Chapter 4
Water Quantity Requirements

4-1. Introduction

The United States is blessed with an abundant supply of both ground and surface waters. Unfortunately, the population is not distributed in the same pattern as are the water resources, and the hydrologic cycle does not operate at steady-state. As a result, local water shortages have already occurred in most parts of the country and may be expected to increase in frequency and occur in other areas in the coming years. Thus, increasing importance is being attached to preparation of water use projections and the planning necessary to ensure that water demands are met in a manner that is both timely and cost-effective. In this chapter these problems are addressed specifically with respect to water supply systems serving municipal and rural communities, military installations, recreation areas, and highway rest areas. In addition, consideration is given to water conservation and its effect on water supply system design. Although the principal thrust of this manual is toward small systems, some discussion of municipal water supply system design is necessary to present the pertinent design information in logical fashion.

4-2. General Considerations

a. Water use rate variation. Water supply system design is complicated to a considerable extent by the fact that water use rates are influenced by a number of factors. For example, municipal use might be affected by some, or perhaps all, of the following:

(1) Climate.

(2) Standard of living.

(3) Extent of sewerage.

(4) Extent of metering.

(5) Price of water.

(6) Season of the year.

(7) Day of the week.

(8) Time of day.

(9) Special events.

(10) Firefighting requirements.

(11) Commercial development.

(12) Industrial development.

(13) Landscape irrigation.

(14) Water quality.

(15) Availability of alternate supplies.

(16) Distribution system pressure.

(17) System maintenance and management.

(18) Real or potential water shortages.

(19) Legal constraints.

The list is not intended to be all-inclusive, nor are all the factors presented independent. However, it is sufficient to make the point that for any given water supply system, many variables can affect water use. Thus, no single water use rate can be used to design every system or even every component of a given system. Specific water use rates that may generally be considered to be important include the following:

(1) Average annual use.

(2) Average monthly use.

(3) Maximum monthly use.

(4) Average weekly use.

(5) Maximum weekly use.

(6) Average daily use.

(7) Maximum daily use.

(8) Maximum hourly use.

(9) Maximum instantaneous use.

For specialized systems, for example those serving recreation areas or highway rest areas, other use rates may also be important. Examples include average weekend use and maximum weekend use. The magnitudes of use variations that may be expected for various types of water supply systems are considered in subsequent sections of this chapter.

b. Average use. A measure of average water use, such as the average daily use, is needed to determine if the yield of a water source is sufficient to safely supply water over long periods of time and to determine the storage capacity needed to assure that an adequate supply is available during critical periods (e.g., droughts).

c. Peak use. A measure of peak use, such as the maximum hourly use, maximum instantaneous use, or fire flow is needed to size distribution facilities (e.g., pipelines, booster pumps, storage) so that peak demands can be satisfied without overtaxing production and treatment facilities or causing excessive pressure losses.

d. Intermediate use. A measure of use between the average and peak values is ordinarily used in the hydraulic design of treatment facilities. Many engineers design treatment processes to operate normally at the average daily flow rate, but be hydraulically capable of passing a greater flow, say the maximum daily flow. This occasional "overloading" or "overrating" of the plant, or portions thereof (e.g., rapid sand filters), may be acceptable even though effluent quality is reduced to some extent. Alternatively, the plant may be designed to operate without overloading at the maximum daily use rate. In this situation, the plant may normally operate at process rates lower than those used in design, or various treatment units may be taken off line and held in reserve until needed. The latter approach is frequently used, especially with rapid sand filters. Another possibility is that the treatment plant may be designed to meet average demands by operating for only a portion of the day. Higher rates of demand can then be met rather easily by extending the hours of operation. This approach is usually uneconomical for larger cities, but can be very attractive for small operations.

4-3. Storage Requirements

a. Introduction. Depending upon the particular situation, several different types of storage facilities may be needed to ensure that an adequate water supply is always available. Examples include raw water storage (e.g., surface water impoundment), finished water storage at the treatment plant (e.g., clear well and backwash tank), and distribution storage (e.g., ground, elevated or hydropneumatic tanks). Regardless of the type of facility, the basic method used to determine the required storage volume is essentially the same.

b. Raw water storage.

(1) General. Where a surface water supply is used, it may be possible to design a supply system to operate without any raw water storage facility dedicated specifically to water supply. Examples might be a small town drawing water from a large multipurpose impoundment, or even a large city taking water from one of the Great Lakes. However, in the general case, some provision must be made to catch water during periods of moderate to high streamflow and store it for later use. The size of the storage facility required is usually determined based upon consideration of hydrologic information such as minimum dry-weather streamflow, average streamflow and rainfall/runoff patterns, and some average measure of water use, for example, the average daily use. The mass diagram, or Rippl, method has traditionally been used to determine storage requirements. This technique is amenable to either a simple graphical or more complex analytical approach, and is widely known since it is covered in many standard water supply and applied hydrology textbooks (Clark, Viessman, and Hammer 1977; Fair, Geyer, and Okun 1966a; Linaweaver, Geyer, and Wolff 1966; Salvato 1982; Steel and McGhee 1979). Essentially the same method is used to size equalization basins used in wastewater treatment (Metcalf and Eddy 1991). The mass diagram technique is very flexible and may be used in either a deterministic or probabilistic format. For more information the reader is directed to the references noted above.

