CHAPTER

2

GENERAL WATER SUPPLY DESIGN CONSIDERATIONS

2-1 WATER DEMAND

A fundamental prerequisite to begin the design of water supply facilities is a determination of the design capacity. This, in turn, is a function of water demand. The determination of water demand consists of four parts: (1) selection of a design period, (2) estimation of the population, commercial, and industrial growth, (3) estimation of the unit water use, and (4) estimation of the variability of the demand.

Design Period

The *design period* (also called the *design life*) is not the same as the *life expectancy*. The design period is the length of time it is estimated that the facility will be able to meet the demand, that is, the *design capacity*. The life expectancy of a facility or piece of equipment is determined by wear and tear. Typical life expectancies for equipment range from 10 to 20 years. Buildings, other structures, and pipelines are assumed to have a useful life of 50 years or more.

New water works are generally made large enough to meet the demand for the future. The number of years selected for the design period is based on the following:

- Regulatory constraints.
- The rate of population growth.
- The interest rate for bonds.
- The useful life of the structures and equipment.
- The ease or difficulty of expansion.
- Performance in early years of life under minimum hydraulic load.

Because state and federal funds are often employed in financing water works, their requirements for establishing the design period often govern the selection of the design period. This time period may be substantially less than the useful life of the plant.

Because of their need for population data and forecast estimates for numerous policy decisions, local government entities in the United States generally have the requisite information for water works planning. In the absence of this data, U.S. census data may be used. Historic records provide a basis for developing trend lines and making forecasts of future growth. For short-range forecasts on the order of 10 to 15 years, data extrapolation is of sufficient accuracy for planning purposes. For long-range forecasts on the order of 15 to 50 years, more sophisticated techniques are required. These methods are beyond the scope of this book. McJunkin (1964) provides a comprehensive discussion of alternative methods for developing a population growth projection estimate.

Although all of the indicators mentioned above may lead to the conclusion that a long design period is favored, serious consideration must be given to the impact of low flow rates in the early years of the facility. In addition to the behavior and efficiency of the unit operations, the impact on the energy efficiency of the equipment should be evaluated. A successful alternative is the use of modular units and construction of hardened facilities without installation of mechanical equipment until the units are needed.

Design periods that are commonly employed in practice and commonly experienced life expectancies are shown in Table 2-1.

TABLE 2-1
Design periods for water works

Type of facility	Characteristics	Design period, y	Life expectancy, y
Large dams and pipelines	Difficult and expensive to enlarge	40–60	100+
Wells	Easy to refurbish/replace	15–25	25+
Treatment plants			
Fixed facilities	Difficult and expensive to enlarge/replace	20–25	50+
Equipment	Easy to refurbish/replace	10–15	10–20
Distribution systems			
Mains > 60 cm	Replacement is expensive and difficult	20–25	60+
Laterals and mains ≤ 30 cm	Easy to refurbish/replace	To full development[a]	40–50

[a]Full development (also called *build-out*) means that the land area being serviced is completely occupied by houses and/or commercial and institutional facilities.

Unit Water Use

When the proposed project is in a community with an existing community supply, the community's historic records provide the best estimate of water use. Conversion of total demand to per capita demand (liters per capita per day, Lpcd) allows for the separation of population growth from the growth in unit consumption. If the proposed project is to improve the water quality, consideration should be given to the likelihood that unit demand will increase because of the improved water quality. In the absence of existing data for the client community, nearby communities with similar demographics are a good alternative source. When the demographics differ in some particular aspect such as a higher or lower density of commercial facilities or a major industrial component, adjustment in the total demand will be appropriate. Although they were developed for wastewater flow rates, Tables 2-2 and 2-3 can provide a basis for adjustment of commercial and institutional users. Likewise, flow rates for recreational facilities may be estimated using Table 2-4 on page 2-6.

Community adoption of the use of one or more flow-reduction devices such as flow-limiting shower heads and low-flush toilets may have a substantial impact on per capita consumption. Typical results are shown in Table 2-5 on page 2-7. The implementation of requirements for water saving devices conserves both water resources and energy. These aspects should be addressed in strategic planning for community development as well as requirements for new or expanded facilities.

Gross estimates of unit demand may be made using statewide data. Hutson et al. (2001) have estimated water use by state and the U. S. Bureau of Census (Census, 2006) maintains a population database by state. Great care should be used in making estimates from generalized data. Due consideration must be given to the following local factors that modify gross estimates:

1. Climate
2. Industrial activity
3. Meterage

TABLE 2-2
Typical wastewater flow rates from commercial sources in the United States

		Flow rate, L/unit · d	
Source	Unit	Range	Typical
Airport	Passenger	10–20	15
Apartment	Bedroom	380–570	450
Automobile service station	Vehicle	30–60	40
	Employee	35–60	50
Bar/cocktail lounge	Seat	45–95	80
	Employee	40–60	50
Boarding house	Person	95–250	170
Conference center	Person	40–60	30
Department store	Restroom	1,300–2,300	1,500
	Employee	30–60	40
Hotel	Guest	150–230	190
	Employee	30–60	40
Industrial building (sanitary wastewater only)	Employee	60–130	75
Laundry (self-service)	Machine	1,500–2,100	1,700
	Customer	170–210	190
Mobile home park	Mobile home	470–570	530
Motel with kitchen	Guest	210–340	230
Motel without kitchen	Guest	190–290	210
Office	Employee	25–60	50
Public restroom	User	10–20	15
Restaurant without bar	Customer	25–40	35
Restaurant with bar	Customer	35–45	40
Shopping center	Employee	25–50	40
	Parking space	5–10	8
Theater	Seat	10–15	10

Adapted from Metcalf and Eddy, 2003.

4. System management
5. Standard of living

The extent of sewerage, system pressure, water price, water loss, age of the community, and availability of private wells also influence water consumption but to a lesser degree.

Climate is the most important factor influencing unit demand. This is shown dramatically in Table 2-6 on page 2-7. The average annual precipitation for the "wet" states is about 100 cm per year,

TABLE 2-3
Typical wastewater flow rates from institutional sources in the United States

		Flow rate, L/unit · d	
Source	Unit	Range	Typical
Assembly hall	Guest	10–20	15
Hospital	Bed	660–1,500	1,000
	Employee	20–60	40
Prison	Inmate	300–570	450
	Employee	20–60	40
School[a]			
With cafeteria, gym, and showers	Student	60–120	100
With cafeteria only	Student	40–80	60
School, boarding	Student	280–380	320

[a]Flow rates are L/unit-school day.
Adapted from Metcalf and Eddy, 2003.

while the average annual precipitation for the "dry" states is only about 25 cm per year. Of course, the dry states are also considerably warmer than the wet states.

The *influence of industry* is to increase average per capita water demand. Small rural and suburban communities will use less water per person than industrialized communities. Tables 2-2 and 2-3 can provide a basis for adjustment for commercial and institutional users.

The third most important factor in water use is whether individual consumers have water meters. *Meterage* imposes a sense of responsibility not found in unmetered residences and businesses. This sense of responsibility reduces per capita water consumption because customers repair leaks and make more conservative water-use decisions almost regardless of price. Because water is so inexpensive, price is not much of a factor.

Following meterage closely is the aspect called *system management.* If the water distribution system is well managed, per capita water consumption is less than if it is not well managed. Well-managed systems are those in which the managers know when and where leaks in the water main occur and have them repaired promptly.

Climate, industrial activity, meterage, and system management are more significant factors controlling water consumption than *standard of living.* The rationale for the last factor is straightforward. Per capita water use increases with an increased standard of living. Highly developed countries use much more water than less developed nations. Likewise, higher socioeconomic status implies greater per capita water use than lower socioeconomic status.

For a community supply system that includes a new treatment plant and a new distribution system, water loss through leaks is not a major factor in estimating demand. For a new plant with an existing old distribution system, water loss through leaks may be a major consideration.

Older communities that lack modern water saving devices will use more water than newer communities with building codes that require water saving devices. For example, modern water closets use about 6 L per flush compared to older systems that use about 18 L per flush.

TABLE 2-4
Typical wastewater flow rates from recreational facilities in the United States

		Flow rate, L/unit · d	
Facility	Unit	Range	Typical
Apartment, resort	Person	190–260	230
Cabin, resort	Person	30–190	150
Colateria	Customer	10–15	10
	Employee	30–45	40
Camp:			
With toilets only	Person	55–110	95
With central toilet and bath facilities	Person	130–90	170
Day	Person	55–75	60
Cottages, (seasonal with private bath)	Person	150–230	190
Country club	Member present	75–150	100
	Employee	40–60	50
Dining hall	Meal served	15–40	25
Dormitory, bunkhouse	Person	75–190	150
Playground	Visitor	5–15	10
Picnic park with flush toilets	Visitor	20–40	20
Recreational vehicle park:			
With individual connection	Vehicle	280–570	380
With comfort station	Vehicle	150–190	170
Roadside rest areas	Person	10–20	15
Swimming pool	Customer	20–45	40
	Employee	30–45	40
Vacation home	Person	90–230	190
Visitor center	Visitor	10–20	15

Adapted from Metcalf and Eddy, 2003.

The total U.S. water withdrawal for all uses (agricultural, commercial, domestic, mining, and thermoelectric power) including both fresh and saline water was estimated to be approximately 5,400 liters per capita per day (Lpcd) in 2000 (Hutson et al., 2001). The amount for U.S. public supply (domestic, commercial, and industrial use) was estimated to be 580 Lpcd in 2000 (Hutson et al., 2001). The American Water Works Association estimated that the average daily household water use in the United State was 1,320 liters per day in 1999 (AWWA, 1999). For a family of three, this would amount to about 440 Lpcd.

Variability of Demand

The unit demand estimates are averages. Water consumption changes with the seasons, the days of the week, and the hours of the day. Fluctuations are greater in small than in large communities, and during short rather than long periods of time (Fair et al., 1970). The variation in demand is normally reported as a factor of the average day. For metered dwellings the U. S. national

TABLE 2-5
Typical changes in water consumption with use of water saving devices

Use	Without water conservation, Lpcd	With water conservation, Lpcd
Showers	50	42
Clothes washing	64	45
Toilets	73	35

Source: AWWA, 1998.

average factors are as follows: maximum day = 2.2 × average day; peak hour = 5.3 × average day (Linaweaver et al., 1967). Figure 2-1 provides an alternative method of estimating the variability. As noted above, when the proposed project is in a community with an existing community supply, the community's historic records provide the best estimate of water use. This includes its variability. The demand for water for fire fighting is normally satisfied by providing storage.

The *Recommended Standards for Water Works* (GLUMRB, 2003) stipulates that the design basis for the water source and treatment facilities shall be for the maximum day demand at the design year. Pumping facilities and distribution system piping are designed to carry the peak hour flow rate.When municipalities provide water for fire protection, the maximum day demand plus fire demand is used to estimate the peak hour flow rate.

2-2 WATER SOURCE EVALUATION

Although the portion of the population of the United States supplied by surface water is 150 percent of that supplied by groundwater, the number of communities supplied by groundwater is more than a factor of 10 times that supplied by surface water (Figure 2-2 on page 2-9). The reason for this

TABLE 2-6
Total fresh water withdrawals for public supply

State	Withdrawal, Lpcd[a]
Wet	
Connecticut	471
Michigan	434
New Jersey	473
Ohio	488
Pennsylvania	449
Average	463
Dry	
Nevada	1,190
New Mexico	797
Utah	1,083
Average	963

Compiled from Hutson et al. (2001).
[a]Lpcd = liters per capita per day.

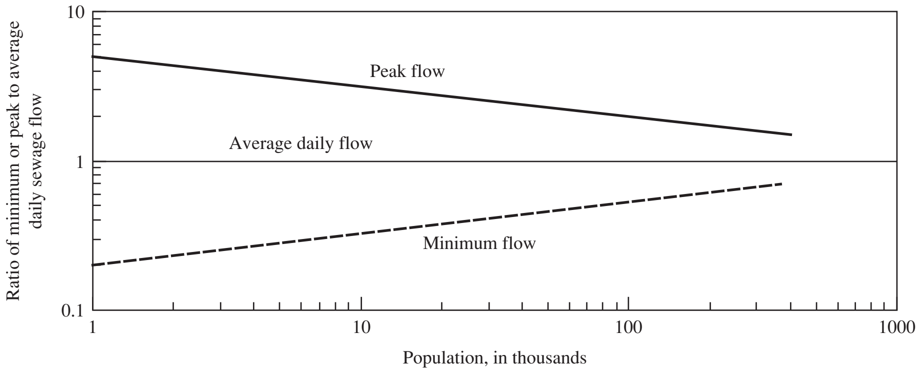

FIGURE 2-1
Ratio of extreme flows to average daily flow

pattern is that larger cities are supplied by large surface water bodies while many small communities use groundwater.

Groundwater has many characteristics that make it preferable as a water supply. First, groundwater is less subject to seasonal fluctuations and long-term droughts. Second, the aquifer provides natural storage that eliminates the need for an impoundment. Third, because the groundwater source is frequently available near the point of demand, the cost of transmission is reduced significantly. Fourth, because natural geologic materials filter the water, groundwater is often more aesthetically pleasing and to some extent protected from contamination.

Groundwater as a supply is not without drawbacks. It dissolves naturally occurring minerals which may give the water undesirable characteristics such as hardness, red color from iron oxidation, and toxic contaminants like arsenic.

