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Water and Wastewater Planning

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8.1 Standards

Waterworks, water distribution systems, sewerage, and sewage treatment works are an integrated system. The primary purpose of this system is to protect the public health and to prevent nuisances. This is achieved as follows:

- Waterworks produce potable waters that are free of pathogens and poisons.
- Water distribution systems prevent the posttreatment contamination of potable water while storing it and delivering it to users upon demand.
- Sewerage systems efficiently and safely collect contaminated used water, thereby preventing disease transmission and nuisance, and transmit it to sewage treatment works without loss or contamination of the surrounding environment.
- Sewage treatment works remove contaminants from the used water prior to its return to its source, thereby preventing contamination of the source and nuisance.

Overall, this system has been successful in controlling waterborne disease, and such disease is now rare in modern industrial economies.

The secondary purpose of sewage treatment is to preserve wildlife and to maintain an ambient water quality sufficient to permit recreational, industrial, and agricultural uses.

Water Treatment

Potable water quality in the U.S. is regulated by the U.S. EPA under authority of the “Safe Drinking Water Act” of 1974 (PL 93–523) and its amendments. The Act applies to any piped water supply that has at least 15 connections or that regularly serves at least 25 people. The U.S. EPA delegates day-to-day administration of the Act to the states. The fundamental obligations of the U.S. EPA are to establish primary regulations for the protection of the public health; establish secondary regulations relating to

taste, odor, color, and appearance of drinking water; protect underground drinking water supplies; and assist the states via technical assistance, personnel training, and money grants. Regulations include criteria for water composition, treatment technologies, system management, and statistical and chemical analytical techniques.

Maximum Contaminant Limits

The U.S. EPA has established *maximum contaminant limits* (MCL), which are legally enforceable standards of quality, and *maximum contaminant limit goals* (MCLG), which are nonenforceable health-based targets. The MCL are summarized in [Table 8.1](#). For comparative purposes, the earlier standards of the U.S. Public Health Service and the standards of the World Health Organization (WHO) are included.

The standards apply at the consumer's tap, not at the treatment plant or at any point in the distribution system.

It should be noted that the bacterial limits are no longer given as *most probable numbers* (MPN) or as *membrane filter counts* (MFC), but rather as the fraction of 100 mL samples that test positive in any month.

The limits on lead and copper are thresholds that require implementation of specific treatment processes to inhibit corrosion and scale dissolution.

Some substances are not yet subject to regulation, but in the interim, they must be monitored. Some substances must be monitored by all facilities; others must be monitored only if monitoring is warranted in the judgment of the state authority.

Violations of Drinking Water Regulations

Water supply systems must notify the people they serve whenever:

- A violation of a National Primary Drinking Water Regulation or monitoring requirement occurs
- Variances or exemptions are in effect
- Noncompliance with any schedule associated with a variance or exemption occurs

If the violation involves an MCL, a prescribed treatment technique or a variance/exemption schedule, a notice must be published in the local newspapers within 14 days. If there are no local newspapers, the notice must be given by hand delivery or posting. In any case, notification by mail or hand delivery must occur within 45 days, and notification must be repeated quarterly as long as the problem persists.

Notification must be made by television and radio within 72 h if any of the following occur: (1) the violation incurs a severe risk to human health as specified by a state agency, (2) the MCL for nitrate is violated, (3) the MCL for total coliform when fecal coliform or *Escherichia coli* are known to be present is violated, or (4) if there is an outbreak of waterborne disease in an unfiltered supply.

Wastewater Treatment

Wastewater discharges are regulated under the Federal Water Pollution Control Act of 1972 (PL 92–500), as amended.