(2) Design criteria. In the eastern United States, raw water reservoirs are usually designed to refill every year. In more arid regions, streamflow is less dependable and water must be stored during wet years for use during extended dry periods. Typical American practice over the last 50 or 60 years has been to size raw water storage facilities to be adequate to compensate for any drought condition expected to occur more often than once in about 20 years, plus some additional reserve storage allocation (e.g., 25 percent). This rule of thumb, combined with the implementation of use reduction measures when reservoir storage is depleted to some critical level, ordinarily results in a reasonable trade-off between storage requirements and user inconvenience. However, in recent years many other methods have appeared in the water supply literature. Regardless of the method used, it is important to consider the effects of evaporation, seepage, and siltation any time a reservoir is to be designed.

(3) Groundwater. When groundwater serves as the source of supply, no provision for long-term raw water storage is usually made. Short-term storage is, however, often useful. A good example is a situation where groundwater is extracted by a number of relatively low-yield wells (i.e., low-yield water supply to total water demand), pumped to a central storage tank and then withdrawn for distribution. This technique is especially useful for equalizing pumping rates when water from some, or all, of the wells requires treatment prior to distribution. The mass diagram approach mentioned in *b*(l) above may be used to size the storage tank so long as the inflow and outflow rates are known.

c. Finished water storage. Distribution storage facilities are used to meet peak demands (including fire flows), allow continued service when the supply is interrupted, equalize system pressures, eliminate continuous pumping, and facilitate the use of economical pipe sizes. While it is possible to size tanks using the mass diagram approach, it is more common to rely on various rules of thumb. Salvato (1982) suggests that, depending upon system size and type, distribution storage volume may vary from about one-half the average daily use, to the maximum daily use, to a 2- or 3-day supply. Even when rule-of-thumb criteria are used to size distribution storage facilities, it may be useful to conduct a mass diagram type of analysis (*b*(l) above) to ensure that peak demands can be met. Storage requirements for filter backwash tanks, clear wells, and other reservoirs can also be determined from mass diagrams if so desired.

4-4. Municipal Water Use

a. Introduction. As previously mentioned (paragraph 4-2*a*), municipal water use varies widely from city to city and from time to time for a given city. American Water Works Association (AWWA) (1975, 1981) and U.S. Geological Survey (1975) present data that indicate clearly that U.S. water use patterns vary considerably with geographical location. This point is further emphasized by the per capita water use data contained in Metcalf and Eddy (1991), Murray and Reeves (1972), and van der Leeder (1975).

b. Design approach. Design values for water use rates are usually determined as follows:

- Select the design period.
- Forecast the population to be served by the end of the design period.
- Estimate the expected average water use rate at the end of the design period.
- Estimate design use rates by multiplying the average use rate by selected factors.
- Determine the required fire demand from insurance requirements.
- From the various use rates calculated above, select those applicable to various system components.

A brief discussion of each step is outlined below. The same basic format is followed in later sections where rural, recreation area, military installation, and highway rest area systems are specifically addressed.

(1) Design period. As a general rule, the design period for portions of the system that may be readily enlarged (e.g., well fields and treatment plants) is chosen as 10 to 25 years. Components that are difficult and costly to enlarge (e.g., large dams) may be designed for a longer period, say 25 to 50 years. Prevailing interest rates are an important factor, with higher rates generally favoring shorter periods. The source of funds is also important. When funding assistance is available (e.g., in the form of grants or subsidized loans) there is a tendency to overdesign. In effect, this represents extension of the design period. Water lines serving residential areas are usually sized for full development since residential requirements in developing areas tend to change rapidly and replacement of such lines is costly.

(2) Population forecasts. Population forecasts are usually based on some combination of official census data; special studies made by various private and public interests (e.g., market surveys); the attitudes of local people (especially business and political leaders) toward expansion; and input from state, regional, and local planning agencies. Most states have developed population forecasting formulas that are adjustable for various regions within the given state. Because population forecasting has long been of interest to sanitary engineers, the topic is adequately covered in most standard water supply and wastewater engineering texts (Clark, Viessman, and Hammer 1977; Technical Manual 5-813-3; Fair, Geyer, and Okun 1966a; Metcalf and Eddy 1991; Steel and McGhee 1979).

(3) Average per capita use. Average per capita water use is usually determined from past experience in the local area or similar areas, regulatory agency requirements, or the water supply literature. Many studies of municipal water use have been reported and an overall average of about 450 to 800 liters per capita per day (L/cd) (100 to 175 gallons per capita per day (gpcd)) seems to be applicable for the United States. Publications prepared by the AWWA, U.S. Geological Survey and others (Metcalf and Eddy (1991), Murray and Reeves (1972), and van der Leeder (1975)) indicate an estimated national average of 755 L/cd (166 gpcd) for 1975. However, the reported range of values (less than 227 L/cd (50 gpcd) to more than 2273 L/cd (500 gpcd)) is so wide that specific knowledge about the area to be served should take precedence over national, or even regional, averages. A substantial improvement in water use forecasting can be realized by disaggregating municipal water use as described below.

