CHAPTER TWO

DEMAND OF WATER

The need of a water development activity arises from the demand for water for some purpose. After the demand for a purpose is established, the availability of water resources in the vicinity of the demand center is assessed through application of the principles of hydrology. The reconciliation of the demand with the available resources in an optimal manner is the objective of water resources planning. Where the resources are restricted compared to the demand, as for irrigation in some regions, the problem is approached from a consideration of how much demand can be satisfied with the available water resources. There may be conflicting demands when more than one purpose is involved. These have to be resolved by establishing a priority ranking among water uses. The location, type, and components of a project as well as its functional characteristics are dependent on the purpose and magnitude of demands. Thus the project cost is a function of the demand. But the demand for water is affected to some extent by the cost of providing water via the project. Therefore, demand is not a static problem that can be finally determined at one time. The tentative estimates made initially are reviewed at a later stage in the planning process. The project estimates are also revised accordingly. The procedures for making tentative estimates of demand for each major purpose of development are discussed in subsequent sections.

2.1 DEMAND FOR WATER SUPPLY

Definitions of relevant terms follow.

Withdrawal uses are diversion of surface water or groundwater from its source of supply, such as irrigation and water supply. *Nonwithdrawal uses* are on-site uses such as navigation and recreation.

*Consumptive uses** are that portion of withdrawn quantity which is no longer available for further use because it is used up by crops, human beings, industrial plant processes, evapotranspiration, and so on.

Water supply requirements usually have the highest priority among the developmental uses, and water of good quality is needed. Although the total quantity of withdrawal in big cities may be relatively large, the consumptive water use is small since 80 to 90% of the total intake is returned to the river system (of course, its quality is degraded). Water requirements of a city can be divided into three broad categories:

1. Municipal requirements
2. Large industrial requirements
3. Waste dilution requirements

The order of magnitude of three kinds of requirements is indicated in Figure 2.1 for a typical city of a population of about 150,000. If the sewage and industrial waste

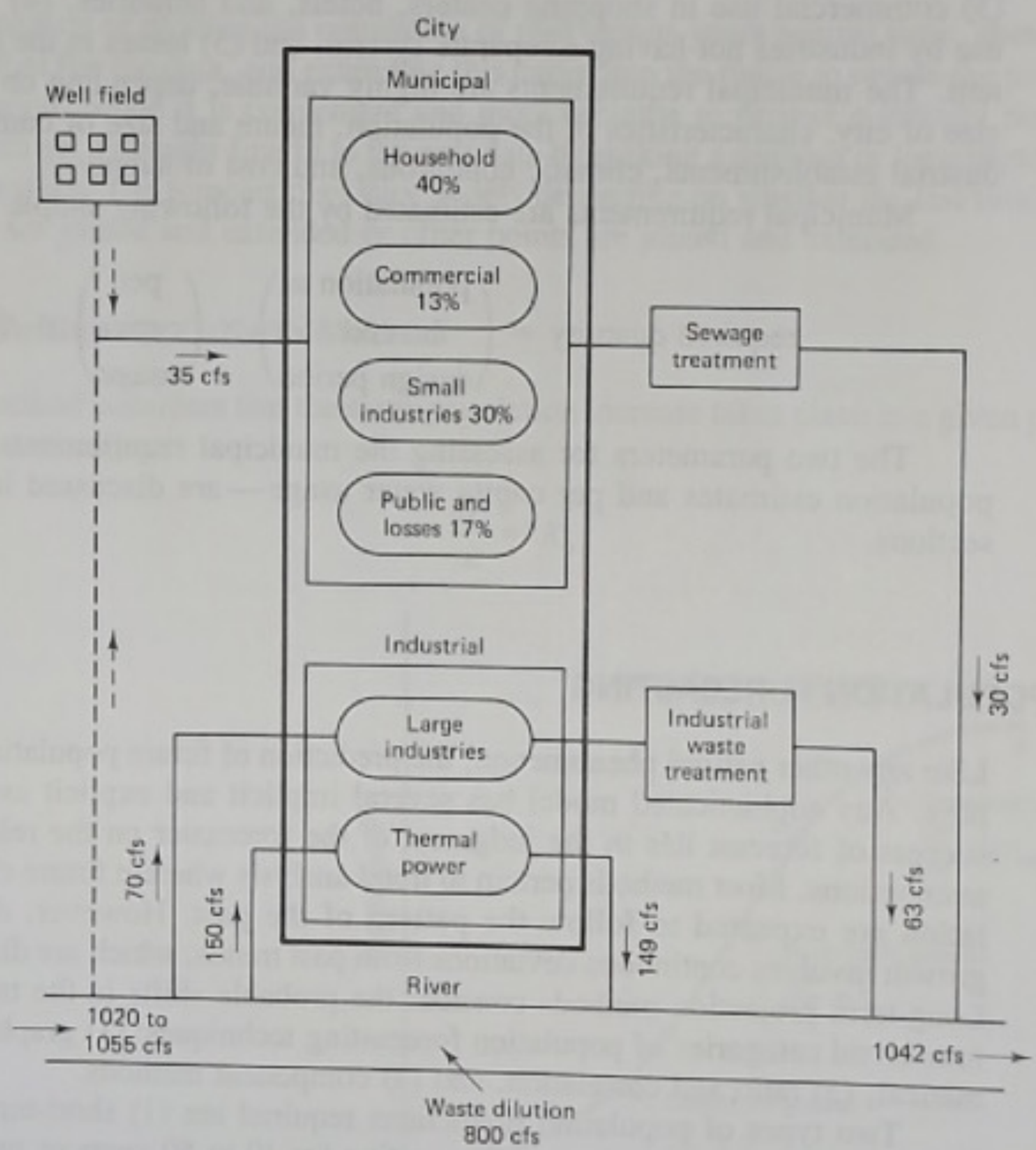

Figure 2.1 Water requirements of a city of 150,000 population.

*In the context of agriculture, the consumptive use requirement of a crop is defined as the amount of water needed for crop growth.

are discharged after a treatment, the waste dilution requirements can be reduced substantially as discussed subsequently. The total water supply requirement in a river system for a number of cities is not equal to the sum of the requirements of the individual city because the dilution requirement is a nonwithdrawal use that is available to all cities on the same river, and the consumptive requirement is only a small fraction of the total. Thus, if all cities are situated on the same river with sufficient distance in between for purification of the discharged wastes, the total requirements will be only slightly more than the largest city requirements.

2.2 MUNICIPAL REQUIREMENTS

This includes (1) such domestic uses as drinking, cooking, washing, sprinkling, and air conditioning, (2) public uses such as in public buildings and for firefighting, (3) commercial use in shopping centers, hotels, and laundries, (4) small industrial use by industries not having a separate system, and (5) losses in the distribution system. The municipal requirements are highly variable, depending on such factors as size of city, characteristics of the population, nature and size of commercial and industrial establishments, climatic conditions, and cost of supply.

Municipal requirements are estimated by the following simple relation:

$$\text{required quantity} = \begin{pmatrix}\text{population at}\\ \text{the end of}\\ \text{design period}\end{pmatrix} \times \begin{pmatrix}\text{per}\\ \text{capita}\\ \text{usage}\end{pmatrix} \qquad [L^3] \qquad (2.1)$$

The two parameters for assessing the municipal requirements in eq. (2.1)—population estimates and per capita water usage—are discussed in the following sections.

2.3 POPULATION FORECASTING

Like any other natural phenomenon, the prediction of future population is quite complex. Any sophisticated model has several implicit and explicit assumptions. The success of forecast lies in the judgment of the forecaster on the reliability of these assumptions. Most methods pertain to trend analysis wherein future changes in population are expected to follow the pattern of the past. However, dynamic human growth involves continuous deviations from past trends, which are difficult to assess. Long-term projection methods consider the probable shifts in the trends. There are four broad categories of population forecasting techniques: (1) graphical, (2) mathematical, (3) ratio and correlation, and (4) component methods.

Two types of population predictions required are (1) short-term estimates for 1 to 10 years, and (2) long-term forecasting for 10 to 50 years or more. The choice of methods to use for these two types of estimates is different, as described below.

2.4 SHORT-TERM ESTIMATES

Certain techniques from the categories of graphical and mathematical methods are used for short-term estimates. These methods are essentially trend analyses in graphic or mathematical form. The mathematical approach assumes three forms of population growth, shown by three segments of Figure 2.2. These are referred to as geometric growth, arithmetic growth, and declining rate of growth. Each segment has a separate relation. The historic population data of the study area may be plotted on a regular graph. According to the shape of the plot, the relationship of that segment should be used for population projection. The short-term methods are used for either intercensal estimates for any year between two censuses or postcensal estimates from the last census until the next census.

2.4.1 Graphical Extension Method

This consists of plotting the population of past census years against time, sketching a curve that fits the data, and extending this curve into the future to obtain the projected population. Since it is convenient and more accurate to project a straight line, it is aimed to get a straight-line fit to the past data by making a semilog or logarithmic plot, as necessary. The forecast may vary widely depending on whether the last two known points are joined and extended or other points are joined and extended.

2.4.2 Arithmetic Growth Method

This method considers that the same population increase takes place in a given period. Mathematically,

$$\frac{dP}{dt} = K_a$$

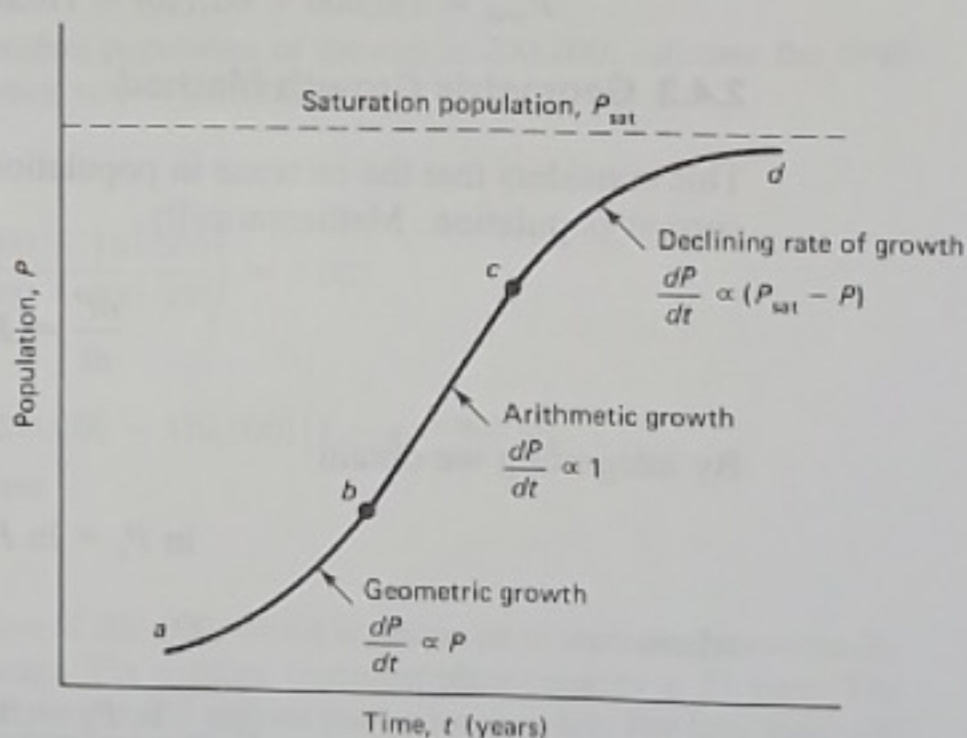

Figure 2.2 Population growth curve.

where

$$P = \text{population}$$
$$t = \text{time, years}$$
$$K_a = \text{uniform growth-rate constant}$$

By integrating the equation above, we obtain

$$P_t = P_0 + K_a t \tag{2.2}$$

where

$$P_t = \text{projected population } t \text{ years after } P_0$$
$$P_0 = \text{present population}$$
$$t = \text{period of projection}$$

and

$$K_a = \frac{P_2 - P_1}{\Delta t} \quad [\mathrm{T}^{-1}] \tag{2.3}$$

where P_1 and P_2 are recorded populations at some Δt interval apart.