Stream Standards

Terms like “pollution” and “contamination” require quantitative definition before abatement programs can be undertaken. Quantification permits engineering and economic analysis of projects. In the U.S., water bodies are first classified as to suitability for “beneficial uses.” Possible uses include:

- Wildlife preservation — warm water habitats, exceptional warm water habitats, cold water habitats
- Historic and/or scenic preservation
- Recreation — primary or contact recreation (i.e., swimming) and secondary or noncontact recreation (e.g., boating)
- Fisheries — commercial and sport
- Agricultural usage — crop irrigation and stock watering
- Industrial usage — process water, steam generation, cooling water
- Navigation

TABLE 8.1 Maximum Contaminant Concentrations Allowable in Drinking Water (Action Levels)

Parameter	Authority		WHO ^d
	U.S. PHS ^a	U.S. EPA ^{b,c}	
Pathogens and Parasites			
Total coliform bacteria (no./100mL)	1	<5% positive samples in a set of = 40 per month, or <1 sample positive in a set of <40 per month	0
Inorganic Poisons (mg/L)			
Antimony	—	0.006	—
Arsenic	0.05	0.05 (Interim)	0.05
Asbestos (Million fibers > 10 µM per liter)	—	7	—
Barium	1	2	—
Beryllium	—	0.004	—
Cadmium	0.01	0.005	0.005
Chromium (Total)	0.05	0.1	0.05
Copper	—	1.3 90 th percentile action level, requires corrosion control	—
Cyanide	0.2	0.2	0.1
Fluoride	See nuisances	4	—
Lead	0.05	0.015 90 th percentile action level, requires corrosion control	0.05
Mercury (inorganic)	—	0.002	0.001
Nickel	—	0.1	—
Nitrate (as N)	10	10	10
Nitrite (as N)	—	1	—
Nitrate plus nitrite (as N)	—	10	—
Selenium	0.01	0.05	0.01
Sulfate	—	Deferred (400 to 500?)	—
Thallium	—	0.002	—
Organic Poisons (µg/L, Except as Noted)			
Acrylamide	—	Use in treatment, storage, and distribution; restricted	—
Alachor	—	2	—
Aldicarb	—	3	—
Aldicarb sulfoxide	—	4	—
Aldicarb Sulfone	—	3	—
Aldrin and Dieldrin	—	—	0.03
Atrazine	—	3	—
Benzene	—	5	10
Benzo[a]pyrene	—	0.2	0.01
Bromobenzene	—	Monitor	—
Bromochloromethane	—	Monitor if ordered	—
Bromodichloromethane	—	Monitor	—
Bromoform	—	Monitor	—
Bromomethane	—	Monitor	—
<i>n</i> -Butylbenzene	—	Monitor if ordered	—
<i>sec</i> -Butylbenzene	—	Monitor if ordered	—
<i>tert</i> -Butylbenzene	—	Monitor if ordered	—
Carbofuran	—	40	—
Carbon chloroform extract	200	—	—

TABLE 8.1 (continued) Maximum Contaminant Concentrations Allowable in Drinking Water (Action Levels)