(4) Disaggregated use. Municipal water use can be disaggregated (if sufficient data are available) and allocated to various water use sectors. An example scheme is shown in Table 4-1. Many other arrangements could, of course, be used. Typical allocations expressed as percentages of the average daily use are shown in Table 4-2. Disaggregation generally improves forecasting accuracy since the effects of such factors as climate (i.e., need for irrigation), commercial activity, industrial development, and water conservation programs can be readily considered. Residential water use can be further

Table 4-1
Scheme for Disaggregating Municipal Water Use Using Municipal Water Use Sectors

- Residential
  - Single-family
    - Interior
    - Exterior
  - Multiple-family
    - Interior
    - Exterior
- Commercial
  - Interior
  - Exterior
- Industrial
  - Process
  - Cooling
  - Sanitary
- Public and Institutional
  - Interior
  - Exterior
  - Hydrant Flow
- Unaccounted-for
  - Metering Error
  - Loss

disaggregated as shown in Table 4-3 (interior use only) and Table 4-4. A frequency distribution graph (USEPA 1980) indicates the frequency with which various average daily residential water use rates may be expected to be exceeded. Limited data will often preclude the complete disaggregation of water use. However, if at all possible, disaggregation should proceed at least to the level of separating residential, commercial, and industrial use. In regions where lawn watering is practiced, every effort should be made to consider residential interior and exterior use separately. This latter category can account for as much as 80 percent of afternoon residential use during a summer drought and thus has a great effect on peak as well as average use.

(5) Other water use rates.

(a) Regardless of the method used to determine the average water use (i.e., per capita estimation or disaggregation by sector), it is common to apply multipliers (factors) to the value selected to estimate other use rates. Some of these multipliers are shown in Table 4-5. The range of values indicates that significant differences exist between systems. As a general rule, the ratio of peak to average use rate increases with

Table 4-2
Disaggregated Municipal Water Use as Percentage of Average Daily Use

	Use Sector					Average
Reference[1]	Residential	Commercial	Industrial	Public	Unaccounted-for	Daily Use (gpcd)[2]
Linaweaver, Geyer, and Wolff 1966	41	18	24		17	-
California Department of Water Resources 1976	68	10	18		4	-
Murray and Reeves 1972	38	32			30	166
AWWA 1975	42	18	22		18	179
Deb 1978	52	17	15	7	9	153
Deb 1978[3]	39	12	31	5	13	162
Deb 1978[4]	40	15	25	5	15	160
Frey, Gamble, and Sauerlender 1975	49	12	21		18	166
Fair, Geyer, and Okun 1966a	33	43		7	17	150
Steel and McGhee 1979[5]	44	15	24	9	8	177

[1] Entries in this column are included in Appendix A.
[2] Gallons per capita per day. To convert to liters per capita per day, multiply by 3.7854.
[3] Average of 27 Pennsylvania utilities.
[4] 1978 national average.
[5] Projected for 2000 AD.

Table 4-3
Disaggregated Interior Residential Water Use as Percentage of Average Daily Interior Residential Use

	Use Sector						
Reference[1]	Toilet Flushing	Bathing	Laundry	Dishwashing and Cooking	Drinking	Miscellaneous	Average Daily Use (gpcd)[2]
Linaweaver, Geyer, and Wolff 1966	30	35	20	15			-
California Department of Water Resources 1976	42	32	14	12			-
Deb 1978	40	30	15	6	5	4	60
Dufor and Becker 1962	41	37	7	11		4	-
Bailey and Wallman 1971	39	34	14	11		2	64
U.S. Environmental Protection Agency 1980	35	20	22	23			46
U.S. Environmental Protection Agency 1981	40	30		25		5	65

[1] Entries in this column are included in Appendix A.
[2] Gallons per capita per day. To convert to liters per capita per day, multiply by 3.7854.

Table 4-4
Disaggregated Residential Water Use as Percentage of Average Daily Residential Use

	Use Sector		
Reference[1]	Interior	Exterior	Average Daily Use (gpcd)[2]
Linaweaver, Geyer, and Wolff 1966	77	26	80
California Department of Water Resources 1976	56	44	-
Bailey et al. 1969	93	7	-
Dufor and Becker 1962	96	4	55
Deb 1978	94	6	64

[1] Entries in this column are included in Appendix A.
[2] Gallons per capita per day. To convert to liters per capita per day, multiply by 3.7854.

decreasing system size and increasing use of water for lawn watering.

(b) Residential water use and water use rates have been studied by a number of researchers. However, many water supply textbooks rely heavily on the results of a project undertaken for the Federal Housing Administration by Johns Hopkins University during the 1960's. Reporting on this project, Linaweaver, Geyer, and Wolff (1966) presented mathematical relationships that may be used to estimate average residential water use for metered and sewered areas for any period of interest. The basic expression in SI units is

$$\bar{Q} = \bar{Q}_d + 6010(a)(\bar{L}_s)(\bar{E}_{pot} - \bar{P}_{eff}) \text{ with } \bar{Q} > \bar{Q}_d \qquad (4\text{-}1)$$

where

$\bar{Q}$ = expected average demand for any period (liters per day)

$\bar{Q}_d$ = expected average residential use for periods of a day or longer (liters per day)

a = number of dwelling units considered

$\bar{L}_s$ = average irrigable area per dwelling unit (hectares)

$\bar{E}_{pot}$ = estimated average potential evapotranspiration for the period in question (millimeters of water per day)

$\bar{P}_{eff}$ = amount of natural precipitation effective in satisfying evapotranspiration and thus reducing the need for lawn watering (millimeters of water per day)

$\bar{Q}_d$ may be estimated as a function of the average market value of the dwelling units as follows:

Table 4-5
Relative Water Use Rates

Reference[1]	Use Rate: Average Monthly	Average Daily	Average Maximum Monthly	Average Maximum Weekly	Maximum Daily	Maximum Hourly
Salvato 1982	1	-	1.5	-	2.25	4.5
Salvato 1982	1		1.5		4.5	9
Salvato 1982	-	1	-	-	4	9.5
Salvato 1982	-	1	-	-	-	6
Steel and McGhee 1979	-	1	1.28	1.48	1.8	2.7
Alabama State Board of Health 1978	-	1	-	-	1.5	2.25
Fair, Geyer, and Okun 1966a	-	1	-	-	1.5	2.5
Clark, Viessman, and Hammer 1977	-	1	-	-	1.35	3.4
Clark, Viessman, and Hammer 1977	-	1	-	-	2.9	6.1
Clark, Viessman, and Hammer 1977	-	1	-	-	4.1	9.1
Clark, Viessman, and Hammer 1977	-	1	-	-	4.2	12.1
Metcalf and Eddy 1991	-	1	1.2	1.4	1.8	-

[1] Entries in this column are included in Appendix A.

$$\bar{Q}_d = 594 + 13.1\ (V) \tag{4-2}$$

where V is the average market value of the dwelling units ($1,000's) corresponding to the year 1963. Clark, Viessman, and Hammer (1977) suggest that this method is still valid if property values are deflated to 1963 conditions using local indices. During high demand periods precipitation becomes negligible and Equation 4-1 reduces to:

$$\bar{Q} = \bar{Q}_d + 6010\,(a)\,(\bar{L}_s)\,(\bar{E}_{pot}) \tag{4-3}$$

The estimated average potential evapotranspiration may be estimated from climatological data. However, in a study of some 41 residential areas scattered over the United States, Linaweaver, Geyer, and Wolff (1966) found an average value of 7.11 millimeters (mm) (0.28 inches (in.)) of water per day. They also developed a series of design curves that may be used to estimate maximum daily and peak hourly water use rates based on housing density (dwelling units per acre) and the number of dwelling units served. These curves are reproduced in some water supply texts (Clark, Viessman, and Hammer 1977).

(6) Fire flows. The volume of water used annually for fighting fires in a typical municipality is ordinarily very small compared to the total use. However, short-term fire demands can be very high and in many cases govern the design of distribution facilities. Fire flow requirements are usually based on the recommendations of insurance industry groups (Insurance Services Office), and for residential areas generally range from 30 to 500 liters per second (L/s) (500 to 8000 gallons per minute (gpm)) depending upon the population served. For the central business district of large cities, the fire flow requirement may be as much as 760 L/s (12 000 gpm) for a single fire plus an additional 500 L/s (8000 gpm) for a second fire. The duration for which these flows must be maintained varies from 4 to 10 hours depending upon the size of the community. If a given system is incapable of delivering the recommended fire flow, fire insurance rates are adjusted upwards. As a general rule, it is assumed that the system must be able to deliver the fire flow concurrently with the maximum daily demand at a pressure of not less than 138 kilopascals (kPa) (20 pounds (force) per square inch). Thus, it is not surprising that the fire condition often controls distribution system design.

c. Commercial and industrial use. Industrial and commercial water use should be estimated separately if possible and then added to other disaggregated uses to reach an estimate of total municipal use. Furthermore, if sufficient information is available, individual, industrial and commercial users should be considered separately. Unfortunately, it is very difficult to predict what the water use of a given industrial or commercial establishment will be without very specific data. However, such estimates are often based on average use rates since data on individual operations are ordinarily not available. Some general guidance is presented in several sources (McGauhey 1968; Metcalf and Eddy 1972; Planning and Management Consultants, Inc., 1980b; Salvato 1960, 1982). Kollar and MacAuley (1980) have presented a rather detailed analysis of industrial water requirements.

4-5. Rural Water Use

a. Introduction. Design criteria that are appropriate for larger municipal water systems are often quite inappropriate for smaller community water systems. Generally, the average per capita water use for small community water systems (especially rural systems) is less than the average per capita residential water use for large cities. This is not universally true, however, since cultural factors, property values, the extent of lawn watering, and many other variables may influence use for a given community. Occasionally small community water systems serve commercial and/or industrial users, which have a major impact on facility design.

b. Design approach.

(1) Special considerations.

(a) For municipal systems, the peak water use rate considered in the design of distribution facilities is usually either the maximum hourly demand or the combination of the fire flow and the maximum daily demand. While design is frequently controlled by the latter case, such is not always true. However, the diversity of customers served, the grid-system layout of distribution piping, and the use of a 150- or 200-mm- (6- or 8-in.) diameter minimum pipe size combine to make consideration of urban residential demands for periods of less than one hour generally unnecessary.

(b) The population served by rural water systems tends to be rather disperse (i.e., low areal population density) with two to five service connections per mile of pipe being fairly typical. Thus, rural systems must be designed from a somewhat different perspective than municipal systems. Typically, fire protection to the extent recommended by the insurance industry is uneconomical, piping systems must be of the branching rather than the grid type, and the minimum pipe size is quite small (say 50-mm (2-in.) diameter). Unfortunately regulatory agencies have not always recognized these differences.

(c) Because in larger high-density residential areas and municipalities the extra costs associated with providing fire flow capacity are spread over many customers, the price of water service is not affected to a significant degree. The economics of rural systems are, however, entirely different and generalized fire protection is usually completely infeasible. Of course it may be argued that any dependable public water supply offers some fire protection. And, in some special cases, it may be economical to provide standard fire flows to a small area located near the water source or a major distribution point (e.g., elevated tank).

(d) Widespread development of rural water systems has occurred only during the recent past. Perhaps the two most important factors leading to this growth have been governmental assistance programs, primarily those of the Farmers Home Administration (FmHA), and the development and acceptance of polyvinyl chloride (PVC) pipe. A third factor has been the willingness of some state regulatory agencies to relax their design criteria somewhat to accommodate rural needs.