Yield

One of the first considerations in selecting a water supply source is the ability of the source to provide an adequate quantity of water. One measure of quantity is yield. *Yield* is the average flow available over a long period of time.

Surface Water

When the proposed surface water supply is to be the sole source of water, the design basis is the long-term or "safe" yield. The components of the design are: (1) determination of the allowable withdrawal, (2) completion of a complete series analysis and, if the design drought duration exceeds the recorded data interval, completion of a partial duration series analysis, and (3) completion of an extreme-value analysis to determine the probable *recurrence interval* (*return period*) of a drought.

The allowable withdrawal is determined from regulatory constraints. Obviously, the municipality desiring to use the surface water for supply cannot withdraw all of the available water. Enough must be left for the ecological health of the river or stream as well as for downstream users.

In some cases, such as the Great Lakes, the water body is so large that the classic analysis of drought conditions is not warranted. However, the fluctuation of the lake level does impact the design of the intake structure, and it must be evaluated.

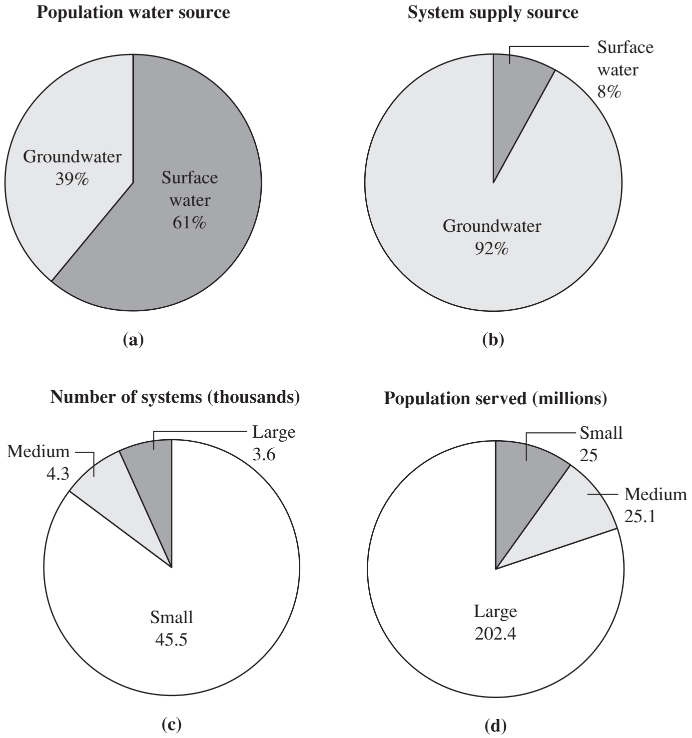

FIGURE 2-2
(*a*) Percentage of the population served by drinking-water system source. (*b*) Percentage of drinking-water systems by supply source. (*c*) Number of drinking-water systems (in thousands) by size. (*d*) Population served (in millions of people) by drinking-water system size.

Source: 1997 National Public Water Systems Compliance Report. U.S. EPA, Office of Water. Washington, D.C. 20460. (EPA-305-R-99-002).
(*Note:* Small systems serve 25-3,300 people; medium systems serve 3301–10,000 people; large systems serve 10,000 + people.)

Complete Series. A complete series analysis is used to construct a flow-duration curve. This curve is used to determine whether or not the long-term average flow exceeds the long-term average demand. All of the observed data are used in a complete series analysis. This analysis is usually presented in one of two forms: as a *yield curve* (also known as a *duration curve,* Figure 2-3) or as a cumulative probability distribution function (CDF). In either form the analysis shows the percent of time that a given flow will be equaled or exceeded. The percent of time is interpreted as the probability that a watershed will yield a given flow over a long period of time. Thus, it is sometimes called a *yield analysis.*

To perform a yield analysis, discharge data are typed into a spreadsheet in the order of their occurrence. Using the spreadsheet "sort" function, the data are arranged in descending order of flow rate. The percent of time each value is equaled or exceeded is calculated. The spreadsheet is then used to create the duration curve: a plot of the discharge versus the percent of time the discharge is exceeded. This is demonstrated in Example 2-1 using the data in Table 2-7.

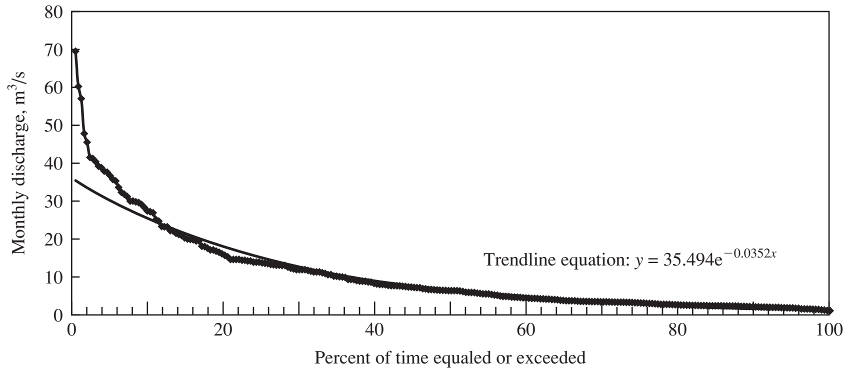

FIGURE 2-3
Complete series analysis for Troublesome Creek at Nosleep.

Example 2-1. Perform a complete series analysis on the Troublesome Creek data in Table 2-7. Determine the mean monthly discharge. If the average demand for Nosleep's municipal supply is 0.25 m^3/s, and the regulatory agency will permit a withdrawal of 5% of the flow, will the Troublesome Creek provide a safe yield?

Solution. A spreadsheet was used to perform the calculations and plot the duration curve. The first few values in the spreadsheet are shown below. A sample calculation for the spreadsheet is shown below the spreadsheet.

Rank	Monthly discharge, m^3/s	% of time equaled or exceeded
1	69.1	0.38
2	59.8	0.76
3	56.6	1.14
4	47.4	1.52
5	45.1	1.89
6	41.1	2.27
7	40.8	2.65

There are 264 values in the table (12 months/year × 22 years of data). The highest discharge in the table is 69.1 m^3/s. It is assigned a rank of 1.

The percent of time this value is equaled or exceeded is:

$$\% \text{ of time} = \frac{1}{264}(100\%) = 0.38\%$$

The plot of the duration curve is shown in Figure 2-3. From the data sort, find that the flow rate that is exceeded 50% of the time is 5.98 m^3/s.

TABLE 2-7
Average monthly discharge of Troublesome Creek at Nosleep, m^3/S

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1987	2.92	5.10	1.95	4.42	3.31	2.24	1.05	0.74	1.02	1.08	3.09	7.62
1988	24.3	16.7	11.5	17.2	12.6	7.28	7.53	3.03	10.2	10.9	17.6	16.7
1989	15.3	13.3	14.2	36.3	13.5	3.62	1.93	1.83	1.93	3.29	5.98	12.7
1990	11.5	4.81	8.61	27.0	4.19	2.07	1.15	2.04	2.04	2.10	3.12	2.97
1991	11.1	7.90	41.1	6.77	8.27	4.76	2.78	1.70	1.46	1.44	4.02	4.45
1992	2.92	5.10	28.7	12.2	7.22	1.98	0.91	0.67	1.33	2.38	2.69	3.03
1993	7.14	10.7	9.63	21.1	10.2	5.13	3.03	10.9	3.12	2.61	3.00	3.82
1994	7.36	47.4	29.4	14.0	14.2	4.96	2.29	1.70	1.56	1.56	2.04	2.35
1995	2.89	9.57	17.7	16.4	6.83	3.74	1.60	1.13	1.13	1.42	1.98	2.12
1996	1.78	1.95	7.25	24.7	6.26	8.92	3.57	1.98	1.95	3.09	3.94	12.7
1997	13.8	6.91	12.9	11.3	3.74	1.98	1.33	1.16	0.85	2.63	6.49	5.52
1998	4.56	8.47	59.8	9.80	6.06	5.32	2.14	1.98	2.17	3.40	8.44	11.5
1999	13.8	29.6	38.8	13.5	37.2	22.8	6.94	3.94	2.92	2.89	6.74	3.09
2000	2.51	13.1	27.9	22.9	16.1	9.77	2.44	1.42	1.56	1.83	2.58	2.27
2001	1.61	4.08	14.0	12.8	33.2	22.8	5.49	4.25	5.98	19.6	8.50	6.09
2002	21.8	8.21	45.1	6.43	6.15	10.5	3.91	1.64	1.64	1.90	3.14	3.65
2003	8.92	5.24	19.1	69.1	26.8	31.9	7.05	3.82	8.86	5.89	5.55	12.6
2004	6.20	19.1	56.6	19.5	20.8	7.73	5.75	2.95	1.49	1.69	4.45	4.22
2005	15.7	38.4	14.2	19.4	6.26	3.43	3.99	2.79	1.79	2.35	2.86	10.9
2006	21.7	19.9	40.0	40.8	11.7	13.2	4.28	3.31	9.46	7.28	14.9	26.5
2007	31.4	37.5	29.6	30.8	11.9	5.98	2.71	2.15	2.38	6.03	14.2	11.5
2008	29.2	20.5	34.9	35.3	13.5	5.47	3.29	3.14	3.20	2.11	5.98	7.62

The regulatory agency will only permit a withdrawal of 5%. Therefore, the allowable withdrawal will be

$$(0.05)(5.98\ \text{m}^3/\text{s}) = 0.30\ \text{m}^3/\text{s}$$

This is sufficient to meet the safe yield of 0.25 m^3/s required for the municipality.

If the determination is made that the 50 percentile allowable withdrawal is less than the required safe yield, then, even with storage, the safe yield cannot be met. An alternative source should be investigated. If the determination is made that the allowable withdrawal will be adequate, then an analysis is performed to determine the need for a storage reservoir for droughts. This analysis is called an *annual series* or *extreme-value* analysis.

Annual Series. Extreme-value analysis is a probability analysis of the largest or smallest values in a data set. Each of the extreme values is selected from an equal time interval. For example, if the largest value in each year of record is used, the extreme-value analysis is called an *annual maxima series.* If the smallest value is used, it is called an *annual minima series.*

Because of the climatic effects on most hydrologic phenomena, a water year or *hydrologic year* is adopted instead of a calendar year. The U.S. Geological Survey (U.S.G.S.) has adopted the 12-month period from October 1 to September 30 as the hydrologic year for the United States. This period was chosen for two reasons: "(1) to break the record during the low-water period near the end of the summer season, and (2) to avoid breaking the record during the winter, so as to eliminate computation difficulties during the ice period." (Boyer, 1964)

The procedure for an annual maxima or minima analysis is as follows:

1. Select the minimum or maximum value in each 12-month interval (October to September) over the period of record.
2. Rank each value starting with the highest (for annual maxima) or lowest (for annual minima) as rank number one.
3. Compute a return period using Weibull's formula (Weibull, 1939):

$$T = \frac{n+1}{m} \tag{2-1}$$

where T = average return period, y
n = number of years of record
m = rank of storm or drought

4. Plot the annual maxima or minima series on a special probability paper known as Gumbel paper. (A blank copy of Gumbel paper may be downloaded from the website: http//www.mhhe.com/davis1e.) Although the same paper may be used for annual minima series, Gumbel recommends a log extremal probability paper (axis of ordinates is log scale) for droughts (Gumbel, 1954).

From the Gumbel plot, the return period for a flood or drought of any magnitude may be determined. Conversely, for any magnitude of flood or drought, one may determine how frequently it will occur.

In statistical parlance a Gumbel plot is a linearization of a Type I probability distribution. The logarithmically transformed version of the Type I distribution is called a log-Pearson Type III distribution. The return period of the mean (X) of the Type I distribution occurs at $T = 2.33$ years. Thus, the U.S.G.S. takes the return period of the mean annual flood or drought to be 2.33 years. This is marked by a vertical dashed line on Gumbel paper (Figure 2-4).

The data in Table 2-8 were used to plot the annual minima line in Figure 2-4. The computations are explained in Example 2-2.

Example 2-2. In continuing the evaluation of the Troublesome Creek as a water supply for Nosleep (Example 2-1), perform an annual minima extreme-value analysis on the data in Table 2-7. Determine the recurrence interval of monthly flows that fail to meet an average demand of 0.31 m^3/s. Also determine the discharge of the mean monthly annual minimum flow.

Solution. To begin, select the minimum discharge in each hydrologic year. The first nine months of 1987 and the last three months of 2008 cannot be used because they are not complete hydrologic years. After selecting the minimum value in each year, rank the data and compute the return period. The 1988 water year begins in October 1987.

The computations are summarized in Table 2-8. The return period and flows are plotted in Figure 2-4. From Figure 2-4 find that for the 22 years of record, the minimum flow exceeds a demand of 0.31 m^3/s and that the mean monthly minimum flow is about 1.5 m^3/s.

However, as noted in Example 2-1, the regulatory agency will only permit removal of 5% of the flow. The fifth column in Table 2-8 shows the computation of 5% of the flow. Obviously, storage must be provided if the Troublesome Creek is to be used as a water source.

