today. Based on its proven usefulness and economy, zeolite softening will be chosen for this installation.

TABLE 11. Typical Sand-Filter Characteristics

Slow sand filters	
Usual filtration rate	2.5 to 6.0 × 10^6 gal/(acre·day) [2339 to 5613 L/(m^2·day)]
Sand depth	30 to 36 in. (76 to 91 cm)
Sand size	35 mm
Sand uniformity coefficient	1.75
Water depth	3 to 5 ft (0.9 to 1.5 m)
Water velocity in underdrains	2 ft/s (0.6 m/s)
Cleaning frequency required	2 to 11 times per year
Units required	At least two to permit alternate cleaning
Fast sand filters	
Usual filtration rate	100 to 200 × 10^6 gal/(acre·day) [24.7 to 49.4 kL/(m^2·day)]
Sand depth	30 in. (76 cm)
Gravel depth	18 in. (46 cm)
Sand size	0.4 to 0.5 mm
Sand uniformity coefficient	1.7 or less
Units required	At least three to permit cleaning one unit while the other two are operating

5. *Select the disinfection method to use*

Chlorination by the addition of chlorine to the water is the principal method of disinfection used today. To reduce the unpleasant effects that may result from using chlorine alone, a mixture of chlorine and ammonia, known as chloramine, may be used. The ammonia dosage is generally 0.25 ppm or less. Assume that the chloramine method is chosen for this installation.

6. *Select the method of taste and odor control*

The methods used for taste and odor control are: (*a*) aeration, (*b*) activated carbon, (*c*) prechlorination, and (*d*) chloramine. Aeration is popular for groundwaters containing hydrogen sulfide and odors caused by microscopic organisms.

Activated carbon absorbs impurities that cause tastes, odors, or color, generally, 10 to 20 lb (4.5 to 9.1 kg) of activated carbon per million gallons of water is used, but larger quantities—from 50 to 60 lb (22.7 to 27.2 kg)—may be specified. In recent years, some 2000 municipal water systems have installed activated carbon devices for taste and odor control.

Prechlorination and chloramine are also used in some installations for taste and odor control. Of the two methods, chloramine appears more popular at present.

Based on the data given for this water-supply system, method *b, c,* or *d* would probably be suitable. Because method *b* has proven highly effective, it will be chosen tentatively, pending later investigation of the economic factors.

Related Calculations. Use this general procedure to choose the treatment method for all types of water-supply systems where the water will be used for human consumption. Thus, the procedure is suitable for municipal, commercial, and industrial systems.

Hazardous wastes of many types endanger groundwater supplies. One of the most common hazardous wastes is gasoline, which comes from the estimated 120,000 leaking underground gasoline-storage tanks. Major oil companies are replacing leaking tanks with new noncorrosive tanks. But the soil and groundwater must still be cleaned to prevent pollution of drinking-water supplies.

Other contaminants include oily sludges, organic (such as pesticides and dioxins), and nonvolatile organic materials. These present especially challenging removal and disposal problems for engineers, particularly in view of the stringent environmental requirements of almost every community.

A variety of treatment and disposal methods are in the process of development and application. For oily waste handling, one process combines water evaporation and solvent extraction to break down a wide variety of hazardous waste and sludge from industrial, petroleum-refinery, and municipal-sewage-treatment operations. This process typically produces dry solids with less than 0.5 percent residual hydrocarbon content. This meets EPA regulations for nonhazardous wastes with low heavy-metal contents.

Certain organics, such as pesticides and dioxins, are hydrophobic. Liquified propane and butane are effective at separating hydrophobic organics from solid particles in tainted sludges and soils. The second treatment method uses liquified propane to remove organics from contaminated soil. Removal efficiencies reported are: polychlorinated biphenyls (PCBs) 99.9 percent; polyaromatic hydrocarbons (PAHs) 99.5 percent; dioxins 97.4 percent; total petroleum hydrocarbons 99.9 percent. Such treated solids meet EPA land-ban regulations for solids disposal.

Nonvolatile organic materials at small sites can be removed by a mobile treatment system using up to 14 solvents. Both hydrophobic and hydrophilic solvents are used; all are nontoxic; several have Food and Drug Administration (FDA) approval as food additives. Used at three different sites (at this writing) the process reduced PCB concentration from 500 to 1500 ppm to less than 100 ppm; at another site PCB concentration was reduced from an average of 30 to 300 ppm to less than 5 ppm; at the third site PCBs were reduced from 40 ppm to less than 3 ppm.

STORM-WATER RUNOFF RATE AND RAINFALL INTENSITY

What is the storm-water runoff rate from a 40-acre (1.6-km^2) industrial site having an imperviousness of 50 percent if the time of concentration is 15 min? What would be the effect of planting a lawn over 75 percent of the site?

Calculation Procedure:

1. *Compute the hourly rate of rainfall*

Two common relations, called the *Talbot formulas*, used to compute the hourly rate of rainfall R in./h are $R = 360/(t + 30)$ for the heaviest storms and $R = 105/(t + 15)$ for ordinary storms, where t = time of concentration, min. Using the equation for the heaviest storms because this relation gives a larger flow rate and produces a more conservative design, we see $R = 360/(15 + 30) = 8$ in./h (0.05 mm/s).