(2) Average water use.

(a) A limited number of controlled studies of rural water use have appeared in the literature. In most cases, average rural residential water use has been found to be somewhat less than average urban residential use. At least two factors would seem to be important in interpreting this finding. One is that many rural families have historically been less dependent on high-water-use appliances than have urban families. This is partly the result of economic factors and partly the result of the fact that rural areas are generally unsewered. Cultural differences are also probably significant in this regard. A second, though related, factor is that the unit price of rural water is generally higher than that of urban water. Several authors have suggested that rural water demand is rather price elastic. For example, data from a study of some 150 rural water systems in Kentucky (Grunwald et al. 1975) was used to develop the following expression in English units:

$$Q = 7.57\ P^{-0.92} \tag{4-4}$$

where

Q = average monthly water use, per dwelling unit

P = unit price of water, dollars per 1,000 gallons

The magnitude of the exponent on P makes it obvious that price is indeed an important factor. Hughes and Israelsen (1976) compared this expression to data obtained for a number of small water systems, mainly in the western United States, and found a similar trend. However, their data indicated that the coefficient on P might be somewhat low. It is likely that this results from increased irrigation use for the western systems.

(b) When all factors are considered, it seems quite reasonable to assume that rural residential water use will eventually approach the urban value. An abundant supply of high-quality water for domestic use is a major determinant of the quality of life in rural areas. Agricultural and industrial demands for water must also be met. Therefore, for design purposes, average residential water use rates for small rural water systems may be taken as equal to average residential use rates for nearby urban areas. Such an approach will almost certainly be conservative, especially for unsewered areas or areas not previously served by a public water system. Goodwin and Doeksen (1984) reported that data collected for 660 observations in Oklahoma indicated the following equation (English units) provided the best statistical reliability and economic consistency:

$$
\begin{aligned}
Q_m = & -1505.73 + 954.86N + 33.85Y + 102.76E \\
& + 55.49C + 183.60H + 953.86G \\
& + 2221.92I
\end{aligned}
\tag{4-5}
$$

where

Q_m = average monthly water use per customer, gallons

N = number of persons in the household

Y = year the house was built

E = total years of education for household head

C = number of cattle watered

H = number of horses watered

G = garden, dummy variable where G = 1 if garden is watered and G = 0 if no.

I = income, dummy variable where I = 1 if income exceeds \$40,000 and I = 0 if \$40,000 or less

As a general rule a reasonable degree of conservation in this regard will not be excessively expensive since costs of the system components most directly affected (e.g., transmission piping, raw water storage, and treatment facilities) are less related to flow rates (especially for small flows) than are costs of other components such as distribution piping.

(c) Some care must be used in the selection of design flow rates for small rural water systems since it may not be desirable to operate treatment facilities on a 24-hour-per-day basis. This situation arises, in part, because many of the capital costs associated with small treatment facilities are relatively unrelated to facility capacity. For example, Hansen, Gumerman, and Culp (1979) have reported typical complete package surface water treatment plant costs. These costs are exclusive of raw water intake and pumping facilities, clear well storage, high service pumping, land, and site work, except for foundation preparation. The costs indicate that, for the lower flow rates, the differential price paid for extra plant capacity is relatively small. For example, capacity can be doubled from 15 to 30 L/min (4 to 8 gpm) (at a filtration rate of 80 L/min per square meter (2 gpm per ft^2)) for an additional investment of only about 14 percent. If a shift to high rate filtration is acceptable, capacity can be increased by a factor of five for essentially the same incremental cost.

(d) An additional factor to consider is that small system operating costs tend to be dominated by operator salaries. Therefore, it is often economical to produce all the treated water needed on a typical day during a relatively short period, say 4 to 8 hours. This approach results in savings in the salaries of operating personnel, provides ample time for routine maintenance, often does not increase debt service to a significant extent ((c) above), and allows the flexibility to produce extra water occasionally by simply extending operation for an hour or so. As the average demand increases, plant "capacity" can be increased without additional capital expenditure by gradually lengthening the normal operating day. Thus, especially when a surface water source is used, the combination of a larger than necessary treatment plant and reduced operating time can be very attractive. Groundwater supplies frequently do not need treatment other than disinfection and, therefore, generally require less operator control. Thus the foregoing argument may not be valid for systems relying on wells or springs.

(e) If the decision is made to produce water for only a fraction of a day, raw water storage, pumping, transmission, and treatment, and finished water transmission, storage, and pumping facilities must all be designed accordingly.

(3) Peak water use.

(a) Since fire flows are not usually considered in the design of rural water systems, some other measure of peak water use must be used. The similarity of the customers normally served (i.e., mostly residential) is such that many designers and regulatory agency personnel feel that the maximum instantaneous demand should be used to size distribution facilities.

(b) It is obvious that for a given portion of a typical rural water system, maximum (peak) instantaneous demand should be a function of the number of customers served. Furthermore, it would certainly seem reasonable to assume that as the number of customers served increases, the ratio of peak instantaneous demand to customers served should decrease. That is, as the number of customers served increases, it becomes increasingly unlikely that all customers will demand water at the maximum rate simultaneously. Thus, peak instantaneous residential water use may logically be estimated as the product of the number of water services, or connections, times some peak use rate per connection (which is a function of the number of connections).

(c) Representative relationships between peak instantaneous residential use per connection and the number of connections served are presented in Alabama State Board of Health (1978), Ginn, Corey, and Middlebrooks (1966), and Hughes and Israelson (1976). Obviously differences of opinion exist.