Partial-Duration Series. It often happens that the second largest or second smallest flow in a water year is larger or smaller than the maxima or minima from a different hydrologic year. To take these events into consideration, a partial series of the data is examined. The procedure for performing a partial-duration series analysis is very similar to that used for an annual series. The

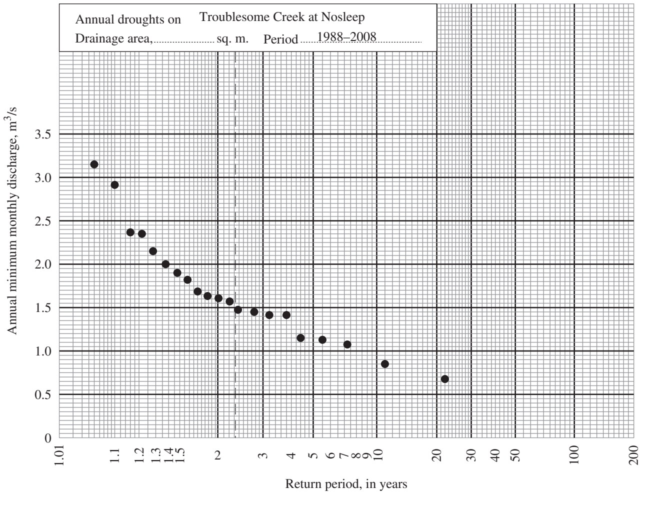

FIGURE 2-4
Annual droughts on Troublesome Creek at Nosleep.

TABLE 2-8
Tabulated computations of annual minima for Troublesome Creek at Nosleep

Year	Annual minima, m^3/s	Rank	Return period, years	5% of Minima, m^3/s
1992	0.67	1	22.00	0.034
1997	0.85	2	11.00	0.043
1988	1.08	3	7.33	0.054
1995	1.13	4	5.50	0.057
1990	1.15	5	4.40	0.058
1996	1.42	6	3.67	0.071
2000	1.42	7	3.14	0.071
1991	1.46	8	2.75	0.073
2004	1.49	9	2.44	0.075
1994	1.56	10	2.20	0.078
2001	1.61	11	2.00	0.081
2002	1.64	12	1.83	0.082
2005	1.69	13	1.69	0.085
1989	1.83	14	1.57	0.092
2003	1.90	15	1.47	0.099
1998	1.98	16	1.38	0.099
2007	2.15	17	1.29	0.108
2006	2.35	18	1.22	0.118
1993	2.38	19	1.16	0.119
1999	2.92	20	1.10	0.146
2008	3.14	21	1.05	0.157

theoretical relationship between an annual series and partial series is shown in Table 2-9. The partial series is approximately equal to the annual series for return periods greater than 10 years (Langbein, 1949).

If the time period over which the event occurs is also taken into account, the analysis is termed a *partial-duration series.* While it is fairly easy to define a flood as *any* flow that exceeds the capacity of the drainage system, in order to properly define a drought, one must specify the low flow and its duration. For example, if a roadway is covered with water for 10 minutes, one can say that it is flooded. In contrast, if the flow in a river is below our demand for 10 minutes, one certainly would not declare it a drought! Thus, a partial-duration series is particularly relevant for low-flow conditions.

Low-Flow Duration. From an environmental engineering point of view, three low-flow durations are of particular interest. The 10-year return period of seven days of low flow has been selected by many states as the critical flow for water pollution control. Wastewater treatment plants must be designed to provide sufficient treatment to allow effluent discharge without driving the quality of the receiving stream below the standard when the dilution capacity of the stream is at a 10-year low.

A longer duration low flow and longer return period are selected for water supply. In the Midwest, drought durations of 1 to 5 years and return periods of 25 to 50 years are used in the

TABLE 2-9
Theoretical relationship between partial series and annual series return periods in years

Partial series	Annual series
0.5	1.18
1.0	1.58
1.45	2.08
2.0	2.54
5.0	5.52
10.0	10.5
50.0	50.5
100.0	100.5

Source: W. B. Langbein, "Annual Floods and Partial Duration Series," *Transactions of the American Geophysical Union,* vol. 30, pp. 879–881, 1949.

design of water-supply reservoirs. Where water supply is by direct draft (withdrawal) from a river, the duration selected may be on the order of 30 to 90 days with a 10-year return period. When alternate sources are available, shorter return periods may be acceptable.

When to Use Which Series. The probability of occurrence (l/T) computed from an annual series will not be the same as that found from a complete series. There are many reasons for this difference. Among the most obvious is the fact that, in computing an annual series, 1/12 of the data is treated as if it were all of the data when, in fact, it is not even a representative sample. It is only the extreme end of the possible range of values.

The following guidelines can be used to decide when to use which analysis:

1. Use a complete series to determine the long-time reliability (yield) for water supply.
2. Use an annual minima series to determine the need for storage.
3. Use a partial-duration series to predict low-flow conditions.

In practice the complete-series analysis can be performed to decide whether or not it is worth doing a partial series for water supply. If the complete series indicates that the mean monthly flow will not supply the demand, then computation of an annual minima series to determine the need for storage is not worth the trouble, since it would be impossible to store enough water to meet the demand.

Volume of Reservoirs. The techniques for determining the storage volume required for a reservoir are dependent both on the size and use of the reservoir. This discussion is limited to the simplest procedure, which is quite satisfactory for small water-supply impoundments.* It is called the *mass diagram* or *Rippl method* (Rippl, 1883). The main disadvantage of the Rippl method is

*It is also useful for sizing storm-water retention ponds and wastewater equalization basins.

that it assumes that the sequence of events leading to a drought or flood will be the same in the future as it was in the past. More sophisticated techniques have been developed to overcome this disadvantage. These techniques are left for advanced hydrology classes.

The Rippl procedure for determining the storage volume is an application of the mass balance method of analyzing problems. In this case it is assumed that the only input is the flow into the reservoir (Q_{in}) and that the only output is the flow out of the reservoir (Q_{out}). Therefore,with the assumption that the density term cancels out because the change in density across the reservoir is negligible,

$$\frac{dS}{dt} = \frac{d(\text{In})}{dt} - \frac{d(\text{Out})}{dt} \tag{2-2}$$

becomes

$$\frac{dS}{dt} = Q_{\text{in}} - Q_{\text{out}} \tag{2-3}$$

If both sides of the equation are multiplied by *dt,* the inflow and outflow become volumes (flow rate × time = volume), that is,

$$dS = (Q_{\text{in}})(dt) - (Q_{\text{out}})(dt) \tag{2-4}$$

By substituting finite time increments (Δt), the change in storage is then

$$(Q_{\text{in}})(\Delta t) - (Q_{\text{out}})(\Delta t) = \Delta S \tag{2-5}$$

By cumulatively summing the storage terms, the size of the reservoir can be estimated. For water supply reservoir design, Q_{out} is the demand, and zero or positive values of storage (ΔS) indicate there is enough water to meet the demand. If the storage is negative, then the reservoir must have a capacity equal to the absolute value of cumulative storage to meet the demand. This is illustrated in the following example.

Example 2-3. Using the data in Table 2-7, determine the storage required to meet Nosleep's demand of 0.25 m^3/s for the period from August 1994 through January 1997.

Solution. The computations are summarized in the table below.

Month	Q_{in} (m^3/s)	$(0.05)(Q_{in})$ (m^3/s)	$(0.05)(Q_{in})(\Delta t)$ (10^6 m^3)	Q_{out} (m^3/s)	$Q_{out}(\Delta t)$ (10^6 m^3)	ΔS (10^6 m^3)	$\Sigma(\Delta S)$ (10^6 m^3)
1994							
Aug	1.70	0.085	0.228	0.25	0.670	−0.442	−0.442
Sep	1.56	0.078	0.202	0.25	0.648	−0.446	−0.888
Oct	1.56	0.078	0.209	0.25	0.670	−0.461	−1.348
Nov	2.04	0.102	0.264	0.25	0.648	−0.384	−1.732
Dec	2.35	0.1175	0.315	0.25	0.670	−0.355	−2.087

(continued)

1995							
Jan	2.89	0.1445	0.387	0.25	0.670	−0.283	−2.370
Feb	9.57	0.4785	1.158	0.25	0.605	0.553	−1.817
Mar	17.7	0.885	2.370	0.25	0.670	1.701	−0.166
Apr	16.4	0.82	2.125	0.25	0.648	1.477	
May	6.83	0.3415	0.915	0.25	0.670	0.245	
Jun	3.74	0.187	0.485	0.25	0.648	−0.163	−0.163
Jul	1.60	0.08	0.214	0.25	0.670	−0.455	−0.619
Aug	1.13	0.0565	0.151	0.25	0.670	−0.518	−1.137
Sep	1.13	0.0565	0.146	0.25	0.648	−0.502	−1.638
Oct	1.42	0.071	0.190	0.25	0.670	−0.479	−2.118
Nov	1.98	0.099	0.257	0.25	0.648	−0.391	−2.509
Dec	2.12	0.106	0.284	0.25	0.670	−0.386	−2.895
1996							
Jan	1.78	0.089	0.238	0.25	0.670	−0.431	−3.326
Feb	1.95	0.0975	0.236	0.25	0.605	−0.369	−3.695
Mar	7.25	0.3625	0.971	0.25	0.670	0.301	−3.394
Apr	24.7	1.235	3.201	0.25	0.648	2.533	−0.841
May	6.26	0.313	0.838	0.25	0.670	0.169	−0.672
Jun	8.92	0.446	1.156	0.25	0.648	0.508	−0.164
Jul	3.57	0.1785	0.478	0.25	0.670	−0.192	−0.355
Aug	1.98	0.099	0.265	0.25	0.670	−0.404	−0.760
Sep	1.95	0.0975	0.253	0.25	0.648	−0.395	−1.155
Oct	3.09	0.1545	0.414	0.25	0.670	−0.256	−1.411
Nov	3.94	0.197	0.511	0.25	0.648	−0.137	−1.548
Dec	12.7	0.635	1.701	0.25	0.670	1.031	−0.517
1997							
Jan	13.8	0.69	1.848	0.25	0.670	1.178	

The data in the first and second columns of the table were extracted from Table 2-7. The third column is the product of the second column and the regulatory restriction of 5%. The fourth column is the product of the second column and the time interval for the month. For example, for August (31 d) and September (30 d), 1994:

$$(0.085 \text{ m}^3/\text{s})(31 \text{ d})(86{,}400 \text{ s/d}) = 227{,}664 \text{ m}^3$$
$$(0.078 \text{ m}^3/\text{s})(30 \text{ d})(86{,}400 \text{ s/d}) = 202{,}176 \text{ m}^3$$

The fifth column is the demand given in the problem statement.

The sixth column is the product of the demand and the time interval for the month. For example, for August and September 1994:

$$(0.25 \text{ m}^3/\text{s})(31 \text{ d})(86{,}400 \text{ s/d}) = 669{,}600 \text{ m}^3$$
$$(0.25 \text{ m}^3/\text{s})(30 \text{ d})(86{,}400 \text{ s/d}) = 648{,}000 \text{ m}^3$$

The seventh column (ΔS) is the difference between the third and fifth columns. For example, for August and September 1994:

$$227{,}664 \text{ m}^3 - 669{,}600 \text{ m}^3 = -441{,}936 \text{ m}^3$$
$$202{,}176 \text{ m}^3 - 648{,}000 \text{ m}^3 = -445{,}824 \text{ m}^3$$

The last column($\Sigma(\Delta S)$) is the sum of the last value in that column and the value in the sixth column. For August 1994, it is $-441{,}936$ m^3 since this is the first value.

For September 1994, it is

$$(-441{,}936 \text{ m}^3) + (-445{,}824 \text{ m}^3) = -887{,}760 \text{ m}^3$$

The following logic is used in interpreting the table. From August 1994 through March 1995, the demand exceeds the flow, and storage must be provided. The maximum storage required for this interval is 2.370×10^6 m^3. In April 1995, the storage (ΔS) exceeds the deficit ($\Sigma(\Delta S)$)from March 1995. If the deficit is viewed as the volume of water in a virtual reservoir with a total capacity of 2.370×10^6 m^3, then in March 1995, the volume of water in the reservoir is 2.204×10^6 m^3 ($2.370 \times 10^6 - 0.166 \times 10^6$). The April 1995 inflow exceeds the demand and fills the reservoir deficit of 0.166×10^6 m^3.

Because the inflow (Q_{in}) exceeds the demand (0.25 m^3/s) for the months of April and May 1995, no storage is required during this period. Therefore, no computations were performed.

From June 1995 through December 1996, the demand exceeds the inflow, and storage is required. The maximum storage required is 3.695×10^6 m^3. Note that the computations for storage did not stop in May 1996, even though the inflow exceeded the demand. This is because the storage was not sufficient to fill the reservoir deficit. The storage was sufficient to fill the reservoir deficit in January 1997.

Comment. These tabulations are particularly well suited to spreadsheet programs.

The storage volume determined by the Rippl method must be increased to account for water lost through evaporation and volume lost through the accumulation of sediment.

Groundwater

Unlike surface water supplies, groundwater is less subject to seasonal fluctuations and long-term droughts. The design basis is the long term or "safe" yield. The safe yield of a ground water basin is the amount of water which can be withdrawn from it annually without producing an undesired result. (Todd, 1959) A yield analysis of the aquifer is performed because of the potential for overpumping the well with consequent failure to yield an adequate supply as well as the potential to cause dramatic ground surface settlement, detrimental dewatering of nearby ponds or streams or, in wells near the ocean, to cause salt water intrusion.

Confined Aquifer. The components of the evaluation of the aquifer as a water supply are: (1) depth to the bottom of the aquiclude, (2) elevation of the existing piezometric surface, (3) drawdown for sustained pumping at the design rate of demand, and (4) recharge and drought implications.