2. *Compute the storm-water runoff rate*

Apply the *rational method* to compute the runoff rate. This method uses the relation $Q = AIR$, where Q = storm-water runoff rate, ft^3/s; A = area served by sewer, acres; I = coefficient of runoff or percentage of imperviousness of the area; other symbols as before. So $Q = (40)(0.50)(8) = 160$ ft^3/s (4.5 m^3/s).

3. *Compute the effect of changed imperviousness*

Planting a lawn on a large part of the site will increase the imperviousness of the soil. This means that less rainwater will reach the sewer because the coefficient of imperviousness of a lawn is lower. Table 12 lists typical coefficients of imperviousness for various surfaces. This tabulation shows that the coefficient for lawns varies from 0.05 to 0.25. Using a value of $I = 0.10$ for the $40(0.75) = 30$ acres of lawn, we have $Q = (30)(0.10)(8) = 24$ ft^3/s (0.68 m^3/s).

TABLE 12. Coefficient of Runoff for Various Surfaces

Surface	Coefficient
Parks, gardens, lawns, meadows	0.05–0.25
Gravel roads and walks	0.15–0.30
Macadamized roadways	0.25–0.60
Inferior block pavements with uncemented joints	0.40–0.50
Stone, brick, and wood-block pavements with tightly cemented joints	0.75–0.85
Same with uncemented joints	0.50–0.70
Asphaltic pavements in good condition	0.85–0.90
Watertight roof surfaces	0.70–0.95

TABLE 13. Coefficient of Runoff for Various Areas

Area	Coefficient
Business:	
Downtown	0.70−0.95
Neighborhood	0.50−0.70
Residential:	
Single-family	0.30−0.50
Multiunits, detached	0.40−0.60
Multiunits, attached	0.60−0.75
Residential (suburban)	0.25−0.40
Apartment dwelling	0.50−0.70
Industrial:	
Light industry	0.50−0.80
Heavy industry	0.60−0.90
Playgrounds	0.20−0.35
Railroad yards	0.20−0.40
Unimproved	0.10−0.30

The runoff for the remaining 10 acres (40,460 m^2) is, as in step 2, $Q = (10)(0.5)(8) = 40$ ft^3/s (1.1 m^3/s). Hence, the total runoff is $24 + 40 = 64$ ft^3/s (1.8 m^3/s). This is $160 - 64 = 96$ ft^3/s (2.7 m^3/s) less than when the lawn was not used.

Related Calculations. The time of concentration for any area being drained by a sewer is the time required for the maximum runoff rate to develop. It is also defined as the time for a drop of water to drain from the farthest point of the watershed to the sewer.

When rainfall continues for an extended period T min, the coefficient of imperviousness changes. For impervious surfaces such as watertight roofs, $I = T/(8 + T)$. For improved pervious surfaces, $I = 0.3T/(20 + T)$. These relations can be used to compute the coefficient in areas of heavy rainfall.

Equations for R for various areas of the United States are available in Steel—*Water Supply and Sewerage*, McGraw-Hill. The Talbot formulas, however, are widely used and have proved reliable.

The time of concentration for a given area can be approximated from $t = I(L/Si^2)^{1/3}$ where L = distance of overland flow of the rainfall from the most remote part of the site, ft; S = slope of the land, ft/ft; i = rainfall intensity, in./h; other symbols as before. For portions of the flow carried in ditches, the time of flow to the inlet can be computed by using the Manning formula.

Table 13 lists the coefficient of runoff for specific types of built-up and industrial areas. Use these coefficients in the same way as shown above. Tables 12 and 13 present data developed by Kuichling and ASCE.

SIZING SEWER PIPES FOR VARIOUS FLOW RATES

Determine the size, flow rate, and depth of flow from a 1000-ft (304.8-m) long sewer which slopes 5 ft (1.5 m) between inlet and outlet and which must carry a flow of 5 million gal/day (219.1 L/s). The sewer will flow about half full. Will this sewer provide the desired flow rate?

Calculation Procedure:

1. *Compute the flow rate in the half-full sewer*

A flow of 1 million gal/day = 1.55 ft^3/s (0.04 m^3/s). Hence, a flow of 5 million gal/day = 5(1.55) = 7.75 ft^3/s (219.1 L/s) in a *half-full* sewer.

2. *Compute the full-sewer flow rate*

In a *full sewer,* the flow rate is twice that in a half-full sewer, or 2(7.75) = 15.50 ft^3/s (0.44 m^3/s) for this sewer. This is equivalent to 15.50/1.55 = 10 million gal/day (438.1 L/s). Full-sewer flow rates are used because pipes are sized on the basis of being full of liquid.

3. *Compute the sewer-pipe slope*

The pipe slope S ft/ft = $(E_i - E_0)/L$, where E_i = inlet elevation, ft above the site datum; E_o = outlet elevation, ft above site datum; L = pipe length between inlet and outlet, ft. Substituting gives S = 5/1000 = 0.005 ft/ft (0.005 in./in.).

4. *Determine the pipe size to use*

The Manning formula $v = (1.486/n)R^{2/3}S^{1/2}$ is often used for sizing sewer pipes. In this formula, v = flow velocity, ft/s; n = a factor that is a function of the pipe roughness; R = pipe hydraulic radius = 0.25 pipe diameter, ft; S = pipe slope, ft/ft. Table 14 lists values of n for various types of sewer pipe. In sewer design, the value n = 0.013 for pipes flowing full.