(d) Hughes and Israelson (1976) and Hughes and Caufield (1977) have reported FmHA claims that some 5,000 systems have been designed using minimal standards (e.g., 3.8 L/min (1 gpm)/connection for 100 or more connections) without apparent difficulty (i.e. without subsequent customer complaints). However, in these cases it seems highly likely that the flow to individual homes is occasionally limited by the hydraulic capacity of the distribution lines. On the other hand, the recommendations of some state regulatory agencies (e.g., Alabama State Board of Health 1978) seem overly conservative, especially when one considers the frictional losses associated with flow at, for example, 55 L/min (15 gpm) through a typical 20-mm- (3/4-in.-) diameter water service line and 15-mm- (½-in.-) diameter house piping system (about 12 m (40 ft) of water in a 30-m (100-ft) run of 20-mm- (3/4-in.-) diameter plastic service line alone).

(e) The conservative approach to design taken by many regulatory agencies stems directly from missions related primarily to protection of public health. Thus, design standards and criteria are adopted that ensure the integrity of a water supply system against hypothetical simultaneous events having a probability of occurrence very near zero. While this is in many ways an admirable attitude, it is so costly that the result may be that the rural population in a given area is forced to continue to rely on individual water supplies of questionable quality because a community supply system cannot be economically justified. Clearly a common-sense approach to balancing these and other potential risks is needed. This has been recognized by many states.

(f) Where agreeable to regulatory agency personnel, and in the absence of good local data, the design use (flow) rates shown in Figure 4-1 are suggested. According to Ginn, Corey, and Middlebrooks (1966), and Hughes and Canfield (1977) these use rates correspond to a return interval of approximately 27 years. That is, demands in excess of those shown may be expected to occur once in about 27 years. These criteria should be more than satisfactory from the viewpoint of rural customer satisfaction and provide ample protection of public health.

(4) Other water use rates. The lack of available data makes the estimation of rural water use rates at least as much an art as a science. Two use rates that have not been discussed, but that may have design significance, are the maximum daily and maximum monthly demands. After surveying the literature, Hughes and Israelson (1976) suggested that a maximum daily rate of about 2.3 L/min (0.6 gpm) per residential connection appeared reasonable. Their data also indicate that a ratio of maximum monthly use rate to average monthly use rate of around 1.5 should be sufficient for most design purposes. An alternative approach is to estimate peak daily and monthly use rates from the “peaking” factors discussed above in paragraph 4-4*b*(5). As previously discussed, the use of these factors is likely to overestimate rural demands, at least in the short run.

4-6. Recreation Area Water Use

a. Introduction. Water systems serving recreation areas are similar in some respects to rural community systems, but also differ in some respects. As a rule they are rather compact, have branching type distribution piping, and must respond to widely varying water use rates that may be affected by many variables including the following:

(1) Location.

(2) Type of facilities provided.

(3) Visitation rates.

(4) Visitation patterns.

(5) Season of the year.

(6) Day of the week.

(7) Special events.

(8) Irrigation requirements.

(9) Weather conditions.