The depth to the bottom of the aquiclude (Figure 2-5) sets the limit of drawdown of the piezometric surface. If the piezometric surface drops below the bottom of the aquiclude, ground settlement will begin to occur and, in addition to structural failure of the well, structural damage will occur to buildings and roadways. In populated areas of the United States, regulatory agencies gather hydrogeologic data reported by well drillers and others that may be used to estimate the depth to the aquiclude. In less densely populated areas, exploration and evaluation by a professional hydrogeologist is required.

The existing piezometric surface sets the upper bound of the range of drawdown. That is, the difference between the existing piezometric surface and the bottom of the aquiclude (s_{max} in Figure 2-5) is the maximum allowable drawdown for a safe yield. As noted above, in populated areas, regulatory agencies will have a database that includes this information. Otherwise, a hydrogeologic exploration will be required.

Drawdown Estimation. The derivation of equations relating well discharge to water-level drawdown and the hydraulic properties of the aquifer is based on the following assumptions (Bouwer, 1978):

1. The well is pumped at a constant rate.
2. Flow toward the well is radial and uniform.
3. Initially the piezometric surface is horizontal.
4. The well fully penetrates the aquifer and is screened for the entire length.
5. The aquifer is homogeneous, isotropic, horizontal, and of infinite horizontal extent.
6. Water is released from the aquifer in immediate response to a drop in the piezometric surface.

Although the steady state will seldom occur in practice, it may be approached after prolonged pumping when the piezometric surface declines at a very slow rate. The Thiem equation

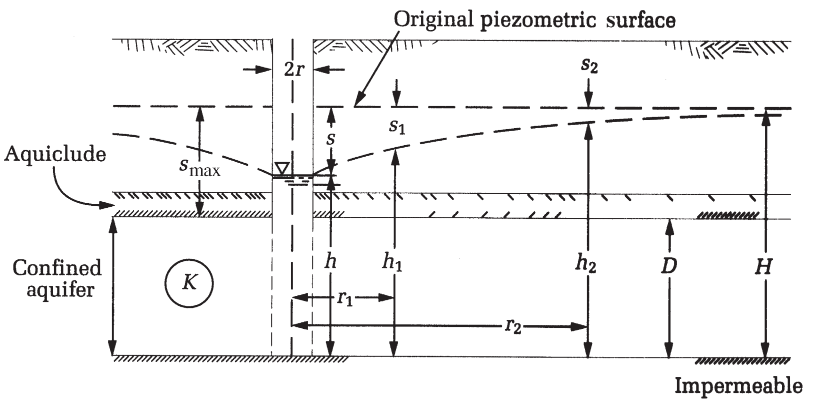

FIGURE 2-5
Geometry and symbols for a pumped well in a confined aquifer. (*Source:* H. Bouwer, 1978.)

may be used to estimate the maximum pumping rate that can be sustained by a single well in a confined aquifer (Thiem, 1906):

$$Q = \frac{2\pi T(h_2 - h_1)}{\ln(r_2/r_1)} \tag{2-6}$$

where Q = pumping flow rate, m^3/s
$T = KD$ = transmissivity of aquifer, m^2/s
K = hydraulic conductivity, m/s
D = thickness of aquifer, m
h_1, h_2 = height of piezometric surface at r_1, r_2 from the pumping well

In practice, the transmissivity (T) of the aquifer is determined from a pumping test. For academic purposes, the typical values of hydraulic conductivity given in Table 2-10 may be used.

The maximum sustainable pumping rate is found by setting h_1 equal to the height of the aquifer (D in Figure 2-5) and h_2 equal to the height of the piezometric surface before pumping (H in Figure 2-5). If the required Q cannot be achieved using one well for the design flow, multiple wells may be required. Except for very small demands, this is the rule rather than the exception.

Multiple wells may be used to take advantage of the fact that wells will "recover" their original piezometric surface when pumping ends if there is adequate water in the aquifer. Thus, if the cones of depression of multiple wells do not interfere with one another, the wells can be operated on a schedule that allows them to recover. Theoretically, if the non-pumping time equals the pumping time, the recovery will be complete (Brown, 1963). If the cones of depression do overlap, each well interferes with each of the other wells and the resultant drawdown is increased.

TABLE 2-10
Values of aquifer parameters

Aquifer material	Typical porosity (%)	Range of porosities (%)	Range of specific yield (%)	Typical hydraulic conductivity (m/s)	Range of hydraulic conductivities (m/s)
Unconsolidated					
Clay	55	50–60	1–10	1.2×10^{-6}	$0.1-2.3 \times 10^{-6}$
Loam	35	25–45		6.4×10^{-6}	10^{-6} to 10^{-5}
Fine sand	45	40–50		3.5×10^{-5}	$1.1-5.8 \times 10^{-5}$
Medium sand	37	35–40	10–30	1.5×10^{-4}	10^{-5} to 10^{-4}
Coarse sand	30	25–35		6.9×10^{-4}	10^{-4} to 10^{-3}
Sand and gravel	20	10–30	15–25	6.1×10^{-4}	10^{-5} to 10^{-3}
Gravel	25	20–30		6.4×10^{-3}	10^{-3} to 10^{-2}
Consolidated					
Shale	< 5		0.5–5	1.2×10^{-12}	
Granite	< 1		—	1.2×10^{-10}	
Sandstone	15	5–30	5–15	5.8×10^{-7}	10^{-8} to 10^{-5}
Limestone	15	10–20	0.5–5	5.8×10^{-6}	10^{-7} to 10^{-5}
Fractured rock	5	2–10	—	5.8×10^{-5}	10^{-8} to 10^{-4}

Adapted from Bouwer, 1978, Linsley et al., 1975, and Walton, 1970.

Unconfined Aquifer. The components of the evaluation of the aquifer as a water supply are: (1) depth of the aquifer, (2) annual precipitation and resultant aquifer recharge, and (3) drawdown for sustained pumping at the design rate of demand.

The depth of the aquifer for a unconfined aquifer is measured from the static, unpumped water level to the underlying impermeable layer (Figure 2-6). In theory, the depth of the aquifer sets one dimension of the maximum extent of pumping. Once the water level is lowered to the impermeable layer, the well "drys up." In actuality, this depth cannot be achieved because of other constraints. In populated areas, regulatory agencies have data that permit estimation of the depth of the aquifer. In less densely populated areas, exploration and evaluation by a professional hydrogeologist is required.

Aquifer Recharge. A hydrologic mass balance is used to estimate the potential volume of water that recharges the aquifer. An annual time increment rather than the shorter monthly periods used in surface water analysis may be used for estimation purposes because the aquifer behaves as a large storage reservoir. Under steady-state conditions, the storage volume compensates for dry seasons with wet seasons. Thus, like the analysis of reservoirs, a partial duration series analysis for drought durations of 1 to 5 years with return periods of 25 to 50 years are used in evaluation of an unconfined aquifer as a water source.

Even though vast quantities of water may have accumulated in the aquifer over geologic time periods, the rate of pumping may exceed the rate of replenishment. Even with very deep aquifers where the well does not dry up, the removal of water results in removal of subsurface support. This, in turn, results in loss of surface elevation or land subsidence. Although this occurs in nearly every state in the United States, the San Joaquin Valley in California serves as a classic example. Figure 2-7 is a dramatic photograph showing the land surface as it was in 1977 in relation to its location in 1925. The distance between the 1925 sign and the 1977 sign is approximately 9 m.

Reclaimed Wastewater

Another source of water is recycled or reclaimed water. In regions where potable water is scarce, literally hundreds of communities are recycling wastewater for nonpotable uses. This provides an

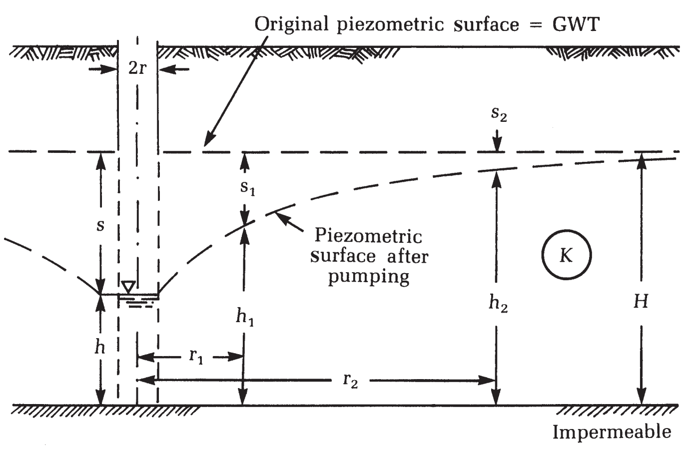

FIGURE 2-6
Geometry and symbols for a pumped well in an unconfined aquifer. (*Source:* H. Bouwer, 1978.)

FIGURE 2-7
Land subsidence in the San Joaquin Valley, 16 km southwest of Mendota, CA.
(*Source:* US Geological Survey Professional Paper 1401-A, *Ground Water in the Central Valley, California—A Summary Report.* Photo by Dick Ireland, USGS, 1977.)

initial means of extending a fully exploited water source. A half dozen cities, including El Paso, Texas and Los Angeles, California, are using treated wastewater to recharge potable aquifers. Los Angeles has been doing so since 1962 (Pinholster, 1995).

2-3 WATER QUALITY

The following four categories are used to describe drinking water quality:

1. *Physical:* Physical characteristics relate to the quality of water for domestic use. They include color, turbidity, temperature, and, in particular, taste and odor.
2. *Chemical:* Chemical characteristics of waters are sometimes evidenced by their observed reactions, such as the comparative performance of hard and soft waters in laundering. Most often, differences are not visible. However, in some cases, such as the oxidation of iron, the reactions result in highly objectionable color.

3. *Microbiological:* Microbiological agents are very important in their relation to public health and may also be significant in modifying the physical and chemical characteristics of water.

4. *Radiological:* Radiological factors must be considered in areas where there is a possibility that the water may have come in contact with radioactive substances. The radioactivity of the water is of public health concern in these cases.

Physical Characteristics

Color. Dissolved organic material from decaying vegetation and certain inorganic matter cause color in water. Occasionally, excessive blooms of algae or the growth of aquatic microorganisms may also impart color. Often the color in water is not true color but *apparent color* that results from a colloidal suspension. Tea is an example of apparent color. While color itself is not usually objectionable from the standpoint of health, its presence is aesthetically objectionable and suggests that the water needs appropriate treatment.

Taste and Odor. Taste and odor (T&O) in water can be caused by foreign matter such as organic compounds, inorganic salts, or dissolved gases. These materials may come from domestic, agricultural, or natural sources. Algae are frequently the source of T&O in surface water supplies. T&O can also result as a byproduct of chlorine disinfection. Drinking water should be free from any objectionable taste or odor at the point of use.

Temperature. The most desirable drinking waters are consistently cool and do not have temperature fluctuations of more than a few degrees. Groundwater and surface water from mountainous areas generally meet these criteria. Most individuals find that water having a temperature between 10°C–15°C is most palatable. Municipal drinking water is not treated to adjust the temperature. However, the temperature of the water may be an important water quality objective for a client and may be an important consideration in the selection of the water source.

Turbidity. The presence of suspended material such as clay, silt, finely divided organic material, plankton, and other particulate material in water is known as turbidity. The unit of measure is a nephelometric turbidity unit (NTU). It is determined by reference to a chemical mixture that produces a reproducible refraction of light. Turbidities in excess of 5 NTU are easily detectable in a glass of water and are usually objectionable for aesthetic reasons.

Clay or other inert suspended particles in and of themselves may not adversely affect health, but water containing such particles may require treatment to make it suitable for disinfection. In general, turbidity reduces disinfection efficiency by consuming the disinfectant and shielding the microorganisms. Following a rainfall, variations in the groundwater turbidity may be considered an indication of surface or other introduced pollution entering the aquifer.

Chemical Characteristics

Arsenic. Arsenic occurs naturally in some geologic formations. It is widely used in timber treatment, agricultural chemicals (pesticides), and the manufacture of computer chips, glass, and alloys. Arsenic in drinking water has been linked to lung and urinary bladder cancer.

Chloride. Most waters contain some chloride. The amount present can be caused by the leaching of marine sedimentary deposits or by pollution from sea water, brine, or industrial or domestic wastes. Chloride concentrations in excess of about 250 mg/L usually produce a noticeable taste in drinking water. Domestic water should contain less than 100 mg/L of chloride to be palatable.

Fluoride. In some areas, water sources contain natural fluoride. Where the concentrations approach optimum levels, beneficial health effects have been observed. In such areas, the incidence of *dental caries* (tooth decay) has been found to be below the 1evels observed in areas without natural fluoride. Many cities choose to add fluoride to the water supply to reduce the incidence of dental caries. The optimum fluoride level for a given area depends upon air temperature because temperature greatly influences the amount of water people drink. Excessive fluoride in drinking water supplies may produce fluorosis (mottling) of teeth,* which increases as the optimum fluoride level is exceeded.

Iron. Small amounts of iron frequently are present in water because of the large amount of iron in the geologic materials. The presence of iron in water is considered objectionable because it imparts a reddish color to the water, stains bathroom fixtures and laundered goods with a yellow to reddish-brown color, and affects the taste of beverages such as tea and coffee.

Lead. Lead occurs in drinking water primarily from corrosion of lead pipes. Lead exposure is associated with a large number of pathological effects including but not limited to interference with red blood cell formation, kidney damage, and impaired cognitive performance.