Parameter	Authority		
	U.S. PHS ^a	U.S. EPA ^{b,c}	WHO ^d
Carbon tetrachloride	—	5	—
Chlordane	—	2	0.3
Chlorobenzene	—	100	—
Chlorodibromomethane	—	Monitor	—
Chloroethane	—	Monitor	—
Chloroform	—	Monitor	30
Chloromethane	—	Monitor	—
<i>m</i> -Chlorotoluene	—	Monitor	—
<i>p</i> -Chlorotoluene	—	Monitor	—
2,4-D	—	70	100
Dalapon	—	200	—
DDT	—	—	1
1,2-Dibromo-3-chloropropane (DBCP)	—	0.2	—
Dibromomethane	—	Monitor	—
<i>m</i> -Dichlorobenzene	—	Monitor	—
<i>o</i> -Dichlorobenzene	—	600	—
<i>p</i> -Dichlorobenzene	—	75	—
Dichlorodifluoromethane	—	Monitor if ordered	—
1,1-Dichloroethane	—	Monitor	—
1,2-Dichloroethane	—	5	10
1,1-Dichloroethylene	—	7	0.3
<i>cis</i> -1,2-Dichloroethylene	—	70	—
<i>trans</i> -1,2-Dichloroethylene	—	100	—
Dichloromethane	—	5	—
1,2-Dichloropropane	—	5	—
1,3-Dichloropropane	—	Monitor	—
2,2-Dichloropropane	—	Monitor	—
1,1-Dichloropropene	—	Monitor	—
1,3-Dichloropropene	—	Monitor	—
Di(2-ethylhexyl)adipate	—	400	—
Di(2-ethylhexyl)phthalate	—	6	—
Dinoseb	—	7	—
Dioxin (2,3,7,8-TCDD)	—	30 × 10 ⁻⁹	—
Diquat	—	20	—
Endothall	—	100	—
Endrin	—	2	—
Epichlorhydrin	—	Use in treatment, storage, and distribution; restricted	—
Ethylbenzene	—	700	—
Ethylene dibromide (EDB)	—	0.05	—
Fluorotrichloromethane	—	Monitor if ordered	—
Glyphosate (aka Rodeo™ and Roundup™)	—	700	—
Heptachlor	—	0.4	0.1
Heptachlor epoxide	—	0.2	—
Hexachlorobenzene	—	1	0.01
Hexachlorobutadiene	—	Monitor if ordered	—
Hexachlorocyclopentadiene (HEX)	—	50	—
Isopropylbenzene	—	Monitor if ordered	—
<i>p</i> -Isopropyltoluene	—	Monitor if ordered	—
Lindane	—	0.2	3
Methoxychlor	—	40	30
Naphthalene	—	Monitor if ordered	—
Oxamyl (Vydate)	—	200	—
Pentachlorophenol	—	1	10
PCB (polychlorinate biphenyl)	—	0.5	—
Picloram	—	500	—

TABLE 8.1 (continued) Maximum Contaminant Concentrations Allowable in Drinking Water (Action Levels)

Parameter	Authority		
	U.S. PHS ^a	U.S. EPA ^{b,c}	WHO ^d
<i>n</i> -Propylbenzene	—	Monitor if ordered	—
Silvex (2,4,5-TP)	—	50	—
Simazine	—	4	—
Styrene	—	100	—
2,3,7,8-TCDD (Dioxin)	—	30 × 10 ⁻⁶	—
1,1,1,2-Tetrachloroethane	—	Monitor	—
1,1,2,2-Tetrachloroethane	—	Monitor	—
Tetrachloroethylene	—	5	—
Toluene	—	1000	—
Toxaphene	—	3	—
1,2,3-Trichlorobenzene	—	Monitor if ordered	—
1,2,4-Trichlorobenzene	—	70	—
1,1,1-Trichloroethane	—	200	—
1,1,2-Trichloroethane	—	5	—
Trichloroethylene	—	5	—
1,2,3-Trichloropropane	—	Monitor	—
2,4,6-Trichlorophenol	—	—	10
Trihalomethanes (Total)	—	100	—
2,4-Trimethylbenzene	—	Monitor if ordered	—
1,3,5-Trimethylbenzene	—	Monitor if ordered	—
Vinyl chloride	—	2	—
Xylene (Total)	—	10,000	—
<i>m</i> -Xylene	—	Monitor	—
<i>o</i> -Xylene	—	Monitor	—
<i>p</i> -Xylene	—	Monitor	—
Radioactivity (pCi/L, except as noted)			
Gross alpha (excl. Ra, u)	—	15	2.7
Gross beta	1000	—	27
Gross beta/photon (mrem/yr)	—	4	—
Radium-226	10	—	—
Radium-226 and 228	—	5	—
Radon-222	—	300	—
Strontium-90	3	—	—
Uranium (mg/L)	—	.03	—
Nuisances (mg/L, except as noted)			
Alkyl benzene sulfonate	0.5	—	—
Aluminum	—	—	0.2
Chloride	250	250	250
Color (Pt-Co Units)	15	15	15
Copper	1	See above	1
Corrosivity (Langelier Index)	—	— ^e	—
Fluoride	0.8–1.7	See above	—
Depending on air temperature			
Hardness (as CaCO ₃)	—	—	500
Hydrogen sulfide	—	—	— ^f
Iron	0.3	0.3	0.3
Manganese	0.05	0.05	0.1
Methylene blue active substances	—	0.5	—
Odor (threshold odor no.)	3	3	— ^g
pH	—	6.5/8.5	6.5/8.5
Phenol (µg/L)	1	—	—
Silver	0.05	0.05	—
Sodium	—	— ^e	200