Example 2.1

The population of a city has been recorded in 1970 and 1985 as 100,000 and 110,000, respectively. Estimate the 1995 population, assuming arithmetic growth.

Solution From eq. (2.3),

$$K_a = \frac{110{,}000 - 100{,}000}{15} = 667$$

From eq. (2.2),

$$P_{1995} = 110{,}000 + 667(10) = 116{,}667 \text{ persons}$$

2.4.3 Geometric Growth Method

This considers that the increase in population takes place at a constant percent of the current population. Mathematically,

$$\frac{dP}{dt} = K_p P$$

By integrating we obtain

$$\ln P_t = \ln P_0 + K_p t \tag{2.4}$$

where

$$K_p = \frac{\ln P_2 - \ln P_1}{\Delta t} \quad [\mathrm{T}^{-1}] \tag{2.5}$$

Example 2.2

In Example 2.1, estimate the population, assuming geometric growth.

Solution From eq. (2.5),

$$K_p = \frac{\ln 110{,}000 - \ln 100{,}000}{15} = 0.0064$$

From eq. (2.4),

$$\ln P_{1995} = \ln 110{,}000 + 0.0064(10) = 11.67$$

$$P_{1995} = 117{,}216 \text{ persons}$$

2.4.4 Declining Growth Rate Method

This assumes that the city has a saturation population and the rate of growth becomes less as the population approaches the saturation level. In other words, the rate of increase is a function of the population deficit $(P_{sat} - P)$, that is,

$$\frac{dP}{dt} = K_D(P_{sat} - P)$$

Upon integration, we have

$$P_t = P_0 + (P_{sat} - P_0)(1 - e^{-K_D t}) \tag{2.6}$$

Rearranging eq. (2.6) gives

$$K_D = -\frac{1}{\Delta t}\ln\left(\frac{P_{sat} - P_2}{P_{sat} - P_1}\right) \qquad [\mathrm{T}^{-1}] \tag{2.7}$$

Example 2.3

In Example 2.1, if the saturation population of the city is 200,000, estimate the 1995 population. Assume a declining rate of growth.

Solution From eq. (2.7),

$$K_D = -\frac{1}{15}\ln\left(\frac{200{,}000 - 110{,}000}{200{,}000 - 100{,}000}\right) = 0.007$$

From eq. (2.6),

$$P_{1995} = 110{,}000 + (200{,}000 - 110{,}000)[1 - e^{-(0.007)(10)}]$$
$$= 116{,}085 \text{ persons}$$

Example 2.4

A city has a present population of 200,000, which is estimated to increase geometrically to 220,000 in the next 15 years. The existing treatment plant capacity is 51 mgd. The rate of input to the treatment plant is 165 gallons per person per day. For how long will the treatment plant be adequate?

Solution

1. From the known population data:

$$\ln 220{,}000 = \ln 200{,}000 + K_p(15)$$

or $K_p = 0.00635$.

2. Population that can be served by the plant:

$$\frac{51.0(10^6)}{165} = 309{,}090 \text{ persons}$$

3. Time to reach the design population:

$$\ln 309090 = \ln 200000 + 0.00635(t)$$

or $t = 68.55$ years.

2.5 LONG-TERM FORECASTING

Long-term predictions are made by techniques from all four categories. The entire past record of historic population data is used in long-term predictions. The mathematical curve-fitting approach is popular because it is relatively easy to apply. McJunkin (1964) indicates, however, that the component and ratio methods offer greater reliability than the traditional graphical–mathematical methods.

2.5.1 Graphic Comparison Method

Several larger cities in the vicinity are selected whose earlier growth exhibited characteristics similar to those of the study area. The population–time curves for these cities and for the study area are plotted as shown in Figure 2.3. From point O' corresponding to the last known population for study area A a horizontal line is drawn intersecting the other curves at O. At O', lines parallel to OB, OC, and OD are drawn as $O'B'$, $O'C'$, and $O'D'$, respectively. These lines establish a range of future growth within which $O'A'$ is extended. This method has a shortcoming since it is not certain that the future growth of the study area will be similar to the past growth of the other areas.

2.5.2 Mathematical Logistic Curve Method

This method is suitable for the study of large population centers such as large cities, states, or nations. On the basis of the study of the growth curve of Figure 2.2, certain mathematical equations of an empirical curve conforming to this shape (S-shape) were proposed. One of the best known functions is the logistic curve in the form

$$P_t = \frac{P_{sat}}{1 + ae^{bt}} \tag{2.8}$$

where

P_t = population at any time t from an assumed origin
P_{sat} = saturation population
a, b = constants

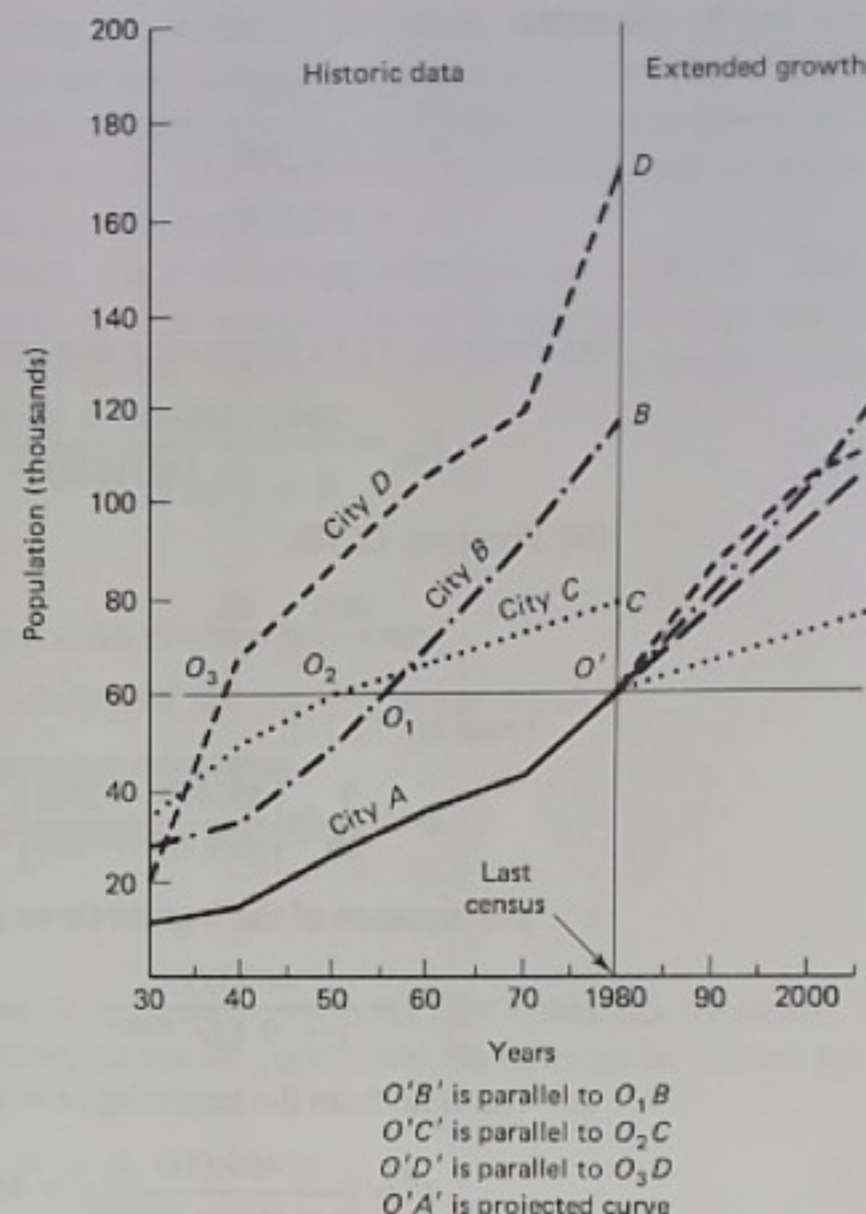

Figure 2.3 Graphical projection by comparison.

The constants are determined by selecting three populations from the record: one in the beginning, P_0, one in the middle, P_1, and one near the end of the record, P_2, associated with the years T_0, T_1, and T_2 such that the number of years (interval) between T_0 and T_1 is the same as that between T_1 and T_2. This interval between T_0 and T_1 is designated as n. The constants are given by

$$P_{sat} = \frac{2P_0P_1P_2 - P_1^2(P_0 + P_2)}{P_0P_2 - P_1^2} \tag{2.9}$$

$$a = \frac{P_{sat} - P_0}{P_0} \tag{2.10}$$

$$b = \frac{1}{n} \ln\left[\frac{P_0(P_{sat} - P_1)}{P_1(P_{sat} - P_0)}\right] \tag{2.11}$$

In eq. (2.8), time t is counted from the year T_0. From eq. (2.9), P_{sat} must be positive and must exceed the latest known population. A test of the validity of logistic growth is that the population data plot as a straight line on specially scaled (logistic) graph paper. (The graph of $\log[(P_{sat} - P)/P]$ versus t is a straight line.)

Example 2.5

In two 20-year periods, a city grew from 45,000 to 258,000 to 438,000. Estimate (a) the saturation population, (b) the equation of the logistic curve, and (c) the population 40 years after the last period.

Solution

$$P_0 = 45{,}000$$
$$P_1 = 258{,}000$$
$$P_2 = 438{,}000$$
$$n = 20$$

(a) From eq. (2.9) (expressing numbers in thousand),

$$P_{sat} = \frac{2(45)(258)(438) - (258)^2(45 + 438)}{(45)(438) - (258)^2} = 469{,}000$$

(b) From eq. (2.10),

$$a = \frac{469 - 45}{45} = 9.42$$

From eq. (2.11),

$$b = \frac{1}{20}\ln\left[\frac{45(469 - 258)}{258(469 - 45)}\right] = -0.122$$

The equation of the logistic curve:

$$P_t = \frac{469{,}000}{1 + 9.42e^{-0.122t}} \tag{2.12}$$

(c) The time from the beginning, $t = 40 + 40 = 80$ years; thus

$$P = \frac{469{,}000}{1 + 9.42e^{-0.122(80)}} = 468{,}740 \text{ persons}$$

2.5.3 Ratio and Correlation Methods

A city or smaller area is a part of a region, state, nation, or larger area. There are many factors and influences affecting population growth occur throughout the region. Thus the growth of the smaller area has some relation to the growth of the larger area. Because a careful projection of the future population of the nation and states (larger area) is made by the authoritative organizations, these may be used to forecast the growth of the smaller area.

In the simplest technique, a constant ratio obtained from the most recent data is used as follows:

$$K_r = \frac{P_i}{P_i'}$$
$$P_t = K_r P_t' \tag{2.13}$$

where

P_i = population of study area at last census
P_i' = population of larger area at last census
P_t = future population for study area
P_t' = estimated future population of larger area
K_r = constant

In a refined technique using the variation in ratios, the ratios of the smaller area to the larger area population are calculated for a series of census years. By using any of the graphical or mathematical methods of short-term estimates (Section 2.4), the trend line of the ratios is projected. The projected ratio in the year of interest is applied to an estimate of the study area population.

In another statistical method, the correlation technique described in Chapter 7 is applied. Between the two series of census data relating to the study area and larger area, a relation is established through the regression analysis. For example, a simple regression equation may be of the following form:

$$P_f = aP'_f + b \tag{2.14}$$

where

P_f = population of the study area
P'_f = population of the region (larger area)
a, b = regression constants

The future population is projected from eq. (2.14).

2.5.4 Component Methods

A population change can occur in only three ways: (1) by birth, (2) by death, and (3) through migration. These components of population change can be linked by the balance equation:

$$P_t = P_0 + B - D \pm M \tag{2.15}$$

where

P_t = forecast population at the end of time t
P_0 = existing population
B = number of births during time t
D = number of deaths during time t
M = net number of migrants during time t (positive value indicates moving into the study area)

Because migration affects the births and deaths in an area, the estimates of net migration are made before estimating the natural change due to births and deaths. The migratory trends may be estimated by applying eq. (2.15) backward to the past census data on population, births, and deaths during a selected period. The school attendance method, comparing the actual children enrollment to the children from birth records, is also used for estimating migration.