Since the Manning formula is complex, numerous charts have been designed to simplify its solution. Figure 12 is one such typical chart designed specifically for sewers.

Enter Fig. 12 at 15.5 ft^3/s (0.44 m^3/s) on the left, and project through the slope ratio of 0.005. On the central scale between the flow rate and slope scales, read the *next larger* standard sewer-pipe diameter as 24 in. (610 mm). When using this chart, always read the next larger pipe size.

5. *Determine the fluid flow velocity*

Continue the solution line of step 4 to read the fluid flow velocity as 5 ft/s (1.5 m/s) on the extreme right-hand scale of Fig. 12. This is for a sewer flowing *full*.

TABLE 14. Values of n for the Manning Formula

Type of surface of pipe	n
Ditches and rivers, rough bottoms with much vegetation	0.040
Ditches and rivers in good condition with some stones and weeds	0.030
Smooth earth or firm gravel	0.020
Rough brick; tuberculated iron pipe	0.017
Vitrified tile and concrete pipe poorly jointed and unevenly settled; average brickwork	0.015
Good concrete; riveted steel pipe; well-laid vitrified tile or brickwork	0.013*
Cast-iron pipe of ordinary roughness; unplaned timber	0.012
Smoothest pipes; neat cement	0.010
Well-planed timber evenly laid	0.009

*Probably the most frequently used value.

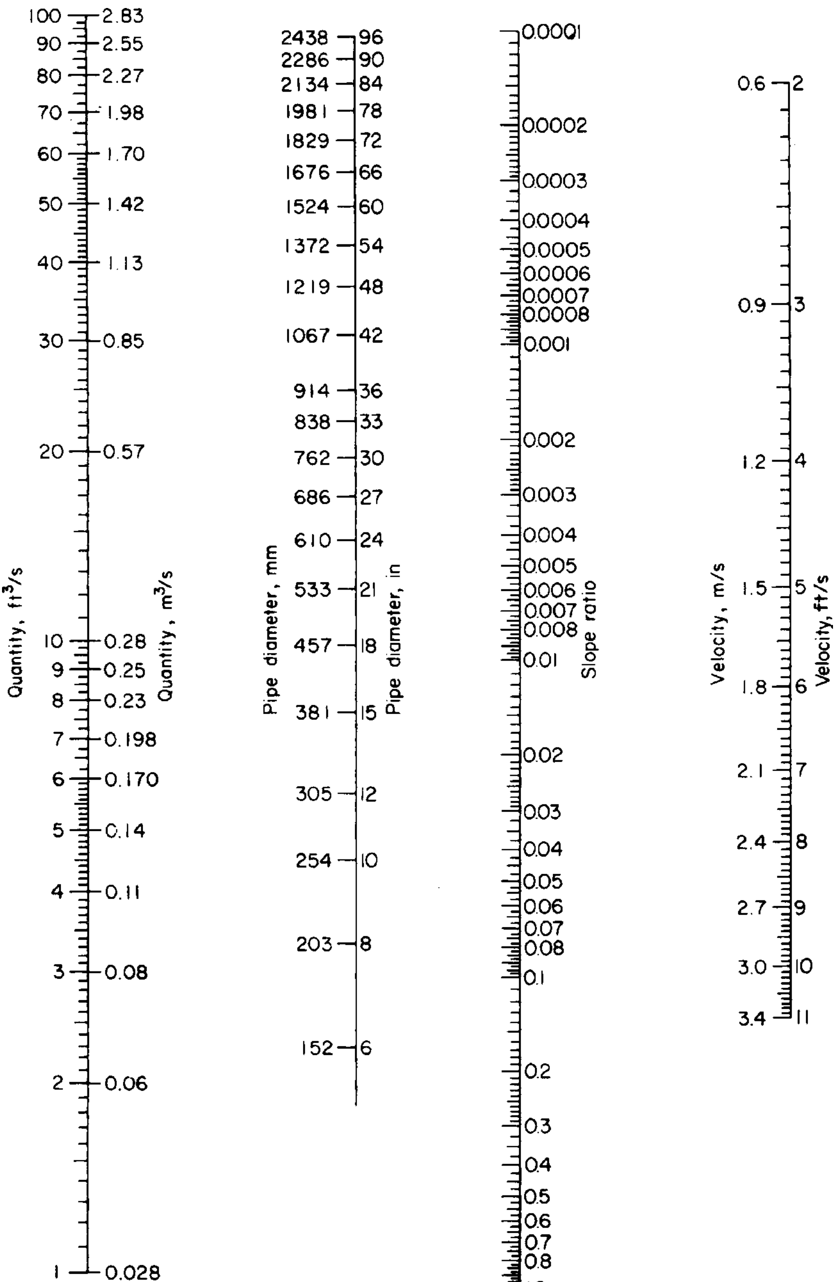


FIGURE 12. Nomogram for solving the Manning formula for circular pipes flowing full and $n = 0.013$.