Manganese. Naturally occurring manganese is often present in significant amounts in groundwater. Anthropogenic sources include discarded batteries, steel alloy production, and agricultural products. It imparts a dark brown or black color to water and stains fixtures and cloth that is washed in it. It flavors coffee and tea with a medicinal taste.

Sodium. The presence of sodium in water can affect persons suffering from heart, kidney, or circulatory ailments. When a strict sodium-free diet is recommended, any water should be regarded with suspicion. Home water softeners may be of particular concern because they add large quantities of sodium to the water.

Sulfate. Waters containing high concentrations of sulfate, caused by the leaching of natural deposits of magnesium sulfate (Epsom salts) or sodium sulfate (Glauber's salt), may be undesirable because of their laxative effects.

Zinc. Zinc is found in some natural waters, particularly in areas where zinc ore deposits have been mined. Zinc is not considered detrimental to health, but it will impart an undesirable taste to drinking water.

Toxic Inorganic Substances. Nitrates (NO_3^-), cyanides (CN^-), and heavy metals constitute the major classes of inorganic substances of health concern. Methemoglobinemia (infant cyanosis or "blue baby syndrome") has occurred in infants who have been given water or fed formula prepared with water having high concentrations of nitrate. Cyanide ties up the hemoglobin sites that bind oxygen to red blood cells. This results in oxygen deprivation. A characteristic symptom

*Mottled teeth are characterized by black spots or streaks and may become brittle when exposed to large amounts of fluoride.

is a blue skin color, which gives the syndrome its name, *cyanosis.* This condition is called *cyanosis.* Cyanide also causes chronic effects on the thyroid and central nervous system.

The toxic heavy metals include arsenic (As), barium (Ba), cadmium (Cd), chromium (Cr), lead (Pb), mercury (Hg), selenium (Se), and silver (Ag). The heavy metals have a wide range of effects. They may be acute poisons (As and Cr^{6+}, for example), or they may produce chronic disease (Pb, Cd, and Hg, for example).

Toxic Organic Substances. There are over 120 toxic organic compounds listed on the U.S. EPA's Priority Pollutant list (Table 2-11). These include pesticides, insecticides, and solvents. Like the inorganic substances, their effects may be acute or chronic.

Microbiological Characteristics

Water for drinking and cooking purposes must be made free from pathogens. These organisms include viruses, bacteria, protozoa, and helminths (worms).

Some organisms that cause disease in people originate with the fecal discharges of infected individuals. Others are from the fecal discharge of animals.

Unfortunately, the specific disease-producing organisms present in water are not easily identified. The techniques for comprehensive bacteriological examination are complex and time-consuming. It has been necessary to develop tests that indicate the relative degree of contamination in terms of an easily defined quantity. The most widely used test estimates the number of microorganisms of the coliform group. This grouping includes two genera: *Escherichia coli* and *Aerobacter aerogenes.* The name of the group is derived from the word "colon". While *E. coli* are common inhabitants of the intestinal tract, *Aerobacter* are common in the soil, on leaves, and on grain; on occasion they cause urinary tract infections. The test for these microorganisms, called the *Total Coliform Test,* was selected for the following reasons:

1. The coliform group of organisms normally inhabits the intestinal tracts of humans and other mammals. Thus, the presence of coliforms is an indication of fecal contamination of the water.
2. Even in acutely ill individuals, the number of coliform organisms excreted in the feces outnumber the disease-producing organisms by several orders of magnitude. The large numbers of coliforms make them easier to culture than disease-producing organisms.
3. The coliform group of organisms survives in natural waters for relatively long periods of time but does not reproduce effectively in this environment. Thus, the presence of coliforms in water implies fecal contamination rather than growth of the organism because of favorable environmental conditions. These organisms also survive better in water than most of the bacterial pathogens. This means that the absence of coliforms is a reasonably safe indicator that pathogens are not present.
4. The coliform group of organisms is relatively easy to culture. Thus, laboratory technicians can perform the test without expensive equipment.

Current research indicates that testing for *Escherichia coli* specifically may be warranted. Some agencies prefer the examination for *E. coli* as a better indicator of biological contamination than total coliforms.

TABLE 2-11
EPA's priority pollutant list

1. Antimony
2. Arsenic
3. Beryllium
4. Cadmium
5a. Chromium (III)
5b. Chromium (VI)
6. Copper
7. Lead
8. Mercury
9. Nickel
10. Selenium
11. Silver
12. Thallium
13. Zinc
14. Cyanide
15. Asbestos
16. 2,3,7,8-TCDD (Dioxin)
17. Acrolein
18. Acrylonitrile
19. Benzene
20. Bromoform
21. Carbon tetrachloride
22. Chlorobenzene
23. Chlorodibromomethane
24. Chloroethane
25. 2-Chloroethylvinyl ether
26. Chloroform
27. Dichlorobromomethane
28. 1,1-Dichloroethane
29. 1,2-Dichloroethane
30. 1,1-Dichloroethylene
31. 1,2-Dichloropropane
32. 1,3-Dichloropropylene
33. Ethylbenzene
34. Methyl bromide
35. Methyl chloride
36. Methylene chloride
37. 1,2,2,2-Tetrachloroethane
38. Tetrachloroethylene
39. Toluene
40. 1,2-trans-dichloroethylene
41. 1,1,1-Trichloroethane
42. 2,4 Dichlorophenol
43. Trichloroethylene
44. Vinyl chloride
45. 2-Chlorophenol
46. 2,4-Dichlorophenol
47. 2,4-Dimethylphenol
48. 2-Methyl-4-chlorophenol
49. 2,4-Dinitrophenol
50. 2-Nitrophenol
51. 4-Nitrophenol
52. 3-Methyl-4-chlorophenol
53. Pentachlorophenol
54. Phenol
55. 2,4,6-Trichlorophenol
56. Acenaphthene
57. Acenaphthylene
58. Anthracene
59. Benzidine
60. Benzo(a)anthracene
61. Benzo(a)pyrene
62. Benzo(a)fluoranthene
63. Benzo(ghi)perylene
64. Benzo(k)fluoranthene
65. bis(2-Chloroethoxy)methane
67. bis(2-Chloroisopropyl)ether
68. bis(2-Ethylhexyl)phthalate
69. 4-Bromophenyl phenyl ether
70. Butylbenzyl phthalate
71. 2-Chloronaphthalene
72. 4-Chlorophenyl phenyl ether
73. Chrysene
74. Dibenzo(a,h)anthracene
75. 1,2-Dichlorobenzene
76. 1,3-Dichlorobenzene
77. 1,4-Dichlorobenzene
78. 3,3-Dichlorobenzidine
79. Diethyl phthalate
80. Dimethyl phthalate
81. Di-n-butyl phthalate
82. 2,4-Dinitrotoluene
83. 2,6-Dinitrotoluene
84. Di-n-octyl phthalate
85. 1,2-Diphenylhydrazine
86. Fluoranthene
87. Fluorene
88. Hexachlorobenzene
89. Hexachlorobutadiene
90. Hexachlorocyclopentadiene
91. Hexachloroethane
92. Indeno(1,2,3-cd)pyrene
93. Isophorone
94. Naphthalene
95. Nitrobenzene
96. N-Nitrosodimethylamine
97. N-Nitrosodi-n-propylamine
98. N-Nitrosodiphenylamine
99. Phenanthrene
100. Pyrene
101. 1,2,4-Trichlorobenzene
102. Aldrin
103. alpha-BHC
104. beta-BHC
105. gamma-BHC
106. delta-BHC
107. Chlordane
108. 4,4'-DDT
109. 4,4'-DDE
110. 4,4'-DDD
111. Dieldrin
112. alpha-Endosulfan
113. beta-Endosulfan
114. Endosulfan sulfate
115. Endrin
116. Endrin aldehyde
117. Heptachlor
118. Heptachlor epoxide
119. PCB-1242
120. PCB-1254
121. PCB-1221
122. PCB-1232
123. PCB-1248
124. PCB-1260
125. PCB-1016
126. Toxaphene

Source: Code of Federal Regulations, 40 CFR 131.36, July 1, 1993.

The two protozoa of most concern are *Giardia lamblia* and *Cryptosporidium parvum.* Both pathogens are associated with gastrointestinal illness. The dormant *Giardia* cysts and *Cryptosporidium* oocysts are carried in animals in the wild and on farms.

Radiological Characteristics

The use of atomic energy as a power source and the mining of radioactive materials, as well as naturally occurring radioactive materials, are sources of radioactive substances in drinking water. Drinking water standards have been established for alpha particles, beta particles, photons emitters, radium-226 and -228, and uranium.

Although no standard has been established for radon, it is of concern because it is highly volatile and is an inhalation hazard from showering. Its decay products (^{218}Po, ^{214}Po, and ^{214}Bi) release alpha, beta, and gamma radiation.

Raw Water Characteristics

The quality of the *raw* (untreated) water plays a large role in determining the unit operations and processes required to treat the water. A comparison of the source water quality with the desired finished water quality provides a basis for selecting treatment processes that are capable of achieving the required treatment efficiency.

In addition to the regulated constituents discussed under "Water Quality Standards" in the next section there are a number of other common analyses used to assess the characteristics of the water with respect to potential treatment requirements. That is, the need for treatment, the difficulty of treatment, and the unit operations and processes that may be required. These are listed in Table 2-12 by the test used for their determination.

If the client's water quality objectives include a soft finished water and the source water is a groundwater or a surface water with a large groundwater contribution, the dissolve cations and anions as well as alkalinity, carbon dioxide, pH, and total hardness are of particular interest. For surface water that will not be softened, sodium, alkalinity, conductivity, pH, and total organic carbon provide useful information beyond the regulated compounds.

For expansion of existing plants, these data may be readily available. Because groundwater quality is not highly variable, annual grab samples provide sufficient data for plant design. Because

TABLE 2-12
Common analyses to characterize raw water

Alkalinity	Iron
Bicarbonate	Manganese
Carbonate	Magnesium
Total	pH
Ammonia	Nitrate
Arsenic	Nitrite
Calcium	Silica
Carbon dioxide	Sodium
Chloride	Total hardness
Conductivity	Total Kjeldahl nitrogen
Hydrogen sulfide	Total organic carbon
Hydroxide	Turbidity

surface water is often highly variable in composition, more extensive time dependent data are desirable.

The ability of a selected design to consistently meet regulatory and client water quality goals is enhanced when the range of the source water quality is within the range of quality that the plant can successfully treat (Logsdon et al., 1999). A probability plot, like that shown in Figure 2-8, provides a comprehensive view of the range of constituent concentrations that must be treated. (A blank copy of probability paper may be downloaded from the website: http//www.mhhe.com/davis1e.) It will be easier to maintain product water quality for source water with a shallow slope (Water A in Figure 2-8) than it will for a source water with a steep slope (Water B).

In addition to the chemical analyses, it is imperative that the design engineer conduct a *sanitary survey* (AWWA, 1999). This is a field investigation that covers a large geographic area beyond the immediate area surrounding the water supply source.The purpose of the sanitary survey is to detect potential health hazards and assess their present and future importance. This assessment includes such things as landfills, hazardous waste sites, fuel storage areas, industrial plants, and wastewater treatment plants. Examples of sources to be investigated during the sanitary survey are listed in Table 2-13.

Water Quality Standards

Water quality standards are a crucial element in setting the design criteria for a water supply project. The standards apply to both the treatment plant and the distribution system. Because of their crucial role, they are examined in detail in the following paragraphs.

The National Safe Drinking Water Act (SDWA) was signed into law on December 16, 1974. The Environmental Protection Agency (EPA) was directed to establish *maximum contaminant levels* (MCLs) for public water systems to prevent the occurrence of any known or anticipated adverse health effects with an adequate margin of safety. EPA defined a *public water system* to be any system that either has 15 or more service connections or regularly serves an average of

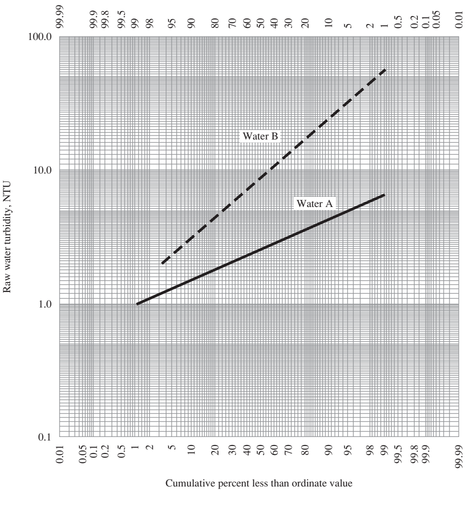

FIGURE 2-8
Cumulative log-probability plot of a water quality constituent.

TABLE 2-13
Examples of sources to be investigated during the sanitary survey

Surface water	Groundwater
Land use and zoning	Land use and zoning
Local geology and soils	Local geology and soils
Cultivated areas	Cultivated areas
Orchards	Orchards
Pastures	Pastures
Bathing areas	Raw materials storage
Gross erosion	Landfills
Marinas	Septic tank tile fields
Septic tank tile fields	Well logs
Sewer outfalls	
Storm water drains	
Swamps	
Upstream tributaries	
Vegetation	

at least 25 or more people daily for at least 60 days out of the year. The SDWA defines two types of public water supply: community and noncommunity. A *community system* serves a residential, year-round, population greater than 25 people or 15 living units. A *noncommunity system* is one that is not a community system but that serves not fewer than 25 individuals on an average daily basis for not less than 60 days per year. The noncommunity systems are further separated into two groups: transient and nontransient. The *transient systems* serve intermittent nonresidential users. Examples are campgrounds and restaurants. *Nontransient systems* are non-residential systems that routinely serve the same individuals. Schools and places of business are examples of this category.