For determining the natural change due to births and deaths, the simplest procedure is to multiply the existing population by the expected birth and death rates, that is,

$$B = K_1 P_0 \Delta t \tag{2.16}$$

$$D = K_2 P_0 \Delta t \tag{2.17}$$

where

$$K_1, K_2 = \text{birth and death rate, respectively}$$
$$\Delta t = \text{forecast period}$$

Better estimates of natural change (births and deaths) are made by the cohort-survival technique, which makes projections to each subcomponent (factor) related to the natural change.

2.6 PER CAPITA WATER USAGE

Per capita use is normally expressed as the average daily rate, which is the mean annual usage of water averaged for a day in terms of gallons (or liters) per capita per day (gpcd). The seasonal, monthly, daily, and hourly variations in the rate are given in percentages of the average. Which of these should be used for the design capacity depends on the component of the water supply system. The layouts of two water supply systems—one for direct pumping from a river or from a wellfield and one for an impounding reservoir—are shown in Figure 2.4. The period of design for which the population projection is to be made and the design capacity criteria of different component structures of the systems are indicated in Table 2.1.

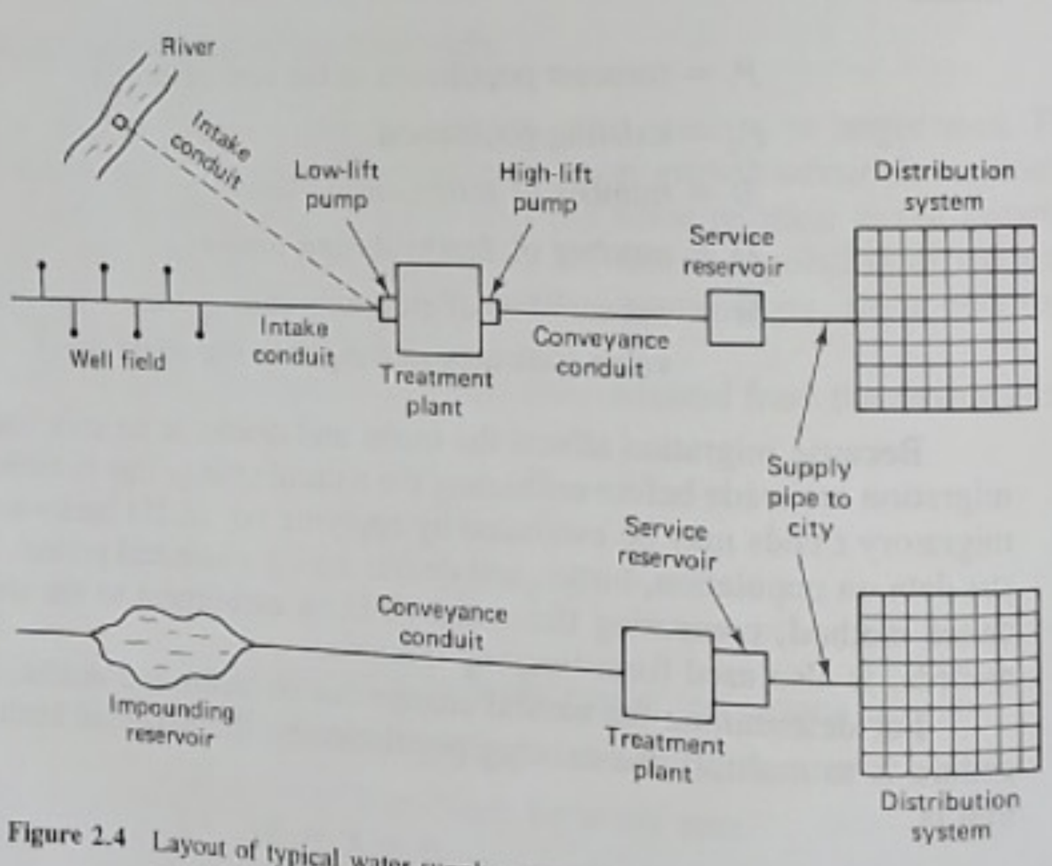

Figure 2.4 Layout of typical water supply systems. [from Fair, Geyer, and Okun (1966)]

TABLE 2.1 DESIGN PERIODS AND CAPACITY CRITERIA FOR CONSTITUENT STRUCTURES

Structure	Design Period[a] (years)	Required Capacity
1. Source of supply		
a. River	Indefinite	Maximum daily (requirements)
b. Wellfield	10–25	Maximum daily
c. Reservoir	25–50	Average annual demand
2. Conveyance		
a. Intake conduit	25–50	Maximum daily
b. Conduit to treatment plant	25–50	Maximum daily
3. Pumps		
a. Low-lift	10	Maximum daily plus one reserve unit
b. High-lift	10	Maximum hourly plus one reserve unit
4. Treatment plant	10–15	Maximum daily
5. Service reservoir	20–25	Working storage (from hourly demand and average pumping) plus fire demand plus emergency storage
6. Distribution		
a. Supply pipe or conduit	25–50	Greater of (1) maximum daily plus fire demand or (2) maximum hourly requirement
b. Distribution grid	Full development	(same as 6a)

[a]"Design period" does not necessarily indicate the life of the structure. A design period takes into account other factors, such as subsequent ease of extension, rate of population growth and shifts in community, and industrial/commercial developments.

2.6.1 Average Daily per Capita per Day Usage

For household use, the per capita requirements of water range between 20 and 90 gallons per day, with a reasonable average of 60 gallons per day. Municipal water use should, however, also include commercial use, small industrial use, public use, and losses in the system. A typical distribution for an average city is given in Table 2.2.

TABLE 2.2 TYPICAL AVERAGE USAGE

Use	Average Use (gpcd)[a]	Percent of Total
Household	60	40
Commercial	20	13
Industrial	45	30
Public	15	10
Loss	10	7
Total	150	100

[a]Liter = gallons × 3.8.

The U.S. Water Resources Council (1968) has projected average usage in the year 2000 at 175 gpcd. When the industrial requirements become relatively important in a city, they should be considered separately.

2.6.2 Variations in Usage

The values in Table 2.2 refer to the daily average of the long-term (many years) usage. The consumption changes with the seasons, varies from day to day in the week, and fluctuates from hour to hour in a day. Knowledge of these variations is important for the design of project components, as indicated in Table 2.1. There are two common trends: (1) the smaller the city, the more variable is the demand; and (2) the shorter the period of flow, the wider is the variation from the average (i.e., the hourly peak flow is much higher than the daily peak). Typical variation in a city water supply in a day is shown in Figure 2.5. The variations are commonly indicated in terms of the percentage of the long-term average value. There are no fixed ratios; each city has its own trend. However, in the absence of data, the following formula of R. O. Goodrich is very convenient for estimating the maximum usage from 2 hours (2/24 day) to a year (365 days) for small cities.

$$p = \frac{180}{t^{0.1}} \quad \text{[unbalanced]} \tag{2.18}$$

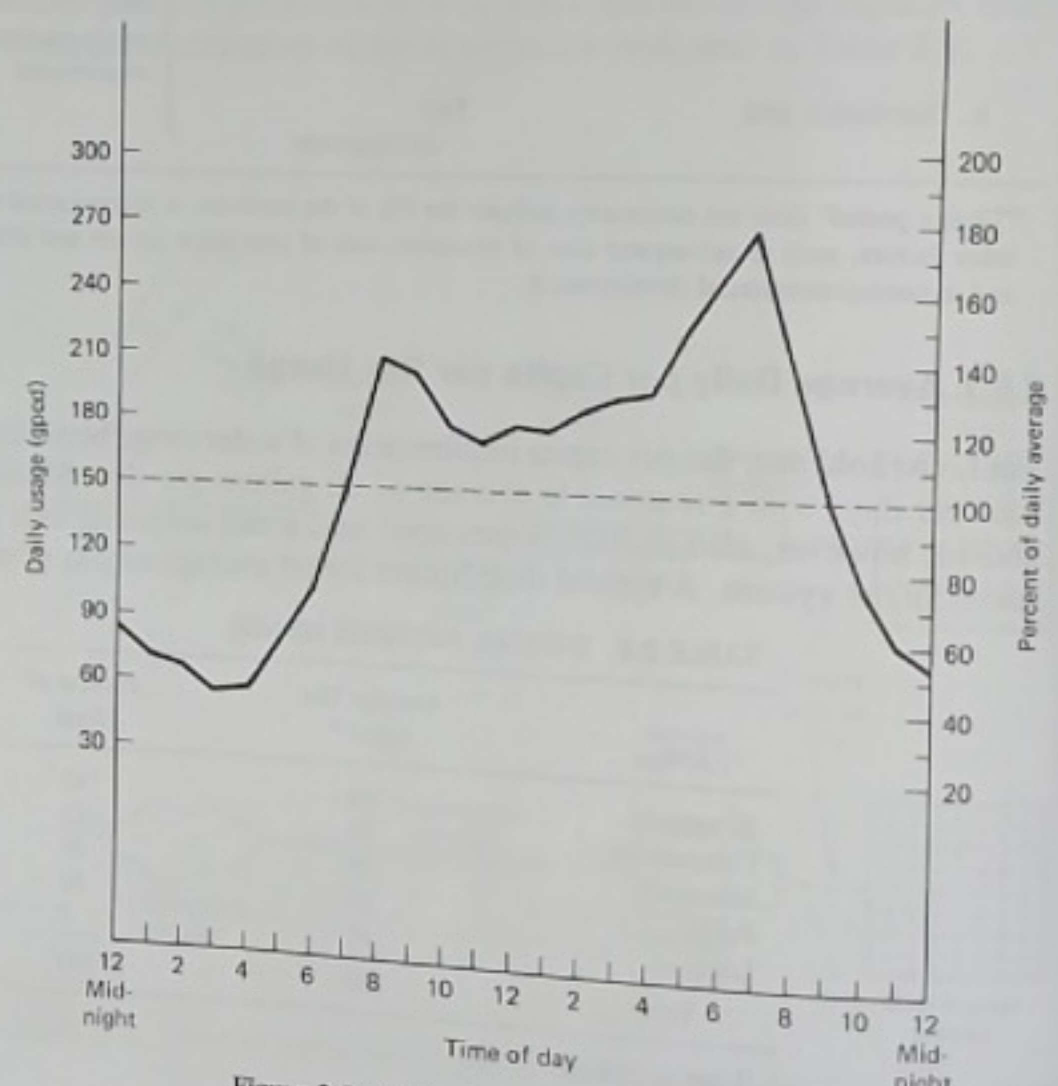

Figure 2.5 Typical variation in usage in a day.

where

$$p = \text{percentage of annual average daily usage}$$
$$t = \text{time, days}$$

From eq. (2.18), the maximum daily use is 180% of the (long-term) average daily usage, and the maximum monthly use is 128%. Larger cities may have smaller peaks.

The maximum hourly consumption in any day is likely to be 150% of the usage for that day (Steel and McGhee, 1979). For a distribution system, the fire demands have also to be added. It is unlikely that water will be drawn at the maximum hourly rate while a serious fire is raging. Hence the capacity is based on the maximum daily usage plus fire demand or maximum hourly usage, whichever is greater.

For pump design, the information on minimum flow rate which is considered to be 25 to 50% of the average daily flow is also important.

Example 2.6

For a city having an average daily water usage of 150 gpcd from the municipal supply, determine the maximum hourly requirement (excluding the fire demands).

Solution

1. Maximum daily usage = 180% of average daily

$$= \frac{180}{100}(150) = 270 \text{ gpcd}$$

2. For a maximum day,

$$\text{maximum hourly usage} = 150\% \text{ of daily use}$$
$$= \frac{150}{100}(270) = 405 \text{ gpcd}$$

2.6.3 Fire Demand

Although the total quantity of water used annually for firefighting purposes (which is included under the category of "public use") is very small, whenever demand rises, the rate of withdrawal is high. The service reservoir and distribution system should make provisions for the fire demands in their capacities.