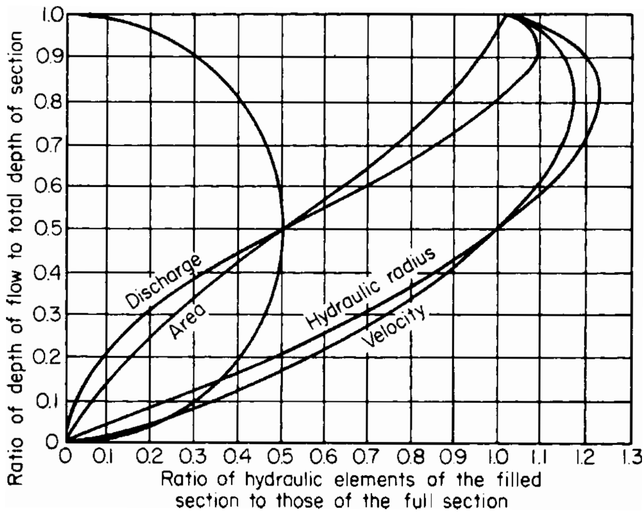


FIGURE 13. Hydraulic elements of a circular pipe.

6. *Compute the half-full flow depth*

Determine the full-flow capacity of this 24-in. (610-mm) sewer by entering Fig. 12 at the slope ratio, 0.005, and projecting through the pipe diameter, 24 in. (610 mm). At the left read the full-flow capacity as 16 ft^3/s (0.45 m^3/s).

The required half-flow capacity is 7.75 ft^3/s (0.22 m^3/s), from step 1. Determine the ratio of the required half-flow capacity to the full-flow capacity, both expressed in ft^3/s. Or $7.75/16.0 = 0.484$.

Enter Fig. 13 on the bottom at 0.484, and project vertically upward to the discharge curve. From the intersection, project horizontally to the left to read the depth-of-flow ratio as 0.49. This means that the depth of liquid in the sewer at a flow of 7.75 ft^3/s (0.22 m^3/s) is 0.49(24 in.) = 11.75 in. (29.8 cm). Hence, the sewer will be just slightly less than half full when handling the designed flow quantity.

7. *Compute the half-full flow velocity*

Project horizontally to the right along the previously found 0.49 depth-of-flow ratio until the velocity curve is intersected. From this intersection, project vertically downward to the bottom scale to read the ratio of hydraulic elements as 0.99. Hence, the fluid velocity when flowing half-full is 0.99(5.0 ft/s) = 4.95 ft/s (1.5 m/s).

Related Calculations. The minimum flow velocity required in sanitary sewers is 2 ft/s (0.6 m/s). At 2 ft/s (0.6 m/s), solids will not settle out of the fluid. Since the velocity in this sewer is 4.95 ft/s (1.5 m/s), as computed in step 7, the sewer meets, and exceeds, the minimum required flow velocity.

Certain localities have minimum slope requirements for sanitary sewers. The required slope produces a minimum flow velocity of 2 ft/s (0.6 m/s) with an n value of 0.013.

Storm sewers handling rainwater and other surface drainage require a higher flow velocity than sanitary sewers because sand and grit often enter a storm sewer. The usual minimum allowable velocity for a storm sewer is 2.5 ft/s (0.76 m/s); where possible, the sewer should be designed for 3.0 ft/s (0.9 m/s). If the sewer designed above were used for storm service, it would be acceptable because the fluid velocity is 4.95 ft/s (1.5 m/s). To prevent excessive wear of the sewer, the fluid velocity should not exceed 8 ft/s (2.4 m/s).

Note that Figs. 12 and 13 can be used whenever two variables are known. When a sewer flows at 0.8, or more, full, the partial-flow diagram, Fig. 13, may not give accurate results, especially at high flow velocities.

SEWER-PIPE EARTH LOAD AND BEDDING REQUIREMENTS

A 36-in. (914-mm) diameter clay sewer pipe is placed in a 15-ft (4.5-m) deep trench in damp sand. What is the earth load on this sewer pipe? What bedding should be used for the pipe? If a 5-ft (1.5-m) wide drainage trench weighing 2000 lb/ft (2976.3 kg/m) of length crosses the sewer pipe at right angles to the pipe, what load is transmitted to the pipe? The bottom of the flume is 11 ft (3.4 m) above the top of the sewer pipe.

Calculation Procedure:

1. *Compute the width of the pipe trench*

Compute the trench width from $w = 1.5d + 12$, where w = trench width, in.; d = sewer-pipe diameter, in. So $w = 1.5(36) + 12 = 66$ in. (167.6 cm), or 5 ft 6 in. (1.7 m).

2. *Compute the trench depth-to-width ratio*

To determine this ratio, subtract the pipe diameter from the depth and divide the result by the trench width. Or, $(15 - 3)/5.5 = 2.18$.

3. *Compute the load on the pipe*

Use the relation $L = kWw^2$, where L = pipe load, lb/lin ft of trench; k = a constant from Table 15; W = weight of the fill material used in the trench, lb/ft^3 other symbol as before.

TABLE 15. Values of k for Use in the Pipe Load Equation*

Ratio of trench depth to width	Sand and damp topsoil	Saturated topsoil	Damp clay	Saturated clay
0.5	0.46	0.46	0.47	0.47
1.0	0.85	0.86	0.88	0.90
1.5	1.18	1.21	1.24	1.28
2.0	1.46	1.50	1.56	1.62
2.5	1.70	1.76	1.84	1.92
3.0	1.90	1.98	2.08	2.20
3.5	2.08	2.17	2.30	2.44
4.0	2.22	2.33	2.49	2.66
4.5	2.34	2.47	2.65	2.87
5.0	2.45	2.59	2.80	3.03
5.5	2.54	2.69	2.93	3.19
6.0	2.61	2.78	3.04	3.33
6.5	2.68	2.86	3.14	3.46
7.0	2.73	2.93	3.22	3.57
7.5	2.78	2.98	3.30	3.67

*Iowa State Univ. Eng. Exp. Sta. Bull. 47.