From 1975 through 1985, the EPA regulated 23 contaminants in drinking water supplied by public water systems. These regulations are known as National Interim Primary Drinking Water Regulations (IPDWRs). In June of 1986, the SDWA was amended. The amendments required EPA to set *maximum contaminant level goals* (MCLGs) and MCLs for 83 specific substances. This list included 22 of the IPDWRs (all except trihalomethanes). The amendments also required EPA to regulate 25 additional contaminants every three years beginning in January, 1991 and continuing for an indefinite period of time.

Table 2-14 lists each regulated contaminant and summarizes its adverse health effects. Some of these contaminant levels are being considered for revision. The notation “TT” in the table means that a treatment technique is specified rather than a contaminant level. The treatment techniques are specific processes that are used to treat the water. Some examples include coagulation and filtration, lime softening, and ion exchange. These are discussed in Chapters 3 through 10.

Lead and Copper. In June 1988, EPA issued proposed regulations to define MCLs and MCLGs for lead and copper, as well as to establish a monitoring program and a treatment technique for both. The MCLG proposed for lead is zero; for copper, 1.3 mg/L. The MCL action levels, applicable to water entering the distribution system, are 0.005 mg/L for lead and 1.3 mg/L for copper.

TABLE 2-14
Standards and potential health effects of the contaminants regulated under the SDWA

Contaminant	Maximum contaminant level goal mg/L	Maximum contaminant level mg/L	Best Available Technology (BAT)	Potential health effects
Organics				
Acrylamide	Zero	TT	PAP	Cancer, nervous system effects
Alachor	Zero	0.002	GAC	Cancer
Atrazine	0.003	0.003	GAC	Liver, kidney, lung, cardiovascular effects; possible carcinogen
Benzene	Zero	0.005	GAC, PTA	Cancer
Benzo(a)pyrene	Zero	0.0002	GAC	Cancer
Bromodichloromethane	Zero	See TTHM	GAC, NF[†]	Cancer
Bromoform	Zero	See TTHM	GAC, NF[†]	Cancer
Carbofuran	0.04	0.04	GAC	Nervous system, reproductive system effects
Carbon tetrachloride	Zero	0.005	GAC, PTA	Cancer
Chlordane	Zero	0.002	GAC	Cancer
Chloroform	0.07	See TTHM	GAC, NF[*]	Cancer
Chlorodibromomethane	No MCLG	See TTHM	GAC, NF[*]	Cancer
2,4-D	0.07	0.07	GAC	Liver, kidney effects
Dalapon	0.2	0.2	GAC	Kidney, liver effects
Di(2-ethylhexyl)adipate	0.4	0.4	GAC, PTA	Reproductive effects
Di(2-ethylhexyl)phthalate	Zero	0.006	GAC	Cancer
Dibromochloropropane (DBCP)	Zero	0.0002	GAC, PTA	Cancer
Dichloroacetic acid	No MCLG	See HAA5	GAC, PTA	Cancer
p-Dichlorobenzene	0.075	0.075	GAC, PTA	Kidney effects, possible carcinogen
o-Dichlorobenzene	0.6	0.6	GAC, PTA	Liver, kidney, blood cells effects
1,2-Dichloroethane	Zero	0.005	GAC, PTA	Cancer
1,1-Dichloroethylene	0.007	0.007	GAC, PTA	Liver, kidney effects, possible carcinogen
cis–1,2-Dichloroethylene	0.07	0.07	GAC, PTA	Liver, kidney, nervous system, circulatory effects
trans–1,2-Dichloroethylene	0.1	0.1	GAC, PTA	Liver, kidney, nervous system, circulatory effects
Dichloromethane (methylene chloride)	Zero	0.005	PTA	Cancer
1,2-Dichloropropane	Zero	0.005	GAC, PTA	Cancer
Dibromoacetic acid	No MCLG	See HAA5	GAC, NF[*]	Cancer
Dichloroacetic acid	No MCLG	See HAA5	GAC, NF[*]	Cancer
Dinoseb	0.007	0.007	GAC	Thyroid, reproductive effects
Diquat	0.02	0.02	GAC	Ocular, liver, kidney effects
Endothall	0.1	0.1	GAC	Liver, kidney, gastrointestinal effects
Endrin	0.002	0.002	GAC	Liver, kidney, nervous system effects
Epichlorohydrin	Zero	TT	PAP	Cancer

Organics				
Ethylbenzene	0.7	0.7	GAC, PTA	Liver, kidney, nervous system effects
Ethylene dibromide (EDB)	Zero	0.00005	GAC, PTA	Cancer
Glyphosate	0.7	0.7	OX	Liver, kidney effects
Haloacetic acids (sum of 5; HAA5)[1]	No MCLG	0.060	GAC, NF*	Cancer
Heptachlor	Zero	0.0004	GAC	Cancer
Heptachlor epoxide	Zero	0.0002	GAC	Cancer
Hexachlorobenzene	Zero	0.001	GAC	Cancer
Hexachlorocyclopentadiene	0.05	0.05	GAC, PTA	Kidney, stomach effects
Lindane	0.0002	0.0002	GAC	Liver, kidney, & nervous, immune, circulatory system effects
Methoxychlor	0.04	0.04	GAC	Development, liver, kidney, nervous system effects
Monochlorobenzene	0.1	0.1	GAC, PTA	Cancer
Monochloroacetic acid	0.07	See HAA5	GAC, NF*	Cancer
Monobromoacetic acid	No MCLG	See HAA5	GAC, NF*	Cancer
Oxamyl (vydate)	0.2	0.2	GAC	Kidney effects
Pentachlorophenol	Zero	0.001	GAC	Cancer
Picloram	0.5	0.5	GAC	Kidney, liver effects
Polychlorinated biphenyls (PCBs)	Zero	0.0005	GAC	Cancer
Simazine	0.004	0.004	GAC	Body weight and blood effects, possible carcinogen
Styrene	0.1	0.1	GAC, PTA	Liver, nervous system effects, possible carcinogen
2,3,7,8-TCDD (dioxin)	Zero	5×10^{-8}	GAC	Cancer
Tetrachloroethylene	Zero	0.005	GAC, PTA	Cancer
Toluene	1	1	GAC, PTA	Liver, kidney, nervous system, circulatory system effects
Toxaphene	Zero	0.003	GAC	Cancer
2,4,5-TP (silvex)	0.05	0.05	GAC	Liver, kidney effects
Trichloroacetic acid	0.02	See HAA5	GAC, NF†	Cancer
1,2,4-Trichlorobenzene	0.07	0.07	GAC, PTA	Liver, kidney effects
1,1,1-Trichloroethane	0.2	0.2	GAC, PTA	Liver, nervous system effects
1,1,2-Trichloroethene	0.003	0.005	GAC, PTA	Kidney, liver effects, possible carcinogen
Trichloroethylene	Zero	0.005	GAC, PTA	Cancer
Trihalomethanes (sum of 4; TTHM's)[2]	No MCLG	0.080	GAC, NF†	Cancer
Vinyl chloride	Zero	0.002	PTA	Cancer
Xylenes (total)	10	10	GAC, PTA	Liver, kidney, nervous system effects

(continued)

TABLE 2-14
Standards and potential health effects of the contaminants regulated under the SDWA *(continued)*

Contaminant	Maximum contaminant level goal mg/L	Maximum contaminant level mg/L	Best Available Technology (BAT)	Potential health effects
Inorganics				
Antimony	0.006	0.006	C-F[3], RO	Decreased longevity, blood effects
Arsenic	Zero	0.010	IX,AA,RO, C-F,LS,ED, OX-F	Dermal, nervous system effects, cancer
Asbestos (fibers > 10 μm)	7 million (fibers/L)	7 million (fibers/L)	C-F[3], DF, DEF	Possible carcinogen by ingestion
Barium	2	2	IX,RO, LS[3]	Blood pressure effects
Beryllium	0.004	0.004	IX,RO, C-F[3] LS[3], AA,IX	Bone, lung effects, cancer
Bromate	Zero	0.010	DC	
Cadmium	0.005	0.005	C-F[3], LS3, IX, RO	Kidney effects
Chlorite	0.8	1.0	DC	Nervous system effects
Chromium (total)	0.1	0.1	C-F[3], LS[3], (Cr III), IX, RO	Liver, kidney, circulatory system effects
Copper	1.3	TT	CC, SWT	Gastrointestinal effects
Cyanide	0.2	0.2	IX, RO, Cl_2	Thyroid, central nervous system effects
Fluoride	4	4	AA, RO	Skeletal fluorosis
Lead	Zero	TT	CC, PE, SWT, LSLR	Cancer, kidney, central and peripheral nervous system effects
Mercury	0.002	0.002	C-F[3] (influent < 10 μg/L), LS[3], GAC, RO (influent < 10 μg/L)	Kidney, central nervous system effects
Nitrate (as N)	10	10	IX,RO,ED	Methemoglobinemia (blue baby syndrome)
Nitrite (as N)	1	1	IX,RO	Methemoglobinemia (blue baby syndrome)
Nitrate + nitrite (both as N)	10	10	IX,RO	
Selenium	0.05	0.05	C-F[3] (Se IV), LS[3], AA,RO,ED	Nervous system effects
Thallium	0.0005	0.002	IX, AA	Liver, kidney, brain, intestine effects
Radionuclides				
Beta particle and photon emitters	Zero	4 mrem	C-F,IX,RO	Cancer
Alpha particles	Zero	15 pCi/L	C-F,RO	Cancer
Radium-226 + radium-228	No MCLG	5 pCi/L	IX,LS,RO	Cancer
Uranium	Zero	30 μg/L	C-F[3], LS[3], AX	Cancer

Microbials				
Cryptosporidium	Zero	TT	NA	Gastroenteric disease
E. coli	Zero	TT[5]	NA	Gastroenteric disease
Fecal coliforms	Zero	TT[5]	NA	Gastroenteric disease
Giardia lambia	Zero	TT	NA	Gastroenteric disease
Heterotrophic bacteria	No MCLG	TT	NA	Gastroenteric disease
Legionella	Zero	TT	NA	Pneumonialike effects
Total coliforms	Zero	TT[4]	NA	Indicator of gastroenteric infections
Turbidity		PS	NA	Interferes with disinfection, indicator of filtration performance
Viruses	Zero	TT	NA	Gastroenteric disease, respiratory disease, and other diseases, (e.g. hepatitis, myocarditis)

*Consecutive systems can use monochloramine (NH_2Cl) as BAT.

AA–activated alumina, AX–anion exchange, CC–corrosion control, C-F–coagulation and filtration, Cl_2–chlorination, DC–disinfection system control, DEF–deatomaceous earth filtration, DF–direct filtration, EF–enhanced coagulation, ED–electrodialysis, GAC–granular activated carbon, IX–ion exchange, LS–lime softening, LSLR–lead service line replacement, NA–not applicable, N-F–nanofiltration, OX–oxidation, OX-F–oxidation and filtration, PAP–polymer addition practices, PE–public education, PR–precursor removal, PS–performance standard, PTA–packed-tower aeration, RO–reverse osmosis, SWT–source water treatment, TT–treatment technique.

1. Sum of the concentrations of mono-, di-, and trichloroacetic acids and mono- and dibromoacetic acids.
2. Sum of the concentrations of bromodichloromethane, dibromonochloromethane, bromoform, and chloroform.
3. Coagulation-filtration and lime-softening are not BAT for small systems for variance unless treatment is already installed.
4. No more than 5 percent of the samples per month may be positive. For systems collecting fewer than 40 samples per month, no more than 1 sample per month may be positive.
5. If a repeat total coliform sample is fecal coliform- or *E. coli*-positive, the system is in violation of the MCL for total coliforms. The system is also in violation of the MCL for total coliforms if a routine sample is fecal coliform- or *E. coli*-positive and is followed by a total coliform-positive repeat sample.

Compliance with the regulations is also based on the quality of the water at the consumer's tap. Monitoring is required by means of collection of first-draw samples at residences. The number of samples required to be collected will range from 10 per year to 50 per quarter, depending on the size of the water system.

The SDWA amendments forbid the use of pipe, solder, or flux that is not lead-free in the installation or repair of any public water system or in any plumbing system providing water for human consumption. This does not, however, apply to leaded joints necessary for the repair of old cast iron pipes.

Disinfectants and Disinfectant By-Products (D-DBPs). The disinfectants used to destroy pathogens in water and the by-products of the reaction of these disinfectants with organic materials in the water are of potential health concern. One class of DBPs has been regulated since 1979. This class is known as trihalomethanes (THMs). THMs are formed when a water containing an organic precursor is chlorinated. In this case it means an organic compound capable of reacting to produce a THM. The precursors are natural organic substances formed from the decay of vegetative matter, such as leaves, and aquatic organisms. THMs are of concern because they are known or potential carcinogens. The four THMs that were regulated in the 1979 rules are chloroform ($CHCl_3$), bromodichloromethane ($CHBrCl_2$), dibromochloromethane ($CHBr_2Cl$). and bromoform ($CHBr_3$). Of these four, chloroform appears most frequently and is found in the highest concentrations.