Two empirical formulas have been suggested by insurance-related organizations for estimating fire demands. The American Insurance Association has suggested the following relation based on population:

$$Q = 1020\sqrt{P}\,(1 - 0.01\sqrt{P}) \qquad \text{[unbalanced]} \tag{2.19}$$

where

$$Q = \text{required fire flow, gpm}$$
$$P = \text{population, thousands}$$

On the basis of construction type, floor area, and occupancy, the Insurance Services Office proposed

$$Q = 18C\sqrt{A} \qquad \text{[unbalanced]} \tag{2.20}$$

where

Q = required fire flow, gpm

A = total floor area excluding the basement, ft^2

C = coefficient: 1.5 for wood frame construction, 1.0 for ordinary construction, 0.8 for noncombustible construction, and 0.6 for fire-resistant construction

The following limitations apply to eq. (2.20):

1. Flow should not exceed 6000 gpm for a single story, 8000 gpm for a single building, or 12,000 gpm for a single fire.
2. Flow should not be less than 500 gpm.

The duration for which the fire flow has to be maintained is given in Table 2.3. If the time periods indicated cannot be maintained, the community insurance rates are adjusted upward by the insurance companies.

TABLE 2.3 DURATION FOR FIRE FLOW[a]

Required Fire Flow (gpm)	Duration (hr)
<1000	4
1000–1250	5
1250–1500	6
1500–1750	7
1750–2000	8
2000–2250	9
>2250	10

[a] *Source:* Steel and McGhee (1979)

Example 2.7

A city with a population of 20,000 has an average daily usage of 150 gpcd. Determine the fire demand and the design capacity for different components of a water supply project. The working service storage is 1.5 mgd.

Solution

1. From Example 2.6,

$$\text{maximum daily usage} = 270 \text{ gpcd}$$
$$\text{maximum hourly usage} = 405 \text{ gpcd}$$

2. Average daily draft $= \dfrac{150(20{,}000)}{10{,}000} = 3$ mgd.
3. Maximum daily draft $= \dfrac{270(20{,}000)}{1{,}000{,}000} = 5.4$ mgd.
4. Maximum hourly draft $= \dfrac{405(20{,}000)}{1{,}000{,}000} = 8.1$ mgd.
5. Fire flow [from eq. (2.19)]:

$$Q = 1020\sqrt{20}\,(1 - 0.01\sqrt{20}) = 4358 \text{ gpm} \quad \text{or} \quad 6.27 \text{ mgd}$$

6. Maximum daily + fire flow = 5.4 + 6.27 = 11.67 mgd.

7. Pumps: Assume that the required flow is handled by three units and that one reserve unit is installed.
   a. Low-lift pumps $= \frac{4}{3}$(maximum daily) $= \frac{4}{3}(5.4) = 7.2$ mgd
   b. High-Lift pumps $= \frac{4}{3}$(maximum hourly) $= \frac{4}{3}(8.1) = 10.8$ mgd
8. Service reservoir:
   a. Fire flow duration (from Table 2.3) = 10 hr
   b. Total quantity of fire flow in a day $\frac{10}{24}(6.27) = 2.61$ mgd.
   c. Working storage (given) = 1.5 mgd.
   d. Emergency storage = (days) (average daily draft) = 3(3) = 9 mgd.
   e. Service storage = 2.61 + 1.5 + 9 = 13.11 mgd.
9. Design capacities:

Structure	Basis	Capacity (mgd)
River flow	Maximum daily	5.4
Intake conduit, conduit to treatment	Maximum daily	5.4
Low-lift pumps	Maximum daily plus reserve	7.2
High-lift pumps	Maximum hourly plus reserve	10.8
Treatment plant	Maximum daily	5.4
Service storage	Working storage plus fire plus emergency	13.11
Distribution system	Maximum daily plus fire or maximum hourly	11.67

2.7 INDUSTRIAL REQUIREMENTS

Table 2.2 included a typical industrial water component from a public water supply system. This is not adequate for a community with large water-using industries. The large industries are usually served by separate supplies. About 5% of the industries use 80% of the industrial water demands; 70% of the industrial plants use as little as 2% of the demands. Thermal power stations are the heaviest water users, with a requirement of 600 gpcd or 80 gal/kWh on the average. Other major water-using industries are steel, paper, and beverages. The average industrial requirement for the entire country (excluding thermal power stations) is 250 to 300 gpcd. The requirements of individual industries per unit of the production are indicated in Table 2.4.

More than 90% of all the water used for industries, including thermal power stations, is for cooling purposes. Many measures can be taken to reduce the cooling requirements, such as constructing an artificial pond, recirculating the cooling water or using poor-quality water from a different source. The figures listed are applicable

TABLE 2.4 REQUIREMENTS OF MAJOR INDUSTRIES

Industry	Average Water Use
Thermal power	80 gal/kWh
Steel	35,000 gal/ton
Paper	50,000 gal/ton
Woolens	140,000 gal/ton
Coke	3,600 gal/ton
Oil refining	770 gal/barrel

when sufficient water is available. These can be substantially reduced—in the case of some industries by one-tenth—when water is scarce.

2.8 WASTE DILUTION REQUIREMENTS

In earlier times it was a common practice to dump raw municipal and industrial wastes into the same river that served as the source of supply, thus relying on the self-purification properties of the stream. As long as the streamflow is at least 40 times that of the wasteflow and there is a sufficiently long reach of river to the next city, both nuisance and unsafe conditions can be avoided. But with the rapid growth of cities and industrial activities and with increased use of water, the dumping of raw wastes into rivers is no longer permitted. The problem now is to what extent the municipal and industrial wastes* should be treated before discharge into the river. The amount of streamflow required for sufficient natural treatment of municipal and industrial wasteflow is a function of the pollutant characteristics of the waste- and streamflow properties with regard to oxygen content, dissolved minerals, water temperature, and length of the available downstream reach. This relationship is depicted in many models, the most common being the oxygen sag curve. For average conditions it has been found that the raw (fully untreated) waste from municipal and industrial sources, excluding thermal power plants, requires a ratio of streamflow to wasteflow of 40, and thoroughly treated waste requires a ratio of 2, with a linear variation in between as shown in Figure 2.6 (Kuiper, 1965).

If water supply is being planned from a reservoir project, there are three annual cost components to be considered: (1) cost of storage to provide the municipal (and industrial) requirements, (2) cost of storage to produce the required quantity for waste dilution, and (3) cost of waste treatment. Items (2) and (3) act opposite to each other, that is, when the degree of treatment is increased, the cost of treatment rises but the cost of waste dilution storage decreases, and vice versa. The various degrees of treatment and the annual costs associated with them are considered until the lowest cost is found that indicates the most economic treatment of the city waste.

Example 2.8

A city had a total withdrawal (excluding in-stream dilution requirements) of 140 mgd in 1987 distributed as follows: municipal usage, 30 mgd; manufacturing industries, 35 mgd;

*Industrial wastes have to be considered even if the industrial supplies are developed from a different source than the municipal supplies.

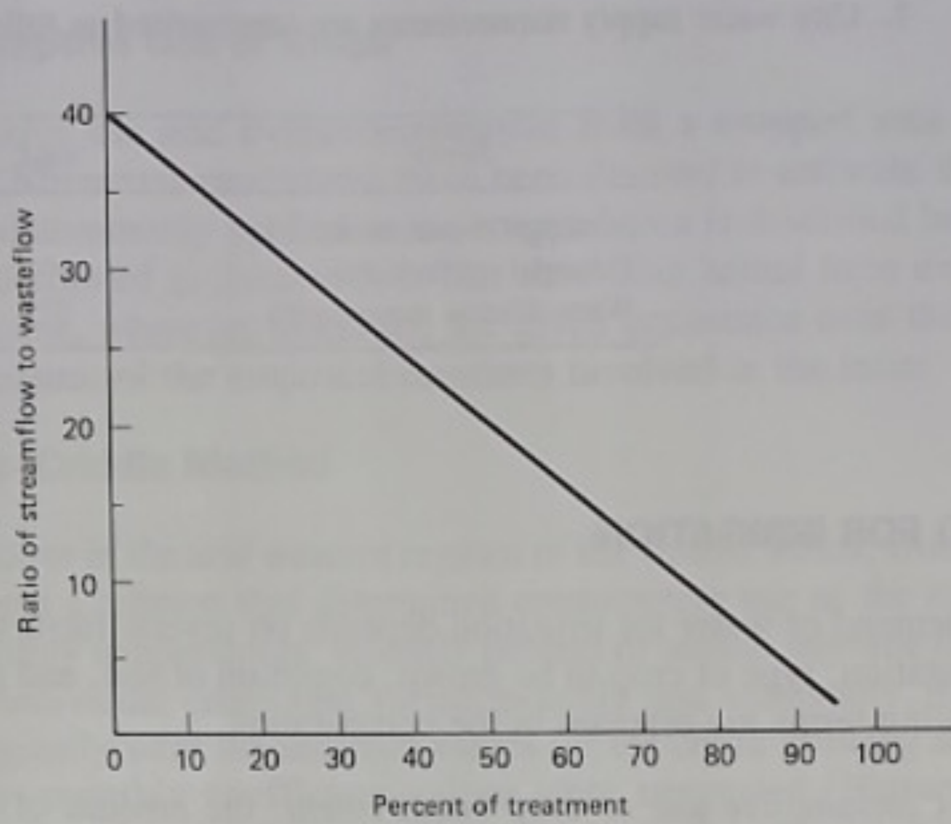

Figure 2.6 Requirements for waste dilution.

and thermal power, 75 mgd. The city had a population of 200,000, which is expected to rise to 220,000 in the year 2000. An industrial expansion of 20% and a thermal power increase of 80 MW is expected in the city by 2000. Estimate the total withdrawal in the year 2000. If the waste is to be discharged after 80% of the treatment, determine the total requirements of water.

Solution

1. Existing per capita municipal usage $= \frac{30(10^6)}{200{,}000} = 150$ gpcd.

   In the year 2000:

2. Municipal requirements $= \frac{(220{,}000)(150)}{1{,}000{,}000} = 33$ mgd.
3. Manufacturing industry requirements $= \frac{120}{100}(35) = 42$ mgd.
4. Thermal power requirements:
   a. Assuming a plant capacity factor of 0.6, the additional energy produced per day is

   $$(80 \text{ MW})\frac{(24 \text{ hr})}{(1 \text{ day})}(0.6)\frac{(1000 \text{ kW})}{(1 \text{ MW})} = 1.15 \times 10^6 \text{ kWh}$$

   b. From Table 2.4, water usage = 80 gal/kWh.
   c. Additional water required $= \frac{80(1.15 \times 10^6)}{10^6} = 92$ mgd.
   d. Total thermal power requirements = 75 + 92 = 167 mgd.
5. Total for municipal and manufacturing usage = 33 + 42 = 75 mgd.
6. Waste dilution requirements:
   a. From Figure 2.5, for 80% treatment the streamflow/wasteflow ratio = 9.
   b. Dilution requirement = 9(75) = 675 mgd.

7. City water supply requirements are summarized as follows:

Sector	mgd
Municipal requirements	33
Industrial requirements	209
Waste dilution requirements	675

2.9 DEMAND FOR IRRIGATION

The demand of water for irrigation depends on several factors, including the method of irrigation, type of crop to be grown, condition of soil, and prevailing climate. The following terms are relevant in the computation.

1. *Consumptive use or crop requirement:* the amount of water needed for crop growth.
2. *Irrigation requirement:* consumptive use *minus* effective rainfall available for plant growth. To this quantity the following items, whichever is applicable, are included: (a) irrigation applied prior to crop growth should be added; (b) water required for leaching should be added; (c) miscellaneous requirements of germination, frost protection, plant cooling, and so on, should be added; and (d) decrease in soil moisture should be subtracted.
3. *Farm delivery requirement:* irrigation requirement *plus* farm losses due to evaporation, deep percolation, surface waste, and nonproductive consumption. The losses are measured by the on-farm irrigation efficiency, which is the percent of farm-delivered water that remains in the root zone and is available for crop growth.
4. *Gross water requirement:* farm delivery requirement *plus* the seepage losses in the canal system from the headworks to the farm unit *plus* the waste of water due to poor operation.