TABLE 16. Weight of Pipe-Trench Fill

Fill	lb/ft^3	kg/m^3
Dry sand	100	1601
Damp sand	115	1841
Wet sand	120	1921
Damp clay	120	1921
Saturated clay	130	2081
Saturated topsoil	115	1841
Sand and damp topsoil	100	1601

Enter Table 15 at the depth-to-width ratio of 2.18. Since this particular value is not tabulated, use the next higher value, 2.5. Opposite this, read $k = 1.70$ for a sand filling.

Enter Table 16 at damp sand, and read the weight as 115 lb/ft^3 (1842.1 kg/m^3). With these data the pipe load relation can be solved.

Substituting in $L = kWw^2$, we get $L = 1.70(115)(5.5)^2 = 5920$ lb/ft (86.4 N/mm). Study of the properties of clay pipe (Table 17) shows that 36-in. (914-mm) extra-strength clay pipe has a minimum average crushing strength of 6000 lb (26.7 kN) by the three-edge-bearing method.

4. *Apply the loading safety factor*

ASTM recommends a factor of safety of 1.5 for clay sewers. To apply this factor of safety, divide it into the tabulated three-edge-bearing strength found in step 3. Or, 6000/1.5 = 4000 lb (17.8 kN).

5. *Compute the pipe load-to-strength ratio*

Use the strength found in step 4. Or pipe load-to-strength ratio (also called the *load factor*) = 5920/4000 = 1.48.

TABLE 17. Clay Pipe Strength

Pipe size, in. (mm)	Minimum average strength, lb/lin ft (N/mm): Three-edge-bearing	Minimum average strength, lb/lin ft (N/mm): Sand-bearing
4 (102)	1000 (14.6)	1500 (21.9)
6 (152)	1100 (16.1)	1650 (24.1)
8 (203)	1300 (18.9)	1950 (28.5)
10 (254)	1400 (20.4)	2100 (30.7)
12 (305)	1500 (21.9)	2250 (32.9)
15 (381)	1750 (25.6)	2625 (38.3)
18 (457)	2000 (29.2)	3000 (43.8)
21 (533)	2200 (32.1)	3300 (48.2)
24 (610)	2400 (35.0)	3600 (52.6)
27 (686)	2750 (40.2)	4125 (60.2)
30 (762)	3200 (46.7)	4800 (70.1)
33 (838)	3500 (51.1)	5250 (76.7)
36 (914)	3900 (56.9)	5850 (85.4)

6. *Select the bedding method for the pipe*

Figure 14 shows methods for bedding sewer pipe and the strength developed. Thus, earth embedment, type 2 bedding, develops a load factor of 1.5. Since the computed load factor, step 5, is 1.48, this type of bedding is acceptable. (In choosing a type of bedding be certain that the load factor of the actual pipe is less than, or equals, the developed load factor for the three-edge-bearing strength.)

The type 2 earth embedment, Fig. 14, is a highly satisfactory method, except that the shaping of the lower part of the trench to fit the pipe may be expensive. Type 3 granular embedment may be less expensive, particularly if the crushed stone, gravel, or shell is placed by machine.

7. *Compute the direct load transmitted to the sewer pipe*

The weight of the drainage flume is carried by the soil over the sewer pipes. Hence, a portion of this weight may reach the sewer pipe. To determine how much of the flume weight

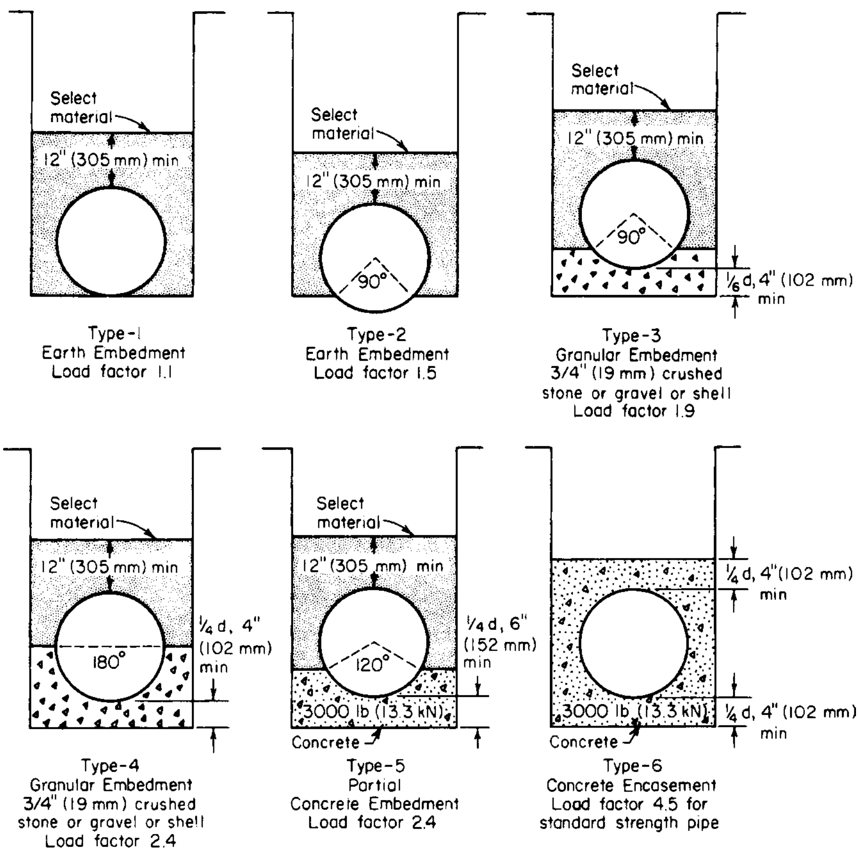


FIGURE 14. Strengths developed for various methods of bedding sewer pipes. (*W. S. Dickey Clay Manufacturing Co.*)