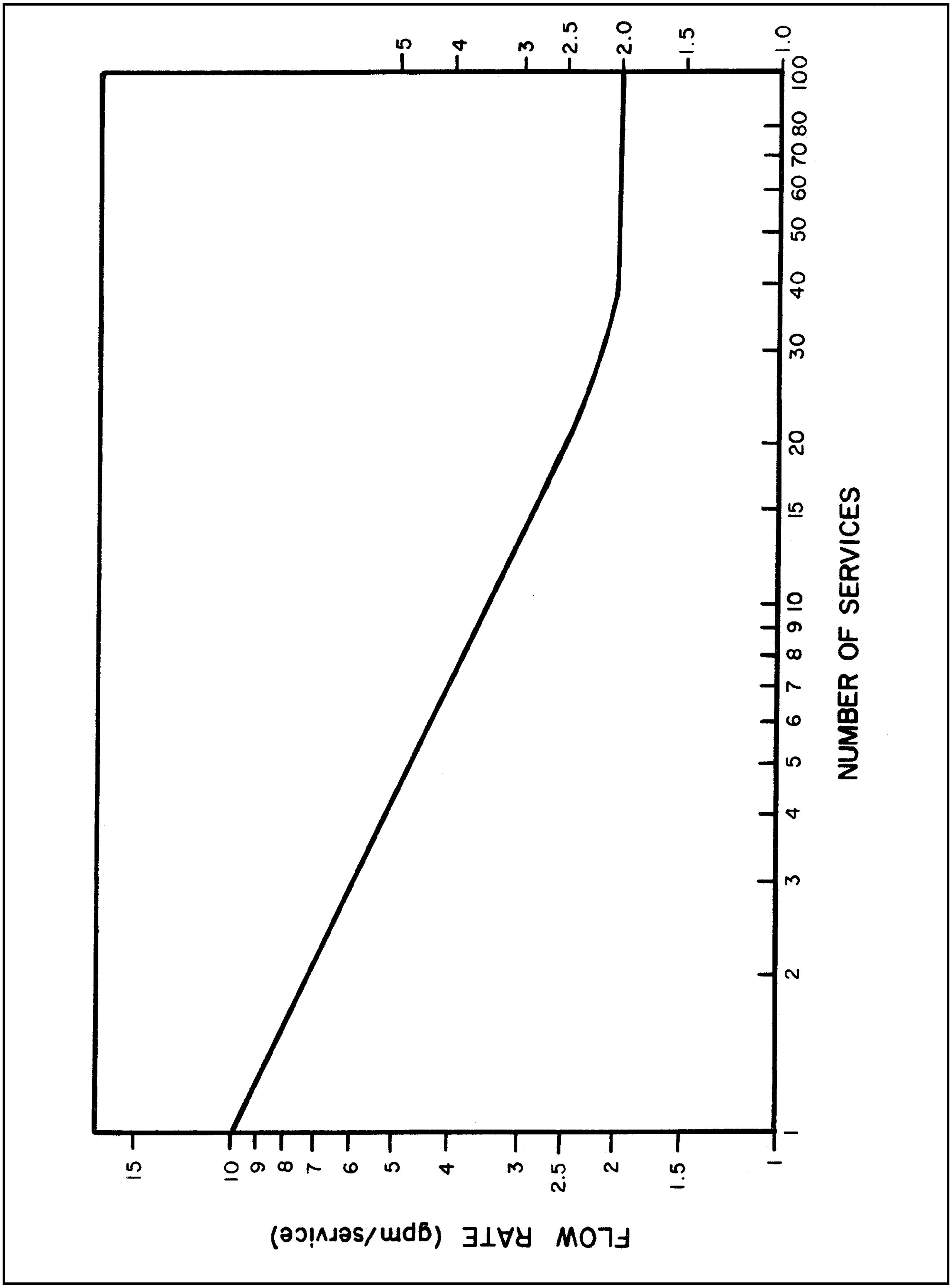

Figure 4-1. Instantaneous peak design flow rates (to convert gpm to liters/min, multiply by 3.7854)

b. Design approach.

(1) Corps guidance.

(a) It is suggested that average water use at Corps facilities be estimated as the sum of 110 to 190 L/day (30 to 50 gpd) for each day-shift employee (night-shift employees are generally neglected), 570 L/day (150 gpd) for each dwelling, 20 L/day (5 gpd) for each visitor expected to use flush-type toilets, plus any additional requirements (e.g., cooling water or lawn watering). It is further suggested that peak demands be based on a combination of 190 L/day (50 gpd) per dwelling and "reasonable" assumptions as to maximum frequency of use of facilities. These and other values can be developed from U.S. Environmental Protection Agency publication EPA 570/9-91-004, May 1991, entitled "Manual of Individual and Non-Public Water Supply Systems." Typically, most designers obtain their minimum demand standards from state or local standards. Many states probably use these standards. Many of these drinking water systems will require environmental permitting for construction; hence, the use of state-approved standards is recommended.

(b) USEPA (1991) recommends the use of the average water use rates summarized in Table 4-6.

(c) Francingues and Green (1976) have reported a detailed study of water use at a typical Corps recreation area near Memphis, TN. They found that campsite occupancy varied widely (zero to 98 percent of the design value) and that the maximum average observed weekend occupancy (78 percent) occurred, as expected, on a holiday weekend. A typical weekend visitation consisted of an average of 4 persons per occupied campsite (the range was 2 to 6) for a 2-day period (the range was 1 to 3). Some 63 percent of all recreational vehicles observed were equipped with wastewater holding tanks, and 79 percent of those vehicles made use of the trailer dumping station. A summary of observed water use for the period between 23 May and 1 September is presented in California Department of Water Resources (1976). The average water use of 458 L (121 gallons) per occupied campsite per day compares favorably with the 450-L (119-gallon) figure reported by Matherly et al. (as cited in Francingues et al. 1975) for the Sullivan Access Area at Lake Shelbyville, IL. The reported average per capita use of 87 L/day (23 gpd) is somewhat lower than the 114 L/day (30 gpd) suggested in EM 1110-2-400. However, division of the observed 458 L/day (121 gpd) per occupied campsite by an average of 4 persons per camping party yields a per capita use rate of about 110 L/day (30 gpd). Therefore the recommended 110 L/day (30 gpd) seems very reasonable. Peak water use

Table 4-6
Water Requirements for Recreation Areas

Type of Facility	Gallons per day[1]
Bath houses (per bather)	10
Camps: Construction, semipermanent (per worker)	50
Day with no meals served (per camper)	15
Luxury (per camper)	100-150
Resorts, day and night, with limited plumbing (per camper)	50
Tourist with central bath and toilet facilities (per person)	35
Laundries, self-serviced (gallons per washing, i.e., per customer)	50
Parks: Overnight, with flush toilets (per camper)	25
Trailers with individual bath units, no sewer connections (per trailer)	25
Trailers with individual bath, connected to sewer (per person)	50
Picnic: With bathhouses, showers, and flush toilets (per picnicker)	20
With toilet facilities only (gallons per picnicker)	10

[1] To convert to liters per day, multiply by 3.7854.

rates were not reported; however, some inferences can be drawn from the wastewater generation data that were presented. The ratio of peak hour to average daily wastewater flow varied from about 1.8 to about 4.2. It is reasonable to assume a similar ratio for peak hour to average daily water use since consumptive use at recreation areas is typically small (say 15 percent or so).

(2) Peak water.

(a) The existing guidance summarized in the previous section (4-6*b*(1)) is sufficient to estimate average water use for most recreation areas. However, little guidance with respect to peak use rates is available in Corps publications.