The D-DBP rule was developed through a negotiated rule-making process, in which individuals representing major interest groups concerned with the rule (for example, public-water-system owners, state and local government officials, and environmental groups) publicly work with EPA representatives to reach a consensus on the contents of the proposed rule.

Maximum residual disinfectant level goals (MRDLGS) and *maximum residual disinfectant levels* (MRDLS) were established for chlorine, chloramine, and chlorine dioxide (Table 2-15). Because ozone reacts too quickly to be detected in the distribution system, no limits on ozone were set.

The MCLGs and MCLs for disinfection byproducts are listed in Table 2-16. In addition to regulating individual compounds, the D-DBP rule set levels for two groups of compounds: HAA5 and TTHMs. These groupings were made to recognize the potential cumulative effect of several compounds. HAA5 is the sum of five haloacetic acids (monochloroacetic acid, dichloroacetic acid, trichloroacetic acid, monobromoacetic acid, and dibromoacetic acid). TTHMs (total trihalomethanes) is the sum of the concentrations of chloroform ($CHCl_3$), bromodichloromethane ($CHBrCl_2$), dibromochloromethane ($CHBr_2Cl$), and bromoform ($CHBr_3$).

The D-DBP rule is quite complex. In addition to the regulatory levels shown in the tables, levels are established for precursor removal. The amount of precursor required to be removed is a function the alkalinity of the water and the amount of *total organic carbon* (TOC) present.

TABLE 2-15
Maximum residual disinfectant level goals (MRDLGs) and maximum residual disinfectant levels (MRDLs)

Disinfectant residual	MRDLG, mg/L	MRDL, mg/L
Chlorine (free)	4	4.0
Chloramines (as total chlorine)	4	4.0
Chlorine dioxide	0.8	0.8

TABLE 2-16
Maximum contaminant level goals (MCLGs) and maximum contaminant levels (MCLs) for disinfectant by-products (DBPs)

Contaminant	MCLG, mg/L	Stage 1 MCL, mg/L	Stage 2 MCL, mg/L
Bromate	Zero	0.010	
Bromodichloromethane	Zero		
Bromoform	Zero		
Chloral hydrate	0.005		
Chlorite	0.3	1.0	
Chloroform	0.07		
Dibromochloromethane	0.06		
Dichloroacetic acid	Zero		
Monochloroacetic acid	0.03		
Trichloroacetic acid	0.02		
HAA5		0.060	0.060[a]
TTHMs		0.080	0.080[a]

[a]Calculated differently in Stage 2.

The D-DBP rule was implemented in stages. Stage 1 of the rule was promulgated in November 1998. Stage 2 was promulgated in 2006.

When chlorine is added to water that contains TOC, the chlorine and TOC slowly react to form THMs and HAA5. The concentrations of THM and HAA5 continuously increase until the reactions go to completion. Compliance with the regulation is based on samples taken from the distribution system. Although the number of samples may vary, generally it is about four samples collected quarterly. In the Stage 1 rule, the sample points are averaged over four quarters of data. Thus, for the case of four samples for four quarters, 16 data points are averaged to determine compliance. In the Stage 2 rule, four samples (one from each quarter) from a single site are averaged. Each site must be below the MCL. This is referred to as a *locational running annual average* (LRAA). Although the MCLs in Stage 1 and 2 are the same, compliance is more difficult with the Stage 2 rule.

Surface Water Treatment Rule (SWTR). The Surface Water Treatment Rule (SWTR) and its companion rules, the Interim Enhanced Surface Water Treatment Rule (IESWTR) and the Long-Term Enhanced Surface Water Treatment Rules (LT1ESWTR and LT2ESWTR), set forth primary drinking water regulations requiring treatment of surface water supplies or groundwater supplies under the direct influence of surface water. The regulations require a specific treatment technique-filtration and/or disinfection in lieu of establishing maximum contaminant levels (MCLs) for turbidity, *Cryptosporidium, Giardia lamblia,* viruses, *Legionella,* and heterotrophic bacteria, as well as many other pathogenic organisms that are removed by these treatment techniques. The regulations also establish a maximum contaminant level goal (MCLG) of zero for *Giardia lamblia, Cryptosporidium,* viruses, and *Legionella.* No MCLG is established for heterotrophic plate count or turbidity.

Turbidity Limits. Treatment by conventional or direct filtration must achieve a turbidity level of less than 0.3 NTU in at least 95 percent of the samples taken each month. Those systems using slow sand filtration must achieve a turbidity level of less than 5 NTU at all times and not more than 1 NTU in more than 5 percent of the samples taken each month. The 1 NTU limit may be increased by the state up to 5 NTU if it determines that there is no significant interference with disinfection. Other filtration technologies may be used if they meet the turbidity requirements set for slow sand filtration, provided they achieve the disinfection requirements and are approved by the state.

Turbidity measurements must be performed on representative samples of the system's filtered water every four hours or by continuous monitoring. For any system using slow sand filtration or a filtration treatment other than conventional treatment, direct filtration, or diatomaceous earth filtration, the state may reduce the monitoring requirements to once per day.

Disinfection Requirements. Filtered water supplies must achieve the same disinfection as required for unfiltered systems (that is, 99.9 or 99.99% removal, also known as *3-log and 4-log removal* or *inactivation*, for *Giardia lamblia* and viruses) through a combination of filtration and application of a disinfectant.

Giardia and viruses are both fairly well inactivated by chlorine. Thus, with proper physical treatment and chlorination, both can be controlled. *Cryptosporidium,* however, is resistant to chlorination. Depending on the source water concentration, EPA establishes levels of treatment that include physical barriers and disinfection techniques. Ozonation and disinfection with ultraviolet light are effective in destroying *Cryptosporidium.*

Total Coliform. On June 19, 1989, the EPA promulgated the revised National Primary Drinking Water Regulations for total coliforms, including fecal coliforms and *E. coli.* These regulations apply to all public water systems.

The regulations establish a maximum contaminant level (MCL) for coliforms based on the presence or absence of coliforms. Larger systems that are required to collect at least 40 samples per month cannot obtain coliform-positive results in more than 5 percent of the samples collected each month to stay in compliance with the MCL. Smaller systems that collect fewer than 40 samples per month cannot have coliform-positive results in more than one sample per month.

The EPA will accept any one of the five analytical methods noted below for the determination of total coliforms:

Multiple-tube fermentation technique (MTF)

Membrane filter technique (MF)

Minimal media ONPO-MUG test (colilert system) (MMO-MUG)

Presence-absence coliform test (P-A)

Colisure technique

Regardless of the method used, the standard sample volume required for total coliform testing is 100 mL.

A public water system must report a violation of the total coliform regulations to the state no later than the end of the next business day. In addition to this, the system must make public notification according to the general public notification requirements of the Safe Drinking Water Act, but with special wording prescribed by the total coliform regulations.

TABLE 2-17
Secondary maximum contaminant levels

Contaminant	SMCL, mg/L[a]
Chloride	250
Color	15 color units
Copper	1
Corrosivity	Noncorrosive
Foaming agents	0.5
Hydrogen sulfide	0.05
Iron	0.3
Manganese	0.05
Odor	3 threshold odor number units
pH	6.5–8.5
Sulfate	250
Total dissolved solids (TDS)	500
Zinc	5

[a]All quantities are mg/L except those for which units are given.

Secondary Maximum Contaminant Levels (SMCLs). The National Safe Drinking Water Act also provided for the establishment of an additional set of standards to prescribe maximum limits for those contaminants that tend to make water disagreeable to use, but that do not have any particular adverse public health effect. These secondary maximum contaminant levels are the advisable maximum level of a contaminant in any public water supply system. The levels are shown in Table 2-17.

AWWA Goals. The primary and secondary maximum contaminant levels are the maximum allowed (or recommended) values of the various contaminants. However, a reasonable goal may be much lower than the MCLs themselves. The American Water Works Association (AWWA) has issued its own set of goals to which its members try to adhere. These goals are shown in Table 2-18.

2-4 EVALUATION OF PROCESS OPTIONS

In the design process, the data gathered in the sections outlined to this point in the chapter would be sufficient to begin screening alternative supply and treatment options. In most cases a number of options will be available. The pros and cons of these selections are discussed in Chapters 3 through 11.

2-5 PLANT SIZING AND LAYOUT

Once the preliminary selection of the water treatment unit operations and processes has been made (the screening process discussed in Chapter 1), rough calculations are made to determine sizes to be used in examining feasibility of site locations and cost. The elements to be considered

TABLE 2-18
American Water Works Association water quality goals

Contaminant	Goal, mg/L[a]
Turbidity	< 0.1 turbidity units (TU)
Color	< 3 color units
Odor	None
Taste	None objectionable
Aluminum	< 0.05
Copper	< 0.2
Iron	< 0.05
Manganese	< 0.01
Total dissolved solids (TDS)	200.0
Zinc	< 1.0
Hardness	80.0

[a]All quantities are mg/L except those for which units are given.

in plant sizing include: (1) number and size of process units, and (2) number and size of ancillary structures. The layout should include: (1) provision for expansion, (2) connection to the transportation net, (3) connection to the water distribution system, and (4) residuals handling system.

Number and Size of Process Units

To ensure the provision of water to the public water supply, in general, a minimum of two units is provided for redundancy. When only two units are provided, each shall be capable of meeting the plant design capacity. Normally, the design capacity is set at the projected maximum daily demand for the end of the design period. The size of the units is specified so that the plant can meet the design capacity with one unit out of service (GLUMRB, 2003). Consideration should also be given to the efficiency/effectiveness of the process units with the low demand at start up of the facility.

Number and Size of Ancillary Units

The ancillary units include: administration building, laboratory space, storage tanks, mechanical building for pumping facilities, roads, and parking. The size of these facilities is a function of the size of the plant. In small to medium sized facilities, particularly in cold climates and when land is expensive, administration, laboratory, pumping and storage are housed in one building.

The storage tanks include those for chemicals, treated water, and in some instances fuel. Space for storage of chemical residuals must also be provided.

Plant Layout

When space is not a constraint, a linear layout generally allows the maximum flexibility for expansion. Redundancy is enhanced if the units are interconnected in such a way that the flow through the plant can be shuttled from one treatment train to another. Because chemicals must be delivered to the plant, connection to the transportation net becomes an integral part of the layout.

Likewise, because residuals are generally transported off-site, the residuals handling system is an integral part of the plant layout.

2-6 PLANT LOCATION

Ideally a site comparison study will be performed after alternatives have been screened and rough sizing of the processes is complete. Many factors may preclude the ideal situation. For example, in highly urbanized areas the availability of land may preclude all but one site. In some cases the availability of land may force the selection of processes that fit into the available space.

Given that more than one site is available, there are several major issues to be considered. As noted in Chapter 1, cost is a major element in the selection process. In addition, the site should allow for expansion. The location of the plant relative to the transportation net, raw water supply, and the service area should be weighed carefully. The physical characteristics of the site alternatives that must be evaluated include the potential for flooding, foundation stability, groundwater intrusion, and the difficulty in preparing the site. For example, the need for blasting of rock may make the cost prohibitive for an otherwise ideal site. Other issues to be considered include wetland infringement, the availability of alternate, independent sources of power, waste disposal options, public acceptance, and security.

Visit the text website at **www.mhhe.com/davis1e** for supplementary materials and a gallery of additional photos.

2-7 CHAPTER REVIEW

When you have completed studying this chapter, you should be able to do the following without the aid of your textbook or notes:

1. Explain to a client the influence of regulatory constraints on the selection of a design period.
2. For a given population growth rate, select an appropriate design period.
3. Explain to a news media person the influence of local factors such as climate, industrial development, and meterage on a national estimate of unit demand.
4. Explain to a client why groundwater is often preferred as a source of water.
5. Use a yield curve to estimate a safe yield.
6. Describe the potential deleterious effects of overpumping a confined or an unconfined aquifer.
7. Explain the implications of a flat or steep slope in a log-probability plot of a water quality parameter in the design of a water treatment plant.

With the use of this text, you should be able to do the following:

8. Construct a yield curve.
9. Construct an annual minima series.

10. Use mass balance techniques to estimate the required volume of a small reservoir.
11. Estimate the maximum sustainable drawdown of a well pumping from a confined aquifer.
12. Compare the results of a water analysis with water quality criteria and determine deficiencies that need to be remedied by treatment.
13. Estimate the demand flow rate for the average day, maximum day, and peak hour for a small nonindustrial community.

2-8 PROBLEMS

2-1. Estimate the demand (in m^3/d) of a new suburban subdivision of 333 houses for the average, maximum, and minimum day. Assume that both the AWWA household average demand and Figure 2-1 apply. Also assume that each house is occupied by three residents.

2-2. A resort community has been platted in Arizona. The year round population when it is fully developed is estimated to be 7,000. A gross estimate of the average day demand is required for planning purposes. Using Hutson et al. (2001) and census population data, estimate the demand. Assume that the "public water supply" category applies.

2-3. A ski lift operation in Colorado plans to expand to include a 250 room hotel, a restaurant to seat 250, and dormitory-style living quarters for a staff of 25 individuals. Estimate the increase in average daily demand during the ski season that must be provided. Assume the average hotel occupancy is two people per room.

2-4. Using a spreadsheet you have written, perform a complete series analysis on the data for the Squannacook River near West Groton, MA, given on page 2-41 Plot a yield curve. What is the safe yield of the river if the regulatory agency will allow a withdrawal of 6%?