The gross water requirement is indicated in terms of the depth of water over the irrigable area. The basic quantity of interest is the consumptive use of the crops, from which the effective precipitation is subtracted and various losses are added to establish the gross irrigation demand.

A considerable quantity of the water applied to farmland returns to the river system. This includes surface runoff during irrigation, wasted water, canal seepage, and deep percolation. This may be from 30 to 60% of the gross requirement. About one-half of this reaches the river through groundwater flow. The remainder reaches the river as surface runoff during the irrigation season and thus becomes available for use to downstream projects.

Irrigation projects must include subsurface drainage facilities, as a measure against waterlogging and salinity. Drainage is described in Chapter 12.

2.9.1 Consumptive Use of Crops

The consumptive use and evapotranspiration from a cropped area are considered synonymous. Numerous procedures have been devised to estimate the consumptive use. A method commonly applied to the cropped area is described here. Some other methods are discussed in Section 3.8. The data from actual farm experience or experimental basins, wherever available, are given preference over the computational procedures because of the empirical constants involved in the latter.

2.9.2 Blaney–Criddle Method

For the conditions in the arid western regions of the United States, Blaney and Criddle (1945) proposed a relation that determined consumptive use as the multiplication of the mean monthly temperatures, monthly percent of annual daytime hours, and a coefficient for individual crops that varied monthly and seasonally. The coefficients presented originally were the seasonal values for the entire growing season of crops. Subsequently, monthly coefficient values were suggested (Blaney and Criddle, 1962). However, these coefficients did not include the effects of humidity, wind movement, and other climatological factors. The modified Blaney–Criddle method (U.S. SCS, 1964) split the coefficient in two parts to consider these factors indirectly. The modified formula is

$$U = \sum K_t K_c t_m \frac{p}{100} \quad \text{[unbalanced]} \tag{2.21}$$

where

U = consumptive use for any specified growth period, in.
K_t = climatic coefficient related to mean monthly temperatures
K_c = growth stage coefficient
t_m = mean monthly temperature, °F
p = monthly percentage of annual daytime hours (Table 2.5)

The values of K_t are based on the formula

$$K_t = 0.0173 t_m - 0.314 \quad \text{[unbalanced]} \tag{2.22}$$

For $t_m < 36$ °F, use $K_t = 0.30$.

The monthly values of K_c are obtained from Table 2.6(a) for a perennial crop with a year-round growing season. For other seasonal crops, Table 2.6(b) is used, based on the percentage of the growing season covered by the month in question. The monthly consumptive amounts are summed over the growing season to obtain the seasonal consumptive use.

Example 2.9

For the growing season of sugar beets at Limberly, Idaho, located at latitude 42°4 N, the long-term mean monthly air temperatures are given in column 2 of Table 2.7. The crop is planted April 10 and harvested October 15. Estimate the seasonal consumptive use of water.

TABLE 2.5 MONTHLY PERCENTAGE OF DAYTIME HOURS OF THE YEAR

	Latitude (degrees north of equator)													
Month	24	26	28	30	32	34	36	38	40	42	44	46	48	50
Jan.	7.58	7.49	7.40	7.30	7.20	7.10	6.99	6.87	6.76	6.62	6.49	6.33	6.17	5.98
Feb.	7.17	7.12	7.07	7.03	6.97	6.91	6.86	6.79	6.73	6.65	6.58	6.50	6.42	6.32
Mar.	8.40	8.40	8.39	8.38	8.37	8.36	8.35	8.34	8.33	8.31	8.30	8.29	8.27	8.25
Apr.	8.60	8.64	8.68	8.72	8.75	8.80	8.85	8.90	8.95	9.00	9.05	9.12	9.18	9.25
May	9.30	9.38	9.46	9.53	9.63	9.72	9.81	9.92	10.02	10.14	10.26	10.39	10.53	10.69
June	9.20	9.30	9.38	9.49	9.60	9.70	9.83	9.95	10.08	10.21	10.38	10.54	10.71	10.93
July	9.41	9.49	9.58	9.67	9.77	9.88	9.99	10.10	10.22	10.35	10.49	10.64	10.80	10.99
Aug.	9.05	9.10	9.16	9.22	9.28	9.33	9.40	9.47	9.54	9.62	9.70	9.79	9.89	10.00
Sept.	8.31	8.31	8.32	8.34	8.34	8.36	8.36	8.38	8.38	8.40	8.41	8.42	8.44	8.44
Oct.	8.09	8.06	8.02	7.99	7.93	7.90	7.85	7.80	7.75	7.70	7.63	7.58	7.51	7.43
Nov.	7.43	7.36	7.27	7.19	7.11	7.02	6.92	6.82	6.72	6.62	6.49	6.36	6.22	6.07
Dec.	7.46	7.35	7.27	7.14	7.05	6.92	6.79	6.66	6.52	6.38	6.22	6.04	5.86	5.65
Annual	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

TABLE 2.6 CROP-GROWTH-STAGE COEFFICIENT K_c (MODIFIED BLANEY–CRIDDLE METHOD)

(a) PERENNIAL CROPS (NORTHERN HEMISPHERE)

Crop	Average k_c values by months											
	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
Alfalfa	0.63	0.74	0.86	0.99	1.09	1.13	1.11	1.06	0.99	0.90	0.78	0.65
Grass pasture	0.48	0.58	0.74	0.85	0.90	0.92	0.92	0.91	0.87	0.79	0.67	0.55
Grapes	0.20	0.23	0.32	0.49	0.70	0.80	0.81	0.76	0.66	0.50	0.35	0.25
Citrus orchards	0.64	0.66	0.68	0.70	0.71	0.72	0.72	0.71	0.70	0.68	0.66	0.64
Deciduous, with cover	0.63	0.74	0.86	0.98	1.09	1.13	1.12	1.06	0.99	0.90	0.78	0.65
Deciduous, no cover	0.17	0.25	0.39	0.63	0.87	0.96	0.95	0.82	0.53	0.30	0.20	0.16
Avocados	0.27	0.42	0.58	0.71	0.78	0.81	0.78	0.71	0.63	0.54	0.43	0.36
Walnuts	0.10	0.14	0.23	0.43	0.68	0.92	0.98	0.88	0.69	0.49	0.31	0.15

(b) ANNUAL CROPS

Crop	K_c values at listed % of growing season										
	0	10	20	30	40	50	60	70	80	90	100
Field corn (grain)	0.44	0.49	0.58	0.71	0.93	1.05	1.08	1.06	1.01	0.93	0.85
Field corn (silage)	0.44	0.48	0.55	0.65	0.80	0.97	1.06	1.08	1.06	1.02	0.96
Grain sorghum	0.30	0.38	0.60	0.83	1.01	1.07	0.99	0.88	0.76	0.65	0.56
Winter wheat[a]	1.46	1.44	1.42	1.39	1.35	1.30	1.23	1.15	1.03	0.86	0.78
Spring grains	0.29	0.45	0.67	0.89	1.09	1.28	1.31	1.17	0.90	0.55	0.20
Cotton	0.20	0.25	0.33	0.50	0.79	0.97	1.02	0.95	0.81	0.65	0.29
Dry beans	0.50	0.59	0.71	0.87	1.02	1.10	1.12	1.06	0.94	0.81	0.67
Sugar beets	0.45	0.50	0.61	0.79	0.95	1.10	1.20	1.25	1.21	1.13	1.04
Potatoes	0.33	0.40	0.51	0.72	0.98	1.17	1.31	1.37	1.36	1.31	1.23
Tomatoes	0.45	0.45	0.47	0.56	0.75	0.95	1.03	0.99	0.90	0.80	0.70
Melons and cantaloupe	0.44	0.48	0.56	0.65	0.76	0.81	0.81	0.78	0.75	0.71	0.67
Small vegetables	0.29	0.40	0.57	0.69	0.77	0.81	0.82	0.79	0.72	0.58	0.38

[a]Data given only for springtime season of 70 days prior to harvest (after last frost). K_c increases from 0.50 at seeding to 1.46 during period with average temperature below 32 °F.

Source: Davis and Sorensen (1969).

Solution

TABLE 2.7 CONSUMPTIVE USE COMPUTATION

(1) Period	(2) Mean Monthly Temp. (°F)	(3) Number of Days	(4) Midperiod Percent of Total Season[a]	(5) K_c from Table 2.6(b)	(6) K_t[b]	(7) Percent p from Table 2.5	(8) U[c] (in.)
Apr. 10–30	44.5	20	5	0.48	0.46	8.98	0.88
May	55.2	31	19	0.60	0.64	10.12	2.15
June	60.8	30	35	0.87	0.73	10.20	3.94
July	69.5	31	51	1.10	0.89	10.32	7.02
Aug.	68.6	31	68	1.24	0.87	9.61	7.11
Sept.	57.9	30	84	1.18	0.69	8.40	3.96
Oct. 1–15	47.9	15	96	1.10	0.52	7.71	2.11
Total		188					

[a] $\dfrac{\text{Number of days up to middle of the period}}{\text{Total days in growing season}}$.

[b] $K_t = 0.0173(\text{col. 2}) - 0.314$.

[c] $U = \dfrac{(\text{col. 2})(\text{col. 5})(\text{col. 6})(\text{col. 7})}{100}$.

Seasonal consumptive use:

$$
\begin{aligned}
U &= \frac{20}{30}(0.88) + 2.15 + 3.94 + 7.02 + 7.11 + 3.96 + \frac{15}{31}(2.11) \\
&= 25.79 \text{ in.}
\end{aligned}
$$

2.9.3 Effective Rainfall

The portion of the rainfall during the growing season that is utilized in meeting the requirements of crops is termed the "effective rainfall." The remainder is lost through surface runoff and deep percolation. In humid areas, this may provide a major portion of the requirements, whereas in arid areas it may constitute only a small part. The necessity of irrigation in humid regions may arise due to unbalanced distribution of the rainfall.

Effective rainfall is influenced by many factors relating to the (1) soil moisture, (2) cropping pattern, (3) application of irrigation, and (4) rainfall characteristics. Based on the study of extensive data, the Soil Conservation Service (1964) suggested the relationship shown in Table 2.8. The limitation on use is given at the bottom of the table.

Whereas the crop consumptive use requirements vary from year to year by a small margin, the variations in rainfall are large. As such, the frequency analysis of effective rainfall is made as follows:

1. For the region under consideration, available data on monthly rainfall are collected.
2. Using Table 2.8, the effective rainfall figures for each month of record are determined.

TABLE 2.8 AVERAGE MONTHLY EFFECTIVE RAINFALL RELATED TO MEAN MONTHLY RAINFALL AND AVERAGE MONTHLY CONSUMPTIVE USE

Monthly mean rainfall (in.)	Average monthly consumptive use, U (in.)									
	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0
	Average monthly effective rainfall (in.)[a]									
0.5	0.20	0.25	0.30	0.30	0.30	0.35	0.40	0.45	0.50	0.50
1.0	0.55	0.60	0.65	0.70	0.70	0.75	0.80	0.85	0.95	1.00
2.0	1.00	1.25	1.35	1.55	1.55	1.55	1.60	1.70	1.85	2.00
3.0	1.00	1.85	1.95	2.10	2.20	2.30	2.40	2.55	2.70	2.90
4.0	1.00	2.00	2.55	2.70	2.90	2.95	3.15	3.30	3.50	3.80
5.0	1.00	2.00	3.00	3.25	3.50	3.60	3.85	4.05	4.30	4.60
6.0	1.00	2.00	3.00	3.80	4.10	4.25	4.50	4.80	5.10	5.40
7.0	1.00	2.00	3.00	4.00	4.60	4.80	5.05	5.40	5.70	6.05
8.0	1.00	2.00	3.00	4.00	5.00	5.30	5.60	5.90	6.20	
9.0	1.00	2.00	3.00	4.00	5.00	5.75	6.05	6.35		

[a]Based on 3-in. net depth of application. For other net depths of application, multiply by the factors shown below.