TABLE 8.1 (continued) Maximum Contaminant Concentrations Allowable in Drinking Water (Action Levels)

Parameter	Authority		
	U.S. PHS ^a	U.S. EPA ^{b,c}	WHO ^d
Sulfate	250	500	400
Taste	—	—	— ^g
Total dissolved solids	500	500	1000
Turbidity (nephelometric units)	5	All samples = ≤5; 95% of samples ≤ 0.5	5
Zinc	5	5	5
Disinfectants and Disinfection Byproducts (mg/L)			
Chlorine	—	4.	—
Chloramines	—	4.	—
Chlorine dioxide	—	0.8	—
Total trihalomethanes	—	0.080	—
Haloacetic acids	—	0.060	—
Chlorite	—	1.0	—
Bromate	—	0.010	—
Total organic carbon	—	Treatment	—

^a Hopkins, O. C. 1962. *Public Health Service Drinking Water Standards 1962*. U.S. Department of Health Education, and Welfare, Public Health Service, Washington, DC.

^b Pontius, F. W. 1990. "Complying with the New Drinking Water Quality Regulations," *Journal of the American Water Works Association*, 82(2): 32.

^c Auerbach, J. 1994. "Cost and Benefits of Current SDWA Regulations," *Journal of the American Water Works Association*, 86(2): 69.

^d Anonymous. 1984. *Guidelines For Drinking Water Quality: Volume 1. Recommendations*. World Health Organization, Geneva, Switzerland.

^e To be monitored and reported to appropriate agency and/or public.

^f Not detectable by consumer.

^g Not offensive for most consumers.

- Hydropower
- Public water supply source (which assumes treatment prior to use)

Once a water body has been classified, the water volume and composition needed to sustain that usage can be specified. Such a specification is called a "stream standard." A water body is contaminated if any one of the various volume and composition specifications is violated. Stream standards are revised every 3 years. Standards are issued by state agencies subject to review and approval by the U.S. EPA. Commonly recommended standards for various beneficial uses are given in [Tables 8.2](#) through 8.6.

Rules-of-Thumb for Rivers

The determination of whether a particular discharge will cause a stream standard violation is difficult, particularly when the contaminants undergo physical, chemical, or biological transformations and when competing processes like biochemical oxygen demand (BOD) decay and reaeration occur. There are, however, a few rules-of-thumb that are useful guides to permit specification. The rules-of-thumb restrict BOD₅ and settleable solids, which adversely affect stream dissolved oxygen levels and ammonia, which is toxic to many fish.

Fuller (1912) reviewed several field studies conducted in France, Massachusetts, and Ohio on the effect of sewage discharges on rivers and reached the following conclusions:

- The sewage should be settled and skimmed to remove settleable and floatable material.
- The flow in the receiving stream should be at least 4 to 7 cfs per 1000 cap.

Fuller's recommendations result in an increase of 3 to 5 mg/L in the stream BOD₅. The requirement for sedimentation is especially important, because settleable solids have a disproportionate impact on stream oxygen values. Flotsam should be removed to avoid nuisance.

TABLE 8.2 Maximum Contaminant Concentrations Allowable in Sources of Public Water Supplies