TABLE 18. Proportion of Short Loads Reaching Pipe in Trenches

Depth-to-width ratio	Sand and damp topsoil	Saturated topsoil	Damp clay	Saturated clay
0.0	1.00	1.00	1.00	1.00
0.5	0.77	0.78	0.79	0.81
1.0	0.59	0.61	0.63	0.66
1.5	0.46	0.48	0.51	0.54
2.0	0.35	0.38	0.40	0.44
2.5	0.27	0.29	0.32	0.35
3.0	0.21	0.23	0.25	0.29
4.0	0.12	0.14	0.16	0.19
5.0	0.07	0.09	0.10	0.13
6.0	0.04	0.05	0.06	0.08
8.0	0.02	0.02	0.03	0.04
10.0	0.01	0.01	0.01	0.02

reaches the pipe, find the weight of the flume per foot of width, or 2000 lb/5 ft = 400 lb/ft (5.84 kN/mm) of width.

Since the pipe trench is 5.5 ft (1.7 m) wide, step 1, the 1-ft (0.3-m) wide section of the flume imposes a total load of 5.5(400) = 2200 lb (9.8 kN) on the soil beneath it.

To determine what portion of the flume load reaches the sewer pipe, compute the ratio of the depth of the flume bottom to the width of the sewer-pipe trench, or 11/5.5 = 2.0.

Enter Table 18 at a value of 2.0, and read the load proportion for sand and damp topsoil as 0.35. Hence, the load of the flume reaching each foot of sewer pipe is 0.35(2200) = 770 lb (3.4 kN).

Related Calculations. A load such as that in step 7 is termed a *short load*; i.e., it is shorter than the pipe-trench width. Typical short loads result from automobile and truck traffic, road rollers, building foundations, etc. *Long loads* are imposed by weights that are longer than the trench is wide. Typical long loads are stacks of lumber, steel, and poles,

TABLE 19. Proportion of Long Loads Reaching Pipe in Trenches

Depth-to-width ratio	Sand and damp topsoil	Saturated topsoil	Damp yellow clay	Saturated yellow clay
0.0	1.00	1.00	1.00	1.00
0.5	0.85	0.86	0.88	0.89
1.0	0.72	0.75	0.77	0.80
1.5	0.61	0.64	0.67	0.72
2.0	0.52	0.55	0.59	0.64
2.5	0.44	0.48	0.52	0.57
3.0	0.37	0.41	0.45	0.51
4.0	0.27	0.31	0.35	0.41
5.0	0.19	0.23	0.27	0.33
6.0	0.14	0.17	0.20	0.26
8.0	0.07	0.09	0.12	0.17
10.0	0.04	0.05	0.07	0.11

and piles of sand, coal, gravel, etc. Table 19 shows the proportion of long loads transmitted to buried pipes. Use the same procedure as in step 7 to compute the load reaching the buried pipe.

When a sewer pipe is placed on undisturbed ground and covered with fill, compute the load on the pipe from $L = kWd^2$, where d = pipe diameter, ft; other symbols as in step 3. Tables 18 and 19 are the work of Prof. Anson Marston, Iowa State University.

To find the total load on trenched or surface-level buried pipes subjected to both fill and long or short loads, add the proportion of the long or short load reaching the pipe to the load produced by the fill.

Note that sewers may have several cross-sectional shapes—circular, egg, rectangular, square, etc. The circular sewer is the most common because it has a number of advantages, including economy. Egg-shaped sewers are not as popular as circular and are less often used today because of their higher costs.

Rectangular and square sewers are often used for storm service. However, their hydraulic characteristics are not as desirable as circular sewers.

STORM-SEWER INLET SIZE AND FLOW RATE

What size storm-sewer inlet is required to handle a flow of 2 ft^3/s (0.057 m^3/s) if the gutter is sloped $^1/_4$ in./ft (2.1 cm/m) across the inlet and 0.05 in./ft (0.4 cm/m) along the length of the inlet? The maximum depth of flow in the gutter is estimated to be 0.2 ft (0.06 m), and the gutter is depressed 4 in. (102 mm) below the normal street level.

Calculation Procedure:

1. *Compute the reciprocal of the gutter transverse slope*

The *transverse slope* of the gutter across the inlet is $^1/_4$ in./ft (2.1 cm/m). Expressing the reciprocal of this slope as r, compute the value for this gutter as $r = 4 \times 12/1 = 48$.

2. *Determine the inlet capacity per foot of length*

Enter Table 20 at the flow depth of 0.2 ft (0.06 m), and project to the depth of depression of the gutter of 4 in. (102 mm). Opposite this depth, read the inlet capacity per foot of length as 0.50 ft^3/s (0.014 m^3/s).