Net depth of application	0.75	1.0	1.5	2.0	2.5	3.0	4.0	5.0	6.0	7.0
Factor	0.72	0.77	0.86	0.93	0.97	1.00	1.02	1.04	1.06	1.07

Note: Average monthly effective rainfall cannot exceed average monthly rainfall or average monthly consumptive use. When the application of the factors above results in a value of effective rainfall exceeding either, this value must be reduced to a value equal to the lesser of the two.

Source: U.S. SCS (1964).

3. For each year on record, the total effective precipitation for all the months of the growing season is determined.
4. From the resultant figures—one for each year on record—a frequency curve is prepared by the method of Section 8.6.
5. If an irrigation supply is desired which will be adequate 90% of the years (9 of 10 years), the effective rainfall corresponding to the 90% value of the frequency curve is observed.
6. The total effective rainfall is distributed over the months of the growing season in the ratio indicated by the 10 driest years on record.

2.9.4 Farm Losses

The losses that take place from the water delivered to the farm are measured by the on-farm efficiency. Thus

$$\text{on-farm efficiency} = \frac{\text{water utilized in crop growth}}{\text{water delivered to farm}}$$

The principal factors that affect efficiency include (1) the method of applying the water, (2) the texture and condition of the soil, (3) the slope of the land, (4) the preparation of the land (i.e., ditched or bordered), (4) the rate of irrigation flow in relation to the farm size, and (5) the management by the irrigator. The farm losses take place due to (1) the deep percolation beyond the root zone of crops, (2) the surface wastes from the fields, and (3) on-farm distribution losses and nonproductive consumption.

For deep percolation, a minimum allowance of about 20% of the applied water is made. This ensures adequate leaching if the applied water does not contain more than 1300 ppm of dissolved salts and the drained water can be accepted with 6000 ppm of dissolved salts. It may be necessary to pass additional water for leaching if the irrigation water applied is more saline or if the drained water has to have a lower salinity or if the soil requires reclamation. Refer to Hansen et al. (1980) for the leaching requirements.

Surface waste or runoff is inherent in most irrigation systems. This quantity is, however, recovered and becomes available for use within a project or elsewhere in the basin downstream. The surface wastes usually amounts to 6 to 10% of the quantity delivered to the fields. Distribution losses vary from a very small quantity in pipelines to about 15% in unlined ditches.

On-farm efficiency, which is a product of the efficiency of each of the items above, can be achieved in the range of 40 to 70% for a properly designed efficient irrigation system. The higher value in the range above is associated with the sprinkler system.

2.9.5 Conveyance Losses and Waste

Conveyance losses and waste relate to the water lost between the point of diversion from a stream or reservoir to the points of delivery to farms and is measured by the off-farm efficiency. The losses comprise the evaporation through the canal system, the water seeped through the canal system, and the operational wastes discharged into drains or streams.

Evaporation losses from the canal water surface are not too large unless the canal is very shallow and wide. Usually, these are less than 1% of the canal flow.

Seepage losses from canals depend on (1) the permeability of the soil, (2) the wetted surface of the canal, and (3) the difference in level of water in the canal and the adjacent groundwater table. The average unit seepage rates for the western United States are similar to those cited by Hart (Worstell, 1975); see Table 2.9.

TABLE 2.9 AVERAGE SEEPAGE LOSS FROM CANALS IN SOUTHERN IDAHO

Type of Soil	Seepage Loss (ft/day)
1. Medium clay and loam	0.5–1.5
2. Impervious clay	0.5
3. Medium soils	1.0
4. Somewhat pervious soils	1.5–2.0
5. Gravel	2.5–5.0

Source: Hart (1963).

From the lined canals, the loss rates are between about 0.1 and 1.0 ft/day (Worstell, 1975). To determine the total quantity of seepage from a canal system, the

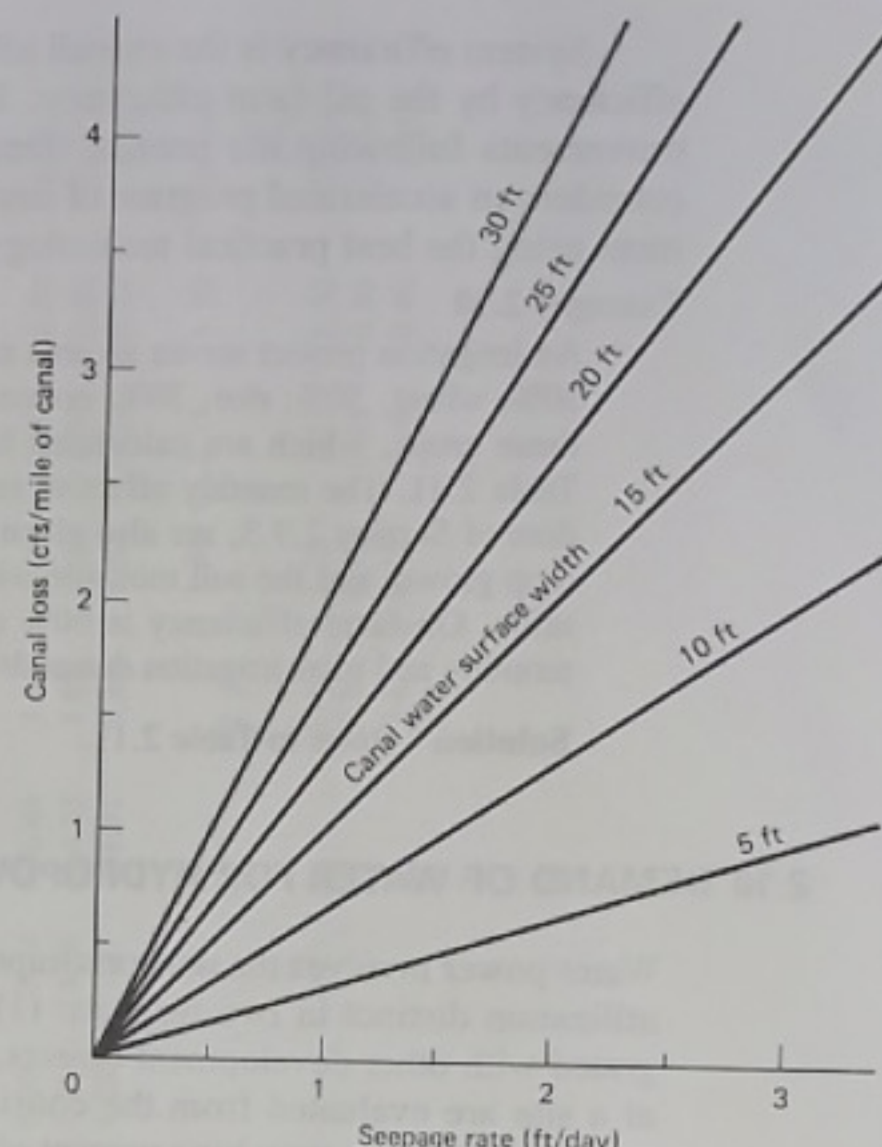

Figure 2.7 Chart to estimate the seepage losses from a canal (from Worstell, 1975).

data required are (1) the predominant soil texture to ascertain the average seepage rates, (2) the widths, and (3) the lengths of the canals. The chart developed by Worstell (1975), illustrated in Figure 2.7, can be used to estimate the seepage loss in cfs per mile for different canal widths. Broadly, the seepage losses range from 15 to 45% of diverted flow on unlined canals and from 5 to 15% on lined canals.

Operational wastes are unavoidable. These result from the inability to release into the canal system the quantity to match exactly all the requirements, operation of the canals at high levels to reduce siltation, unexpected rainfalls, and breaches in the system. These losses range from 5 to 30% of diversions on projects with ample supplies and from 1 to 10% with limited supplies.

Off-farm efficiency, comprising the foregoing items of conveyance losses, ranges between 50 and 90%. In cases where water originates on the farm itself, such as from a well, the off-farm efficiency is 100%. The average irrigation efficiency for the entire United States is indicated in Table 2.10.

TABLE 2.10 AVERAGE EFFICIENCY IN THE UNITED STATES

	Trend Efficiency (%)			High Efficiency (%)		
Year	On-farm	Off-farm	System	On-farm	Off-farm	System
1975	53	78	41			
1985	56	80	45	59	82	48
2000	59	83	49	66	88	58

Source: U.S. SCS (1976).

System efficiency is the overall efficiency obtained by multiplying the on-farm efficiency by the off-farm efficiency. Trend efficiency reflects irrigation/water improvements following the present trend in upgrading of systems. High efficiency considers an accelerated program of improving irrigation systems and water management using the best practical technology available.

Example 2.10

An irrigation project serves an area of 50,000 acres. The cropping pattern* is: alfalfa, 30%; wheat, 50%; rice, 30%; cotton, 20%. The monthly consumptive use values for these crops, which are calculated by the procedure of Section 2.9.2, are given in Table 2.11. The monthly effective rainfall values, which are calculated by the procedure of Section 2.9.3, are also given in the table. The irrigation water applied prior to crop growth and the soil moisture withdrawal in certain cases are also indicated in the table. On-farm efficiency is 60% and off-farm efficiency is 90%. Determine the monthly and total irrigation demands.

Solution Refer to Table 2.11.

2.10 DEMAND OF WATER FOR HYDROPOWER

Water power involves the nonconsumptive use of water. This feature makes the water utilization distinct in two respects: (1) hydropower generation can readily be integrated with other development objects, and (2) all resources (streamflows) available at a site are evaluated from the consideration of power-producing potential. With proper planning, a very high percent of the total available streamflow in a river basin may be used for hydropower through a series of power plants. The problem pertains to locating the potential hydropower sites that are within a reasonable transmission distance of the power market under consideration. Since hydro energy is the product of the available head and the available flow (times a certain constant), the sites having a good combination of head and flow are investigated.

From a consideration of the head, rapids, falls, and dam locations offer good hydropower potential. Whereas the head at a site is practically constant, the available flows are highly variable. The study of maximum flows is important from the viewpoint of the design or installed capacity of the power plant; the average flows are important from the consideration of the energy output and minimum flows are required to predict the dependable plant capacity. Since the entire quantity available at a site (except the flood flows) is utilized in power production, the study of water demands for hydropower amounts to the collection of streamflow data and their analysis. Usually, the analysis relates to the preparation of the flow–duration curve discussed in Section 7.28.2, which indicates the magnitude of discharge against the percentage of time that discharge is exceeded at a site.

There are two types of hydropower plants: (1) a run-of-river plant uses direct streamflows, and its energy output is subjected to the instantaneous flow of the river; and (2) a storage plant with a reservoir is able to produce increased dependable en-

*The cropping pattern is defined as the percent of the total irrigable area devoted to each crop during each of the two principal growing seasons of a year. Each area used for crops in both seasons will be counted twice. The perennial crops using water in all 12 months will also be counted twice. Complete utilization of the land in both seasons will sum up to 200%.

TABLE 2.11 COMPUTATION OF MONTHLY IRRIGATION DEMANDS

Item	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total
R (in.)	0.8	1.0	1.9	2.0	1.4	1.9	3.9	2.3	1.8	1.0	0.8	0.7	19.5
1. Alfalfa (30% of irrigable area)													
U (in.)	1.40	1.71	2.0	2.33	4.14	6.06	7.94	6.65	3.77	3.00	2.10	1.50	42.6
PP (in.)(+)													0
SM (in.)(−)													0
IR, gross (in.)	1.40	1.71	2.0	2.33	4.14	6.06	7.94	6.65	3.77	3.00	2.10	1.50	42.6
IR, net (in.)	0.6	0.71	0.10	0.33	2.74	4.16	4.04	4.35	1.97	2.00	1.30	0.80	23.1
IR, eff. (in.)	0.18	0.21	0.03	0.10	0.82	1.25	1.21	1.31	0.59	0.60	0.39	0.24	6.93
2. Wheat (50%)													
U (in.)	2.11	3.43	6.29	5.90						1.30	1.75	1.50	22.28
PP (in.)(+)									2.10				2.10
SM (in.)(−)				2.20									2.20
IR, gross (in.)	2.11	3.43	6.29	3.70					2.10	1.30	1.75	1.50	22.18
IR, net (in.)	1.31	2.43	4.39	1.70					0.3	0.30	0.95	0.80	22.18
IR, eff. (in.)	0.66	1.21	2.20	0.85					0.15	0.15	0.48	0.40	6.10

TABLE 2.11 (contd.)