Parameters	Authority		
	FWPCA ^a	U.S. EPA ^b	CEC ^c
Pathogens and Parasites (no./100mL)			
Total coliform bacteria	10,000	—	5000
Fecal coliform bacteria	2000	—	2000
Fecal streptococci	—	—	1000
Salmonella	—	—	— ^d
Inorganic Poisons (mg/L)			
Arsenic	0.05	0.05	0.05
Barium	1	1	1
Boron	1	—	1
Cadmium	0.01	0.01	0.005
Chromium	0.05	0.05	0.05
Cyanide	0.2	—	0.05
Fluoride	0.8–1.7	—	0.7/1.7
Lead	0.05	0.05	0.05
Mercury	—	0.002	0.001
Nitrate (as N)	10	10	11.3
Selenium	0.01	0.01	0.01
Silver	0.05	0.05	—
Organic Poisons (µg/L)			
Aldrin and Dieldrin	34	— ^e	—
Chlordane	3	— ^e	—
Chloroform extract	—	—	200
2,4-D (see herbicides)	—	100	—
DDT	42	— ^e	—
Endrin	1	0.2	—
Ether-soluble hydrocarbons	—	—	200
Heptachlor	18	— ^e	—
Heptachlor epoxide	18	—	—
Herbicides (total)	100	—	—
Lindane	56	4	—
Methoxychlor	35	100	—
Organophosphates and carbamates	100	—	—
Pesticides	—	—	2.5
Polycyclic aromatic hydrocarbons	—	—	0.2
Silvex (see herbicides)	—	10	—
Toxaphene	—	5	—
Radioactivity (pCi/L)			
Gross beta	1000	—	—
Radium-226	3	—	—
Strontium-90	10	—	—
Nuisances (mg/L, except as noted)			
Aesthetic qualities	—	— ^f	—
Ammonia (as NH ₄ ⁺)	0.64	—	1.5
Biochemical oxygen demand	—	—	5
Chloride	250	—	200
Color (Pt-Co units)	75	75	100
Conductivity (µs/L)	—	—	1000
Copper	1	1	0.05
Dissolved oxygen	>4.	—	>50.%sat

TABLE 8.2 (continued) Maximum Contaminant Concentrations Allowable in Sources of Public Water Supplies

Parameters	Authority		
	FWPCA ^a	U.S. EPA ^b	CEC ^c
Iron	0.3	0.3	2
Manganese	0.05	0.05	0.1
Methylene blue active substances	0.5	—	0.2
Odor (threshold odor no.)	— ^g	—	10
Oil and grease	— ^h	— ^h	—
pH (pH units)	6–8.5	5–9	5.5–9
Phenol	0.001	0.001	0.005
Phosphate (as P ₂ O ₅)	—	—	0.7
Sulfate	250	—	250
Tainting substances	—	— ⁱ	—
Temperature (°C)	30	—	25
Total dissolved solids	500	250	—
Total Kjeldahl nitrogen	—	—	2
Uranyl ion (as UO ₂ ²⁺)	5	—	—
Zinc	5	5	5

^a Ray, H. C., et al. 1968. Water Quality Criteria, Report of the National Technical Advisory Committee to the Secretary of the Interior. U.S. Department of the Interior, Federal Water Pollution Control Administration.

^b Anonymous. 1976. Quality Criteria for Water. U.S. Environmental Protection Agency, Office of Water Planning and Standards, Criteria and Standards Division, Criteria Branch, Washington, DC.

^c Council of the European Communities. 1975. Council Directive of 16 June 1975.

^d Absent in 1000 mL.

^e Human exposure to be minimized.

^f To be free from substances attributable to wastewaters or other discharges that (1) settle to form objectionable deposits; (2) float as debris, scum, oil, or other matter to form nuisances; (3) produce objectionable color, odor, taste, or turbidity; (4) injure or are toxic or produce adverse physiological response in humans, animals, or plants; and (5) produce undesirable or nuisance aquatic life.

^g Not objectionable.

^h Virtually free.

ⁱ Substances should not be present in concentrations that produce undesirable flavors in the edible portions of aquatic organisms.

In 1913, the Royal Commission on Sewage Disposal published its studies on the development of the biochemical oxygen demand test procedure and its applications. The Commission concluded that the BOD₅ of rivers should be held to less than 4 mg/L. This recommendation is supported by the work of Sladacek and Tucek (1975), who concluded that a BOD₅ of 4 mg/L produced a stream condition called “beta-mesosaprobic” (Kolkwitz and Marrson, 1908, 1909; Fjerdingstad, 1962), in which the stream benthos contains predominately clean water flora and fauna.