3. *Compute the required gutter inlet length*

The gutter must handle a maximum flow of 2 ft^3/s (0.057 m^3/s). Since the inlet has a capacity of 0.50 ft^3/(s·ft) [0.047 m^3/(m·s)] of length, the required length, ft = maximum required capacity, ft^3/s/capacity per foot, ft^3/s = 2.0/0.50 = 4.0 ft (1.2 m). A length of 4.0 ft (1.2 m) will be satisfactory. Were a length of 4.2 or 4.4 ft (1.28 or 1.34 m) required, a 4.5-ft (1.37-m) long inlet would be chosen. The reasoning behind the choice of a longer length is that the extra initial investment for the longer length is small compared with the extra capacity obtained.

4. *Determine how far the water will extend from the curb*

Use the relation $l = rd$, where l = distance water will extend from the curb, ft; d = depth of water in the gutter at the curb line, ft; other symbols as before. Substituting, we find $l = 48(0.2) = 9.6$ ft (2.9 m). This distance is acceptable because the water would extend out this far only during the heaviest storms.

Related Calculations. To compute the flow rate in a gutter, use the relation $F = 0.56(r/n)s^{0.5}d^{8/3}$, where F = flow rate in gutter, ft^3/s; n = roughness coefficient, usually

TABLE 20. Storm-Sewer Inlet Capacity per Foot (Meter) of Length

Flow depth in gutter, ft (mm)	Depression depth, in. (mm)	Capacity per foot length, ft^3/s (m^3/s)
0.2 (0.06)	0 (0)	0.062 (5.76)
	1 (25.4)	0.141 (13.10)
	2 (50.8)	0.245 (22.76)
	3 (76.2)	0.358 (33.26)
	4 (101.6)	0.500 (46.46)
0.3 (0.09)	0 (0)	0.115 (10.69)
	1 (25.4)	0.205 (19.05)
	2 (50.8)	0.320 (29.73)
	3 (76.2)	0.450 (41.81)
	4 (101.6)	0.590 (54.82)

taken as 0.015; s = gutter slope, in./ft; other symbols as before. Where the computed inlet length is 5 ft (1.5 m) or more, some engineers assume that a portion of the water will pass the first inlet and enter the next one along the street.

STORM-SEWER DESIGN

Design a storm-sewer system for a 30-acre (1.21×10^5-m^2) residential area in which the storm-water runoff rate is computed to be 24 ft^3/s (0.7 m^3/s). The total area is divided into 10 plots of equal area having similar soil and runoff conditions.

Calculation Procedure:

1. *Sketch a plan of the sewer system*

Sketch the area and the 10 plots as in Fig. 15. A scale of 1 in. = 100 ft (1 cm = 12 m) is generally suitable. Indicate the terrain elevations by drawing the profile curves on the plot plan. Since the profiles (Fig. 15) show that the terrain slopes from north to south, the main sewer can probably be best run from north to south. The sewer would also slope downward from north to south, following the general slope of the terrain.

Indicate a storm-water inlet for each of the areas served by the sewer. With the terrain sloping from north to south, each inlet will probably give best service if it is located on the southern border of the plot.

Since the plots are equal in area, the main sewer can be run down the center of the plot with each inlet feeding into it. Use arrows to indicate the flow direction in the laterals and main sewer.

2. *Compute the lateral sewer size*

Each lateral sewer handles 24 ft^3/s/10 plots = 2.4 ft^3/s (0.07 m^3/s) of storm water. Size each lateral, using the Manning formula with $n = 0.013$ and full flow in the pipe. Assume a slope ratio of 0.05 for each inlet pipe between the inlet and the main sewer. This means

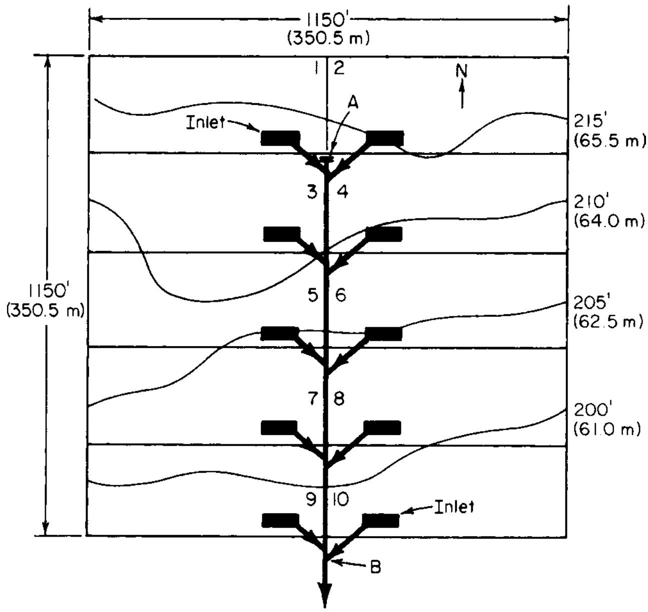


FIGURE 15. Typical storm-sewer plot plan and layout diagram.

that the inlet pipe will slope 1 ft in 20 ft (0.3 m in 6.1 m) of length. In an installation such as this, a slope ratio of 0.05 is adequate.