Item	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total
3. Rice (30%)													
U (in.)					6.80	10.42	12.53	9.50					39.25
PP (in.)(+)				9.00									9.00
SM (in.)(−)								2.50					2.50
IR, gross (in.)				9.00	6.80	10.42	12.53	7.00					45.75
IR, net (in.)				7.00	5.40	8.52	8.63	4.70					34.25
IR, eff. (in.)				2.10	1.62	2.56	2.58	1.41					10.27
4. Cotton (20%)													
U (in.)			1.34	2.82	2.96	5.36	6.86	4.14	1.14				24.62
PP (in.)		1.00											1.00
SM (in.)								1.5	0.5				2.0
IR, gross (in.)		1.00	1.34	2.82	2.96	5.36	6.86	2.64	0.64				23.62
IR, net (in.)		0	0	0.82	1.56	3.46	2.96	0.34	0				9.14
IR, eff. (in.)		0	0	0.16	0.31	0.69	0.59	0.07	0				1.82

TABLE 2.11 (contd.)

Item	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total
Total, IR (eff.) (in.)	0.84	1.42	2.23	3.21	2.75	4.50	4.38	2.79	0.74	0.75	0.87	0.64	25,12
IR for area (acre-ft × 10^3)	3.50	5.92	9.29	13.38	11.46	18.75	18.25	11.62	3.08	3.13	3.62	2.67	104.67
Farm delivery @ 60% efficiency (acre-ft × 10^3)	5.83	9.87	15.48	22.30	19.10	31.25	30.42	19.37	5.13	5.21	6.03	4.45	174.45
Gross requirements (acre-ft × 10^3)	6.48	10.97	17.20	24.78	21.22	34.72	33.80	21.52	5.70	5.79	6.70	4.94	193.83

Abbreviations:

R = effective rainfall
U = consumptive use
PP = irrigation applied prior to crop growth.
SM = soil moisture withdrawal
IR, gross = gross irrigation required = U + PP − SM
IR, net = net irrigation required = IR(gross) − R
IR, effective = IR(net) × percent irrigable area
IR for area = Farm area × Total IR (eff.)

$$\text{Farm delivery} = \frac{\text{IR for area}}{\text{farm efficiency}}$$

$$\text{Gross requirement} = \frac{\text{farm delivery}}{\text{off-farm conveyance efficiency}}$$

ergy on the basis of the controlled water release. If the reservoir serves only to smooth out the weekly fluctuations in streamflows, the plant is said to have a pondage capacity. On the other hand, a reservoir that serves to store water from the wet season to the dry season is said to have a storage capacity.

2.10.1 Power and Energy Production from Available Streamflows

Lowering a water quantity of Q ft^3/sec over h ft will release energy at a rate of (62.4 Qh) ft-lb/sec. Converting this to kW units and including the efficiency term, an equation for power (rate of energy) can be given by

$$P = \frac{Qhe}{11.8} \qquad [\mathrm{FLT}^{-1}] \tag{2.23}$$

where

P = plant capacity, kW
Q = discharge through the turbines, cfs
h = net head on the turbines, ft
e = combined efficiency for turbines and generators

Flow–duration curves developed from long-term monthly streamflow records offer a convenient tool in plant capacity design. The procedure for preparation of a duration curve is described in Section 7.28.2. A typical curve is shown in Figure 2.8.

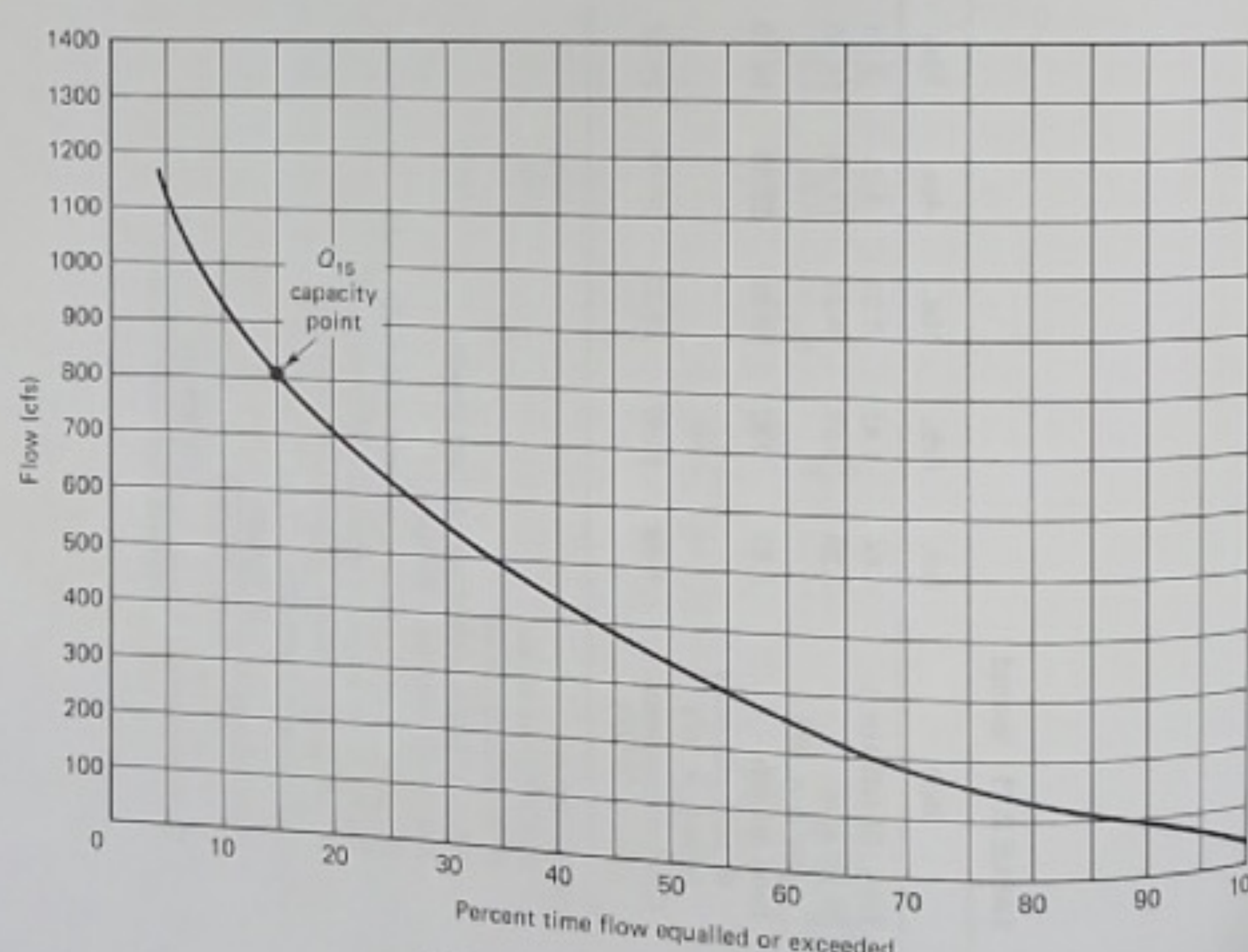

Figure 2.8 Flow–duration curve.

In eq. (2.23), with an average value of head, the efficiency and head are practically constant for a plant. Thus the power is directly proportional to the flow. In other words, the curve in Figure 2.8 indicates the power production with a suitable modification of the vertical scale. The design or installed capacity of a plant is based on the maximum flow, which is usually taken to be Q_{15} (i.e., flow exceeded 15% of the time). Floodflows above this magnitude are allowed to overflow without producing power. Thus the installed capacity is given by (taking $e = 0.84$)

$$P_{\text{instal}} = \frac{Q_{15}h}{14} \qquad [\text{FLT}^{-1}] \tag{2.24}$$

where

P_{instal} = installed capacity, kW

Q_{15} = discharge with 15% exceedence, cfs

h = net head, ft

If the time scale (abscissa) in Fig. 2.8 is expressed in terms of hours in a year, the area under the curve will provide the annual energy production. Mathematically,

$$E = \frac{Q_{\text{av}}h}{14}(8760) \qquad [\text{FL}] \tag{2.25}$$

where

E = annual energy, kWh.

Q_{av} = average discharge, cfs

8760 = number of hours in a year

Q_{av} is the average discharge under the curve in Figure 2.8 taking Q_{15} as the highest magnitude of discharge, similar to eq. (7.59).

A plant capacity factor is the ratio of the average power production to the installed capacity. This is practically equal to the ratio Q_{av}/Q_{15}, assuming that the head and the efficiency are essentially constant. By reservoir storage, both Q_{av} and Q_{min} are improved, and thus the annual energy production and the dependable (firm) power are enhanced. The plant capacity factor also increases, resulting in a more efficient use of a plant. A plant capacity factor of 0.6 is common for storage-type power plants.

Energy computations assume that an adequate number and adequate sizes of turbine units have been installed to utilize the minimum available flow. If only one turbine unit is provided, its operative range is generally from 30 to 110% of the turbine design flow, which means that the turbine will be inoperative during the times the flow is less than 30% of the design value. Thus the energy production will be for a shorter period in a year and the total annual generation will, accordingly, be less.

Similarly, depending on the turbine type, there is an operating limitation on the head. Usually, a turbine can operate in the range 60 to 120% of the design head. It is considered that the available head is fairly constant or that an average value of head is used in energy computations by eq. (2.25) when there are small fluctuations, which is the case with run-of-river projects and projects with remote location of

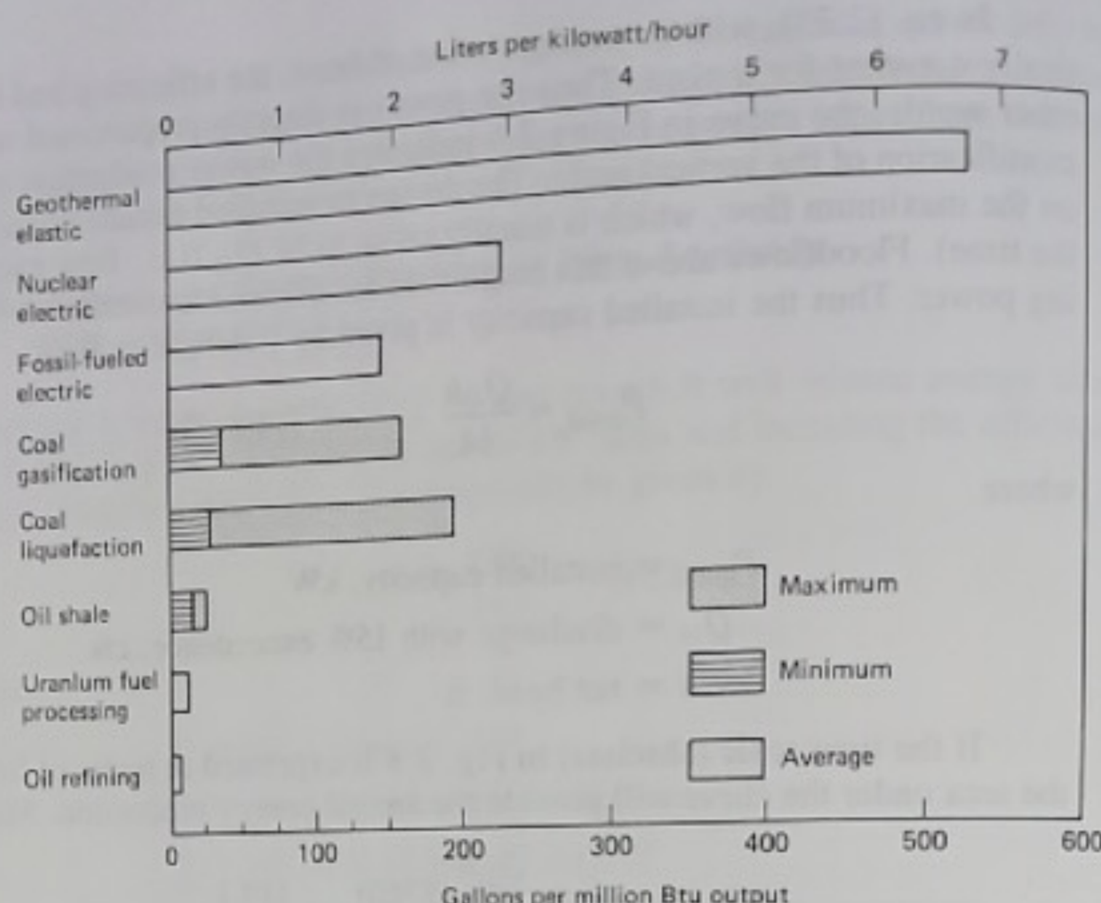

Figure 2.9 Water consumption in refining and conversion processes [from United States Geological Survey (USGS) Circular No. 703, 1974].

power plants. If variations in head are substantial, a sequential analysis is made wherein the energy calculations are made in steps at different intervals.