The unprotonated ammonia molecule, NH₃, is toxic to fish at concentrations about 0.02 mg NH₃/L (Anon., 1976). Ammonia reacts with water to form the ammonium ion, NH₄⁺, which is the predominant form in most natural waters:



The equilibrium constant for this reaction is called the “base ionization constant” and is defined as follows:

$$K_b = \frac{[\text{NH}_4^+] \cdot [\text{OH}^-]}{[\text{NH}_3]} \quad (8.2)$$

TABLE 8.3 Maximum Contaminant Concentrations Allowable in Recreational Waters

Parameter	Authority	
	FWPCA ^a	CEC ^b
Aesthetics	— ^c	— ^d
General recreational use		
Fecal coliform bacteria (no./100mL)	2000	—
Miscellaneous	— ^e	—
Primary contact recreation		
Total coliform bacteria (no./100mL)	—	10,000
Fecal coliform bacteria (no./100mL)	200	2000
Fecal streptococci (no./100mL)	—	100
Salmonella (no./1L)	—	0
Enteroviruses (PFU/10L)	—	0
pH (pH units)	6.5–8.3	6/9
Temperature (°C)	30	—
Clarity (Secchi disc, ft)	4	3.28
Color (Pt-Co units)	—	— ^f
Dissolved oxygen (% sat.)	—	80–120
Mineral oils (mg/L)	—	0.3
Methylene blue active substances (mg/L)	—	0.3
Phenols (mg/L)	—	0.005
Miscellaneous	— ^e	— ^g

^a Ray, H. C., et al. 1968. Water Quality Criteria, Report of the National Technical Advisory Committee to the Secretary of the Interior. U.S. Department of the Interior, Federal Water Pollution Control Administration.

^b Council of the European Communities. 1975. Council Directive of 16 June 1975.

^c All surface waters should be capable of supporting life forms of aesthetic value. Surface waters should be free of substances attributable to discharges or wastes as follows: (1) materials that will settle to form objectionable deposits; (2) floating debris, oil, scum, and other matter; (3) substances producing objectionable color, odor, taste, or turbidity; (4) materials, including radionuclides, in concentrations or in combinations that are toxic or that produce undesirable physiological responses in human, fish, and other animal life and plants; (5) substances and conditions or combinations thereof in concentrations that produce undesirable aquatic life.

^d Tarry residues and floating materials such as wood, plastic articles, bottles, containers of glass, rubber, or any other substance and waste or splinters shall be absent.

^e Surface waters, with specific and limited exceptions, should be of such quality as to provide for the enjoyment of recreational activities based on the utilization of fishes, waterfowl, and other forms of life without reference to official designation of use. Species suitable for harvest by recreational users should be fit for human consumption.

^f No abnormal change.

^g If the quality of the water has deteriorated or if their presence is suspected, competent authorities shall determine the concentrations of pesticides, heavy metals, cyanides, nitrates, and phosphates. If the water shows a tendency toward eutrophication, competent authorities shall check for ammonia and total Kjeldahl nitrogen.

It is also possible to write what is called an “acid ionization constant”:

$$K_a = \frac{[\text{H}^+] \cdot [\text{NH}_3]}{[\text{NH}_4^+]} \quad (8.3)$$

This corresponds to the reaction,



The ionization product of water relates the two constants:

$$K_a K_b = [\text{H}^+] \cdot [\text{OH}^-] = K_w \quad (8.5)$$

The molar fraction of the total ammonia concentration that is unprotonated ammonia is,

$$f = \frac{[\text{NH}_3]}{[\text{NH}_3] + [\text{NH}_4^+]} = \frac{1}{1 + \frac{[\text{H}^+]}{K_a}} = \frac{1}{1 + \frac{K_b}{[\text{OH}^-]}} \quad (8.6)$$

Consequently, a rule-of-thumb estimate of the allowable total ammonia concentration is,

$$\frac{0.02 \text{ mg NH}_3/\text{L}}{f} = \left(1 + \frac{[\text{H}^+]}{K_a} \right) \times 0.02 \text{ mg NH}_3/\text{L} \quad (8.7)$$

Values of the total ammonia concentration that correspond to an unprotonated ammonia concentration of 0.02 mg/L are given in [Table 8.7](#). These differ slightly from the values given by Thurston et al. (1974) because of rounding and differences in values of the acid ionization constants employed.