2-5. Using a spreadsheet you have written, perform a complete series analysis on the data for the Clear Fork Trinity River at Fort Worth, TX, given on page 2-42. Plot a yield curve. What is the safe yield of the river if the regulatory agency will allow a withdrawal of 3%?

2-6. Using a spreadsheet you have written, perform an annual minima analysis on the data for the Squannacook River near West Groton, MA, given on page 2-41 Plot the data on Gumbel paper and determine the minimum monthly discharge for the mean annual drought. If the demand is 0.131 m^3/s, will storage be required if the regulatory agency will allow a withdrawal of 6%?

2-7. Using a spreadsheet you have written, perform an annual minima analysis on the data for the Clear Fork Trinity River at Fort Worth, TX, given on page 2-42. Plot the data on Gumbel paper and determine the minimum monthly discharge for the mean annual drought. If the demand is 0.021 m^3/s, will storage be required if the regulatory agency will allow a withdrawal of 3%?

Squannacook River near West Groton, MA

Mean monthly discharge (m^3/s)

Year	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec
1951	3.48	8.18	6.63	6.63	3.20	2.38	2.40	1.49	1.06	1.82	6.60	4.47
1952	6.68	5.07	6.51	9.94	5.44	3.71	.87	1.01	.69	.45	1.05	3.37
1953	4.79	6.77	11.44	9.80	6.34	1.21	.52	.42	.29	.51	1.34	3.65
1954	2.06	3.20	4.67	4.53	9.71	2.75	1.21	1.05	6.94	2.27	6.26	7.16
1955	2.92	3.06	5.41	6.17	2.77	1.44	.46	1.63	.61	8.38	8.61	2.17
1956	9.15	3.29	3.82	14.56	5.21	2.50	.77	.40	.50	.54	1.14	2.33
1957	2.92	2.63	4.22	3.99	2.65	.87	.37	.22	.22	.29	.97	3.91
1958	5.89	3.48	6.60	12.40	5.35	1.29	.81	.49	.45	.62	1.13	1.49
1959	2.07	2.05	5.41	8.67	2.37	1.22	1.17	.59	.82	2.55	4.08	5.55
1960	3.51	3.96	3.03	14.73	5.52	2.41	1.09	1.21	2.71	2.18	3.34	2.49
1961	1.57	3.09	7.28	11.10	4.67	2.31	1.03	.80	1.23	.99	2.06	1.73
1962	3.14	1.80	5.47	10.93	3.71	1.25	.56	.69	.50	2.95	4.73	4.30
1963	2.19	1.76	6.83	7.53	2.66	.77	.38	.24	.25	.35	1.52	1.98
1964	3.77	2.57	7.33	6.57	1.85	.59	.38	.25	.21	.27	.36	.79
1965	.65	1.33	2.38	3.79	1.47	.59	.23	.20	.19	.27	.45	.64
1966	.61	1.96	5.55	2.92	2.46	.80	.26	.18	.27	.52	1.75	1.35
1967	1.68	1.53	2.64	10.62	6.29	3.17	2.22	.72	.47	.60	1.07	3.03
1968	2.02	2.14	9.60	3.79	3.82	4.79	1.92	.61	.48	.46	1.88	4.33
1969	2.21	2.17	5.81	10.70	2.80	1.01	.58	1.03	.93	.52	5.24	5.83

Clear Fork Trinity River at Fort Worth, TX

Mean monthly discharge (m^3/s)

Year	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec
1940	—	—	—	—	—	—	—	—	—	0.00	5.63	15.4
1941	4.59	23.8	7.50	6.91	10.2	17.0	2.07	2.29	0.20	1.71	0.631	0.926
1942	0.697	0.595	0.614	58.93	24.1	9.09	0.844	0.714	1.21	10.0	2.38	1.87
1943	1.33	1.00	3.99	3.71	8.38	3.77	0.140	0.00	1.33	0.014	0.00	0.139
1944	0.311	4.93	2.83	2.25	13.3	1.68	0.210	0.609	4.11	0.985	0.515	1.47
1945	3.06	30.38	35.23	28.85	5.69	21.7	2.14	0.230	0.162	0.971	0.617	0.541
1946	1.88	5.75	3.54	1.89	6.57	5.86	0.153	1.45	4.02	0.906	12.2	10.3
1947	4.64	2.62	4.87	5.13	2.27	4.25	0.292	0.054	0.535	0.371	0.331	3.51
1948	3.99	16.9	9.06	1.91	2.64	1.11	1.22	0.00	0.00	0.00	0.00	0.003
1949	0.309	4.19	9.94	4.16	55.21	11.1	1.38	0.450	0.447	4.53	0.711	0.614
1950	3.28	14.7	3.26	12.7	15.1	2.50	3.60	2.44	10.6	1.12	0.711	0.801
1951	0.708	0.994	0.719	0.527	1.37	6.20	0.980	0.00	0.00	0.00	0.006	0.090
1952	0.175	0.413	0.297	1.93	3.65	0.210	0.003	0.029	0.007	0.00	0.368	0.167
1953	0.099	0.080	0.134	0.671	0.934	0.008	0.286	0.249	0.041	0.546	0.182	0.066
1954	0.108	0.092	0.114	0.088	0.278	0.017	0.021	0.015	0.008	0.047	0.024	0.063
1955	0.091	0.153	0.317	0.145	0.464	0.640	0.049	0.050	0.119	0.104	0.055	0.058
1956	0.069	0.218	0.026	0.306	1.35	0.30	0.026	0.019	0.029	0.266	0.030	0.170
1957	0.065	0.300	0.385	12.8	23.6	59.9	6.97	1.36	0.501	0.476	0.855	1.55
1958	1.65	1.61	4.59	5.69	28.1	0.589	0.524	0.456	0.549	0.572	0.490	0.566
1959	0.759	0.776	0.120	0.261	0.097	0.685	0.379	0.668	0.473	9.03	1.64	3.65
1960	11.8	4.45	3.26	1.42	0.631	0.379	0.660	0.566	0.467	0.498	0.241	0.648
1961	2.05	1.92	3.40	1.02	0.306	2.34	0.821	0.816	1.08	0.824	0.297	0.504
1962	0.311	2.03	0.467	0.759	0.459	0.236	0.745	1.41	6.94	1.31	0.405	0.767
1963	0.345	0.268	0.379	1.74	3.79	1.48	0.527	0.586	0.331	0.277	0.249	0.266
1964	0.416	0.266	1.16	0.813	1.02	0.374	0.535	0.963	3.96	0.351	1.47	0.886
1965	2.13	14.6	4.16	2.28	20.5	2.45	1.22	0.821	0.776	0.394	0.476	0.213
1966	0.169	0.354	0.462	6.40	23.1	18.5	5.32	0.951	0.294	1.37	0.15	0.134
1967	0.244	0.244	0.198	0.688	1.04	3.65	0.354	0.068	0.697	0.473	0.394	0.558
1968	1.64	3.85	23.2	1.89	15.7	6.12	0.583	0.144	0.220	0.419	0.396	0.206
1969	0.259	0.555	2.66	12.1	21.2	0.745	0.674	0.30	1.56	0.917	0.459	1.94
1970	5.78	6.37	27.0	3.31	15.0	1.03	0.521	0.697	1.23	—	—	—

2-8. Using a spreadsheet you have written and the data in Table 2-7, continue the analysis of the required storage volume begun in Example 2-3 through April, 2001. What size reservoir is required? Is it full at the end of the April 2001? Ignore the need for increased volume for sediment.

2-9. Using a spreadsheet you have written and the data for the Hoko River near Sekiu, WA, below, determine the required storage volume for a uniform demand of 0.35 m^3/s for the period January 1969 through December 1973. Assume a regulatory restriction that allows only 6% of the flowrate to be withdrawn. What size reservoir is required? Is it full at the end of December, 1973? Ignore the need for increased volume for sediment.

2-10. Eudora is served by a single well that pumps at a rate of 0.016 m^3/s. They anticipate the need for a pumping rate of 0.025 m^3/s. They would like to use the current well and replace the pump with a higher capacity pump. The artesian aquifer is 10 m thick with a piezometric surface 40 m above the bottom confining layer. The aquifer is a medium sand. After 300 days of pumping, the drawdown at a nonpumping well 200 m from the pumping well is 1 m. The pumping well is 1 m in diameter. Assuming a typical hydraulic conductivity for medium sand, determine the maximum allowable sustained pumping rate.

Hoko River near Sekiu, WA

Mean monthly discharge (m^3/s)

Year	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec
1963	12.1	15.0	8.55	9.09	5.78	1.28	2.59	1.11	.810	13.3	26.1	20.3
1964	27.3	12.2	18.0	8.21	4.08	3.62	4.53	2.44	4.28	7.67	13.3	14.7
1965	27.4	27.3	5.01	5.61	6.68	1.38	.705	.830	.810	7.31	16.8	19.6
1966	29.8	11.3	17.6	5.18	2.67	2.10	1.85	.986	1.54	10.3	17.0	39.0
1967	35.6	26.8	18.5	6.51	3.43	1.46	.623	.413	.937	25.7	14.2	27.8
1968	34.2	22.4	15.7	9.20	3.68	2.65	1.72	1.55	9.12	16.8	16.5	25.2
1969	17.2	18.5	12.9	12.8	3.74	2.23	1.19	.810	6.15	7.84	9.15	15.9
1970	17.3	12.1	8.50	17.7	3.85	1.32	.932	.708	4.22	7.96	13.9	25.4
1971	32.7	21.0	21.1	8.13	3.43	2.83	1.83	.932	2.22	10.7	22.7	22.0
1972	27.4	26.9	25.4	14.6	3.00	1.00	5.32	.841	2.00	1.14	11.8	37.8
1973	28.0	9.23	11.3	4.13	5.30	4.93	1.63	.736	.810	13.1	29.8	31.5

2-11. Your supervisor has asked you to make a first approximation estimate of the maximum allowable sustained pumping rate for a 1 m diameter well located in a confined aquifer. She has given you the well boring log shown below. Your firm uses a 2 m safety factor to ensure that the piezometric surface is not lowered below the aquiclude. She has said you may assume that the aquifer has a typical hydraulic conductivity. For a first trial assume that the drawdown in an observation well 100 m away from the pumping well is 0.0 m; that is, the pumping well's radius of influence is $\leq$ 100 m.

Typical well boring log

Aquifer material	Depth, m	
Clay and sand	12	static water level = 12 m
Fine gray sand	10	
Hardpan	2	
Shale	2	
Fractured rock	55	
Shale	2	well terminated

2-12. Your firm has been employed to perform a preliminary data analysis for renovation and expansion of a surface water treatment plant. The plant operator has provided the following summary of daily turbidity readings. Using a spreadsheet you have written, perform a cumulative probability analysis on the data shown below and prepare a plot on probability paper. Using the plot, write a short summary report for your supervisor that discusses the following:

- A general description of the behavior of the data.
- Your impression of the degree of difficulty in operating the plant and the need for operational flexibility.
- A recommendation regarding further data analysis.

Monthly Average Turbidity 2005–2007 for Lake Michigan

Month	Turbidity, NTU	Month	Turbidity, NTU	Month	Turbidity, NTU
1	6.89	13	4.26	25	5.76
2	4.63	14	2.95	26	4.37
3	3.42	15	2.07	27	3.59
4	1.40	16	1.56	28	2.31
5	1.25	17	1.11	29	0.66
6	0.91	18	0.59	30	0.73
7	1.18	19	1.01	31	0.84
8	0.79	20	0.59	32	0.67
9	1.07	21	1.09	33	0.87
10	1.06	22	2.70	34	2.09
11	5.41	23	1.07	35	1.68
12	6.15	24	4.51	36	6.20

2-13. Your firm has been employed to perform a preliminary data analysis for renovation and expansion of a surface water treatment plant. The plant operator has provided the following summary of daily turbidity readings. Using a spreadsheet you have written, perform a cumulative probability analysis on the data shown below and prepare a plot on probability paper. Using the plot, write a short summary report for your supervisor that discusses the following:

- A general description of the behavior of the data.
- Your impression of the degree of difficulty in operating the plant and the need for operational flexibility.
- A recommendation regarding further data analysis.

Daily Turbidity for the Alma River, January, 2005

Day	Turbidity, NTU	Day	Turbidity, NTU
1	11.50	17	8.47
2	5.53	18	7.10
3	7.40	19	6.47
4	5.83	20	3.77
5	3.35	21	3.60
6	2.80	22	2.65
7	3.00	23	2.77
8	3.20	24	2.63
9	2.75	25	2.30
10	2.47	26	2.10
11	1.95	27	2.03
12	4.00	28	2.10
13	63.67	29	1.95
14	59.60	30	2.15
15	24.33	31	2.00
16	12.70		

2-9 DISCUSSION QUESTIONS

2-1. Your design office has been contracted to design a municipal well field for a small village. The planning meeting has yielded the following information: no federal or state money will be involved in the project, the bond rate is 6%, the population growth rate is negligible. What design period would you recommend to the client? Explain your reasoning.

2-2. Explain why a favorable yield analysis alone is not a sufficient reason to select a surface water supply.

2-3. An intern has asked why a unit demand of less than 440 Lpcd was selected for a village of 2,000 in the upper peninsula of Michigan. Explain why. Would your answer be different if the village was in the southern half of Arizona? Why?

2-4. Can a well fail without "going dry"? Explain.

2-5. A probability plot of turbidity for a surface water results in a very steep slope. What does this imply for the difficulty of operating the plant?

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