2.10.2 Demands for Other Energy Sources

In addition to the requirements in thermal power stations for cooling purposes as discussed in the context of industrial requirements and for hydropower generation described in the previous section, water is needed in other sources of energy production as well. It is needed for the processes related to the extraction of energy sources, such as the mining and refining of coal, uranium, and oil. It is also needed for the energy conversion processes of heat energy to mechanical energy to electrical energy. Water demands for extraction and conversion processes of various fuel energy sources are indicated in Figure 2.9. Per unit energy production, the consumption of water for cooling purpose in thermal power plants is 40 to 150 times greater than for the other sources of energy listed in Figure 2.9.

Example 2.11

At the Rimmon Pond site on the Naugatuck River near Seymourtown, Connecticut, in the Housatonic basin (drainage area 300 mi²), the flow–duration data from the monthly flow records are as given in Figure 2.8. The average head is 30 ft. Assess the site for its hydropower potential.

1. From Figure 2.8, $Q_{15} = 810$ cfs.
2. From eq. (2.24),

$$P_{\text{install}} = \frac{810(30)}{14} = 1736 \text{ kW}$$

3. $$\begin{aligned} Q_{av} &= 0.175Q_{15} + 0.075Q_{20} + 0.10(Q_{30} + Q_{40} + Q_{50} \\ &\quad + Q_{60} + Q_{70} + Q_{80} + Q_{90}) + 0.05\bar{Q}_{100} \\ &= 0.175(810) + 0.075(705) + 0.10(550 + 430 + 340 + 260 + 180 \\ &\quad + 130 + 90) + 0.05(40) \\ &= 395 \text{ cfs.} \end{aligned}$$

4. From eq. (2.25),

$$E = \frac{395(30)(8760)}{14} = 7.41 \times 10^6 \text{ kWh}$$

5. Plant capacity factor $= \dfrac{Q_{av}}{Q_{15}} = \dfrac{395}{810} = 0.49.$

2.11 DEMAND FOR NAVIGATION

There are three different methods to provide navigable waterways: (1) river regulation, (2) lock-and-dam, and (3) artificial canalization. In the first method, a river channel is improved by means of river training works and dredging. In some sections of the river channel, the natural depth of water is often not sufficient to maintain navigability, which requires release of water from upstream reservoirs. This demand from the reservoirs is likely to be on the order of several thousand cubic feet per second successively for several months. Thus huge reservoir capacities of several million acre-feet are needed for navigation purpose. One of the shortcomings of this method is that the water deficiencies are usually in the lower reaches of a river, while the reservoir sites are in its upper part. This results in many technical, operational, and legal difficulties in maintaining the navigable flow in downstream reaches.

In the second method, the depth of water for low streamflow is increased behind a series of dams through a succession of backwater curves. At each dam, a shiplock is provided to negotiate the difference in water levels upstream and downstream of the dam. The water demand relates to the (1) evaporation losses from the reservoir pools, (2) water requirements for locking operations, and (3) leakages at shiplocks.

Each locking operation requires the release of water in the downstream direction equivalent to the volume of the lock between the upstream and downstream levels. This might involve a flow of over 1000 acre-ft/day (500 cfs). The water for lockages is not accumulative since the water displaced by one lock can subsequently be used by the next lock downstream. Compared to the locking requirements, the evaporation and leakages are insignificant.

The third method provides for an artificially constructed new channel with a number of shiplocks. This method is adopted either to connect two different river systems or in situations where the other two methods are not suitable. As regards the water demand, a flow of several hundred cubic feet per second has to be maintained through the channel. This is supplied from a stream with a natural dependable flow, or by a reservoir. In addition, the requirements of evaporation, lockage operation, and leakage as discussed for the second method, have to be provided for. If an unlined channel is constructed, the seepage losses have to be included also.

PROBLEMS

2.1. A community had a population of 12,000 in 1960, which is increased to 20,000 in 1985. The saturation population is 80,000. Estimate the 1995 population by **(a)** arithmetic growth, **(b)** constant percent increase, and **(c)** decreasing rate of increase.

2.2. Using the following census figures, estimate the population for 1990 by **(a)** the graphical method, and **(b)** the most appropriate mathematical method.

Year	Population (thousands)
1950	35.8
1960	38.2
1970	40.7
1980	43.3

2.3. From the following census data, estimate the 1965 and 1988 population by **(a)** the graphical method, and **(b)** the most appropriate mathematical method.

Year	Population
1950	25,000
1960	28,190
1970	31,780
1980	35,830

2.4. A water supply reservoir has a capacity of 25 acre-ft. It is serving a city having a present population of 60,000, which is expected to increase to 100,000 in 20 years. For how many years will the reservoir be adequate to supply the city? Assume arithmetic growth of the population and an average daily draft from the reservoir of 160 gallons per person. Neglect the losses.

2.5. A community has an estimated population 20 years hence of 15,000. The water treatment plant of the community has a capacity of 7.0 mgd, which is adequate for the next 35 years, with an input rate to the plant of 175 gallons per person per day. If the community is growing at a geometric rate, what is the present population?

2.6. A community has a population of 28,000. It is estimated that 20 years hence its population will be 38,000, and the saturation population is expected to be 80,000. The total water consumption at present has been estimated to be 4.0 million gallons per day. The existing treatment plant has a capacity of 9.2 million gallons per day. Determine in how many years the consumption will reach its design capacity if the community has a declining growth rate.

2.7. The continental United States registered the following populations. Determine **(a)** the saturation population, **(b)** the equation of the logistic curve, and **(c)** the projected population in the year 2000.

Year	Population (millions)
1820	9.6
1900	76.0
1980	225.1

2.8. A city has the following census data. Fit a logistic curve to the data and determine (a) the saturation population, and (b) the population in the year 2000.

Year	Population (thousands)	Year	Population (thousands)
1860	15.0	1930	67.8
1870	20.0	1940	78.0
1880	25.0	1950	83.4
1890	32.0	1960	91.8
1900	40.0	1970	96.6
1910	47.5	1980	103.2
1920	58.8		

2.9. A community located within the city of Problem 2.8 has the following census data. Estimate the population for 1980 by (a) the constant ratio method, and (b) the changing ratio method using the graphical extension.

Year	Population (thousands)	Year	Population (thousands)
1900	6.5	1940	16.0
1910	8.4	1950	17.4
1920	11.2	1960	19.5
1930	13.5		

2.10. Estimate the 1980 and 2000 population of the community in Problem 2.9 by the simple regression analysis.

2.11. Average daily usage of water in a city is 175 gallons per capita per day (gpcd), which excludes the fire demands. Determine (a) the maximum monthly usage in gpcd, (b) the maximum weekly usage in gpcd, and (c) the maximum daily usage in gpcd.

2.12. For Problem 2.11, determine the maximum hourly requirement for water.

2.13. The future population of a community has been estimated to be 100,000. Determine the rate of fire demand and its duration.

2.14. The fire demand of a community is dictated by a six-story building of ordinary construction having a floor area of 20,000 ft^2. Determine the daily requirement for fire-fighting purpose.

2.15. Determine the fire flow for a four-story wood frame building having a floor area of 1000 m^2 which is connected with a six-story building of noncombustible construction that has a floor area of 990 m^2.

2.16. If the population of the community in Problem 2.14 is 50,000 and the average daily usage of municipal supply is 175 gpcd, determine the design flow for the following:

(a) Groundwater source development
(b) Conduit to the treatment plant
(c) Water treatment plant
(d) Pumping plant
(e) Distribution system
(f) Service reservoir if the working storage is 2.5 mgd

2.17. In 1986 the water requirements of a city of 500,000 population were as follows:

Municipal:	87 mgd
Industries:	
Manufacturing	115 mgd
Thermal power	210 mgd
Waste dilution:	6.59 bgd

In the year 2000 it is expected that the population will increase by 10%, industries by 15%, and the thermal power by 100 MW. Determine the requirements by each sector assuming the same level of waste treatment as at present. Assume a plant capacity factor of 0.6.

2.18. At Boise, Idaho, latitude 43°54′N, the long-term mean monthly temperatures are as follows:

Month	Temp. (°F)	Month	Temp. (°F)
Jan.	27.9	July	72.5
Feb.	33.6	Aug.	71.0
Mar.	41.4	Sept.	61.2
Apr.	49.1	Oct.	50.1
May	56.1	Nov.	39.7
June	64.5	Dec.	30.4

Compute the seasonal consumptive use of water for an alfalfa crop having a growing season of April 1 to September 15.

2.19. For the Boise, Idaho, climate in Problem 2.18, compute the seasonal consumptive water use for potatoes. The growing season is May 10 to September 15.

2.20. For the Boise, Idaho, climate in Problem 2.18, compute the seasonal consumptive water use for grain sorghum. The growing season is June 5 to November 2.

2.21. An irrigation project serves an area of 100,000 acres. The cropping pattern is: wheat, 40%; potatoes, 30%; grain sorghum, 35%; and citrus, 25%. The monthly consumptive use and the effective irrigation for these crops are given below. The irrigation water applied prior to crop growth and the soil moisture withdrawals for certain months are also indicated. The on-farm irrigation efficiency is 65% and the off-farm conveyance efficiency is 85%. Determine the monthly diversions and total demand for irrigation.

Item	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
R (in.)	0.8	0.9	1.5	1.9	1.7	2.5	4.5	3.2	2.1	1.4	0.6	0.5
Wheat												
U (in.)	2.1	3.2	5.95	5.70						1.5	1.75	1.40
PP (in.)				1.5					2.2			
SM (in.)												
Potatoes												
U (in.)					1.52	3.65	8.58	8.53	4.95			
PP (in.)				3.00								
SM (in.)								0.50	2.0			
(continued)												

Item	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
Grain sorghum												
U (in.)						2.04	5.36	6.59	3.43	1.39	0.49	
PP (in.)					1.0							
SM (in.)												
Citrus												
U (in.)	1.35	1.75	2.75	4.1	4.4	5.0	6.8	7.5	5.9	4.9	2.5	1.6
PP (in.)												
SM (in.)												

Abbreviations:

R = effective rainfall
U = consumptive use
PP = irrigation applied prior to crop growth
SM = soil moisture withdrawal

2.22. Flow–duration data for the Housatonic River near New Milford Town, Connecticut, are indicated below. The average head at the site is 25 ft. Assess the site with respect to **(a)** potential capacity, **(b)** annual energy generation, and **(c)** plant capacity factor.

Flow (cfs)	450	930	1180	1370	1680	1950	2180	2360
Percent of time flow exceeded	100	90	80	70	50	30	20	15

2.23. At the Harrisville, New York, site on the West Oswegatchie River, the flow–duration data are as given below. The average head is 35 ft. Assess the site for **(a)** potential capacity, **(b)** annual energy generation, and **(c)** plant capacity factor.

Flow (cfs)	1000	600	400	300	200	120	40	20
Percent of time flow exceeded	9	12	15	19	28	47	85	100

2.24. In Problem 2.23, if the plant capacity factor is increased to 0.65 by the storage capacity, determine the percent increase in the annual energy generation.