(b) Peak water use rates suitable for design purposes may be determined by consideration of particular facilities to be provided together with an estimate of the maximum expected visitation (Corps recreation facilities are normally designed to be used to capacity within 3 years of construction). One method that has been used successfully in many applications is based on the concept of "fixture units" (Hunter 1941). Each fixture or group of fixtures is assigned a relative peak demand rate in terms of fixture units. The total peak demand is normally determined by summing the fixture unit values of all the fixtures to be provided and then consulting a design curve. Hunter (1941), Salvato (1982), and USEPA (1980) present the basic information needed to use this method. When fixtures are likely to impose a continuous demand, the continuous portion of the demand should be estimated separately and added to the total fixture demand. Fixture unit values for fixtures not shown may be assumed by comparison to a similar fixture. The fixture unit values shown are for the total demand. Where applicable the hot and cold demand may each be estimated as three-fourths of the value shown. Salvato (1982) has reported that such estimates tend to err somewhat on the high side. This is not altogether undesirable, especially in the design of washhouses where showers and water heaters are to be provided. In such cases, insufficient capacity to deliver water on demand during peak periods can result in scalding (as well as other injuries resulting from panic, falling, etc.) to those taking showers. Information such as that presented in Metcalf and Eddy (1972), Salvato (1982), and USEPA (1974) may also be useful in estimating peak flow at recreation facilities.

4-7. Rest Area Water Use

a. Introduction. Water systems supplying highway rest areas are very similar (though they are often more compact) to recreation area systems. Thus, much of the information presented above, especially in paragraph 4-6, should be directly applicable. Design information specifically developed for highway rest area systems has been collected, organized, and published by the Federal Highway Administration (Folks 1977).

b. Design approach.

(1) Water use rates. The Federal Highway Administration suggests (Folks 1977) that water requirements may be estimated by using average daily traffic volumes for the six peak weekends, assuming that 9 percent of the vehicles will stop, and assuming that 25.4 L (6.7 gallons) of water will be needed per vehicle. A design period of 20 years is used unless there is some specific reason to do otherwise. Thus, the 20-year projected traffic volume should be used in the calculation. Peak water use rates may be estimated by consideration of operating data from existing systems that are similar. Francingues et al. (1975) have reported that approximately 16 percent of the daily use can be expected to occur in a 1-hour period around midday and approximately 67 percent occurs in an 8-hour period from about 8 a.m. to 4 p.m. Of course, special events can alter the timing of peak demands. Additional sources of demand may include fire protection (usually not provided), irrigation (varies from 25 mm (1 in.) per week to 76 mm (3 in.) per week during season depending upon the climate and specific needs), and drinking and wash water (for wastewater tanks) needed for recreational vehicles.

(2) Other considerations.

(a) For new construction, low-water-use fixtures should be used where possible. This practice, when combined with minimizing irrigation needs and avoiding using water for aesthetic purposes (e.g., fountains), unless a plentiful supply is readily available, will reduce the amount of water required without adversely affecting the function of the rest area.

(b) Storage requirements may be determined by the mass diagram approach mentioned in paragraph 4-3*b*(1) once average and peak water use rates are known. It is recommended (Folks 1977) that storage capacity be provided even if it is not absolutely necessary since pumping and pipe costs are generally reduced by storage. A detailed analysis should be conducted to determine the best solution for each individual site.

4-8. Water Conservation

a. Introduction. In recent years conserving water and energy by reducing water use or loss has received increasing attention. Most research reported so far pertains to the implementation of various conservation measures during critical periods (i.e., droughts) and indicates that, at least in the short run, use rates can be reduced dramatically. The extent to which such reductions can be sustained when and/or where water is plentiful is not fully known. However, many flow reduction devices and low-flow fixtures and appliances are now available, and most manufacturers of fixtures and appliances are presently replacing their standard lines with low-use models. Thus, some tendency toward long-term reduction in water use may be expected. AWWA is actively promoting water conservation programs to improve the efficiency of utility operations and reduce the waste of water. Information on possible water conservation practices is presented in AWWA (1975).

b. Flow reduction measures. The USEPA (1981) has reported on a study of various flow reduction measures and has synthesized information from a number of sources. The results indicate that not all flow reduction measures are equally economical. Data gathered from several sources and published by AWWA (1975) indicate that the cost of the water saved varies by orders of magnitude (i.e., 0 to $5 per 3800 L (1000 gallons) saved). It should be noted that many conservation measures are interrelated. Thus, the flow reduction expected to result from the application of a combination of measures may be less than the sum of the individual reductions expected for each measure applied alone (e.g., a pressure-reducing valve on the service line and a low- flow shower head would have considerable interaction).

c. Design implications. From the viewpoint of the design engineer the effects of water conservation programs can be twofold. Firstly, total operation and maintenance costs may be reduced as less water must be supplied. Unfortunately, fixed costs may cause unit operation and maintenance costs to rise; thus, customers may not realize significant dollar savings. Secondly, it may be possible to reduce the capacity of various water supply facilities and/or delay system expansion as the result of conservation. For small specialized water supply systems (such as those serving campgrounds), it may be possible to analyze the effects of conservation by simply considering the implications of the flow reduction expected to result from the use of low-water-use devices. For other systems, the problem may be considerably more complex since existing facilities (not necessarily fitted with water-saving devices) are often responsible for a considerable fraction of the total water use. The Corps has developed and reported (Fair, Geyer, and Okun 1966a; Planning and Management Consultants, Ltd., 1980a, 1980b) a method for forecasting the effects of flow reduction on the design of water supply systems that is applicable to these general cases. Since different conservation measures affect different aspects of total water use, it is a good idea to disaggregate water use into as many sectors as is practicable. A possible scheme for disaggregation was presented in Table 4-1. When conservation is being considered, the effect on each sector should be determined independently and then summed to determine total effect.