By using Fig. 12 for a flow of 2.4 ft^3/s (0.0679 m^3/s) and a slope of 0.05, an 8-in. (203-mm) pipe is required for each lateral. The fluid velocity is, from Fig. 12, 7.45 ft/s (2.27 m/s). This is a high enough velocity to prevent solids from settling out of the water. [The flow velocity should not be less than 2 ft/s (0.61 m/s).]

3. *Compute the size of the main sewer*

There are four sections of the main sewer (Fig. 15). The first section, section 3-4, serves the two northernmost plots. Since the flow from each plot is 2.4 ft^3/s (0.0679 m^3/s), the storm water that this portion of the main sewer must handle is $2(2.4) = 4.8$ ft^3/s (0.14 m^3/s).

The main sewer begins at point A, which has an elevation of about 213 ft (64.9 m), as shown by the profile. At point B the terrain elevation is about 190 ft (57.8 m). Hence, the slope between points A and B is about $213 - 190 = 23$ ft (7.0 m), and the distance between the two points is about 920 ft (280.4 m).

Assume a slope of 1 ft/100 ft (0.3 m/30.5 m) of length, or $1/100 = 0.01$ for the main sewer. This is a typical slope used for main sewers, and it is within the range permitted by a pipe run along the surface of this terrain. Table 21 shows the minimum slope required to produce a flow velocity of 2 ft/s (0.61 m/s).

Using Fig. 12 for a flow of 4.8 ft^3/s (0.14 m^3/s) and a slope of 0.01, we see the required size for section 3-4 of the main sewer is 15 in. (381 mm). The flow velocity in the pipe is 4.88 ft/s (1.49 m/s). The size of this sewer is in keeping with general design practice, which seldom uses a storm sewer less than 12 in. (304.8 mm) in diameter.

TABLE 21. Minimum Slope of Sewers*

Sewer diameter, in. (mm)	Minimum slope, ft/100 ft (m/30.5 m) of length
4 (102)	1.20 (0.366)
6 (152)	0.60 (0.183)
8 (203)	0.40 (0.122)
10 (254)	0.29 (0.088)
12 (305)	0.22 (0.067)
15 (381)	0.15 (0.046)
18 (457)	0.12 (0.037)
20 (505)	0.10 (0.030)
24 (610)	0.08 (0.024)

*Based on the Manning formula with $n = 0.13$ and the sewer flowing either full or half full.

Section 5-6 conveys 9.6 ft^3/s (0.27 m^3/s). Using Fig. 12 again, we find the required pipe size is 18 in. (457.2 mm) and the flow velocity is 5.75 ft/s (1.75 m/s). Likewise, section 7-8 must handle 14.4 ft^3/s (0.41 m^3/s). The required pipe size is 21 in. (533 mm), and the flow velocity in the pipe is 6.35 ft/s (1.94 m/s). Section 9-10 of the main sewer handles 19.2 ft^3/s (0.54 m^3/s), and must be 24 in. (609.6 mm) in diameter. The velocity in this section of the sewer pipe will be 6.9 ft/s (2.1 m/s). The last section of the main sewer handles the total flow, or 24 ft^3/s (0.7 m^3/s). Its size must be 27 in. (686 mm), Fig. 12, although a 24-in. (610.0-mm) pipe would suffice if the slope at point *B* could be increased to 0.012.

Related Calculations. Most new sewers built today are the *separate* type, i.e., one sewer for sanitary service and another sewer for storm service. Sanitary sewers are usually installed first because they are generally smaller than storm sewers and cost less. *Combined sewers* handle both sanitary and storm flows and are used where expensive excavation for underground sewers is necessary. Many older cities have combined sewers.

To size a combined sewer, compute the sum of the maximum sanitary and stormwater flow for each section of the sewer. Then use the method given in this procedure after having assumed a value for *n* in the Manning formula and for the slope of the sewer main.

Where a continuous slope cannot be provided for a sewer main, a pumping station to lift the sewage must be installed. Most cities require one or more pumping stations because the terrain does not permit an unrestricted slope for the sewer mains. Motor-driven centrifugal pumps are generally used to handle sewage. For unscreened sewage, the suction inlet of the pump should not be less than 3 in. (76 mm) in diameter.

New Developments. Water supply and storm-water system design are at the center of great technological and regulatory change. Engineering is being reinvented to meet changing demands, focusing on environmental impact and on securing water supplies and facilities against acts of terrorism or natural disasters.

There is greater emphasis today on advanced storm-water systems and risk assessment for water reuse. New productivity-enhancing water treatment methodologies include membrane filtration, heat drying, UV disinfection, zero discharge (ZD) design, and the application of Geographic Information Systems (GIS) to water supply and storm-water engineering.

To harden water facilities against potential terrorist acts, engineers are developing new, more secure water supply systems, as well as impact-resistant and fire-resistant materials as part of an overall safe design against impact and heat damage.

Engineers have developed a better understanding of existing operations and processes. New best practices have been developed for engineering, building, and running effective public water utilities. Particular attention is paid to upgrading storm-water system design and performance of existing, outdated systems. In addition, engineers today receive a better education in the vital fields of engineering economics and finance.

Further considerations emerging in recent years include storm-water system design for changing urban environments and the design of berms and levees to withstand worst-case events such as damaging hurricanes and severe flooding.

Engineers today have greater concern for the long-term health and environmental impacts of water systems. Most of these changes have come about out of necessity. Hence, there is increasing emphasis on the application of advanced tools such as GIS and ZD design, which makes this a challenging and rewarding field to be in.