Rules-of-Thumb for Lakes

The most common problem in lakes is eutrophication (the overfertilization of a lacustrine ecosystem). Extreme cases of eutrophication lead to toxic algal blooms in the epilimnion and anoxia in the hypolimnion, and even mild cases lead to taste and odor problems and nuisance algal scums.

The potential for eutrophication can be judged by use of Vollenweider (1970) diagrams. These are shown in [Figs. 8.1](#) (for phosphorus) and [8.2](#) (for nitrogen). In each figure, the logarithm of the annual nutrient load per unit area is plotted against the mean depth of the lake, and each lake is judged to be eutrophic or oligotrophic based on the observed epilimnetic algal concentration and the hypolimnetic oxygen concentration. When the plotted points are labeled as to their lakes' trophic status, they form distinct groups as shown. The indicated boundary lines are:

For phosphorus (Fig. 8.1):

$$\text{oligotrophic:} \quad J_p \leq 0.025H^{0.6} \quad (8.8)$$

$$\text{eutrophic:} \quad J_p \geq 0.05H^{0.6} \quad (8.9)$$

For nitrogen (Fig. 8.2):

$$\text{oligotrophic:} \quad J_N \leq 0.4H^{0.6} \quad (8.10)$$

$$\text{eutrophic:} \quad J_N \geq 0.8H^{0.6} \quad (8.11)$$

TABLE 8.4 Maximum Contaminant Concentrations Allowable in Aquatic Habitats^a

Parameter	Habitat	
	Freshwater	Marine
Inorganic Poisons (mg/L, except as noted)		
Alkalinity (as CaCO ₃)	≥20	—
Ammonia (un-ionized)	0.02	—
Beryllium	—	—
Hard water	1.1	—
Soft water	0.011	—
Cadmium	—	5
Hard water	0.0012–0.012	—
Soft water	0.0004–0.004	—
Chlorine (total residual)	0.01	0.01
Salmonid fish	0.002	0.002
Chromium	0.1	—
Copper (X 96-h LC ₅₀)	0.1	0.1
Cyanide	0.005	0.005
Dissolved oxygen	≥5	—
Hydrogen sulfide (undissociated)	0.002	0.002
Iron	1	—
Lead (times the 96-h LC ₅₀)	0.01	—
Mercury (μg/L)	0.05	0.1
Nickel (times the 96-h LC ₅₀)	0.01	0.01
pH (pH units)	6.5/9	6.5/8.5
Phosphorus (elemental, μg/L as P)	—	0.1
Selenium (times the 96-h LC ₅₀)	0.01	0.01
Silver (times the 96-h LC ₅₀)	0.01	0.01
Total dissolved gases (% sat)	110	110
Organic Poisons (μg/L, except as noted)		
Aldrin and Dieldrin	0.003	0.003
Chlordane	0.01	0.004
DDT	0.001	0.001
Demeton	0.1	0.1
Endosulfan	0.003	0.001
Endrin	0.004	0.004
Guthion	0.01	0.01
Heptachlor	0.001	0.001
Lindane	0.01	0.004
Malathion	0.1	0.1
Methoxychlor	0.03	0.03
Mirex	0.001	0.001
Oil and grease (times the 96-h LC ₅₀) ^b	0.01	0.01
Parathion	0.04	0.04
Phthalate esters	3	—
Polychlorinated biphenyls	0.001	0.001
Toxaphene	0.005	0.005
Miscellaneous (mg/L, except as noted)		
Temperature increase (°C)	— ^c	1
Total phosphate (as P)	0.025	—
Turbidity	— ^d	—

^a Anonymous. 1976. Quality Criteria for Water. U.S. Environmental Protection Agency, Office of Water Planning and Standards, Criteria and Standards Division, Criteria Branch, Washington, DC.

TABLE 8.4 (continued) Maximum Contaminant Concentrations Allowable in Aquatic Habitats^a

^b Levels of oils or petrochemicals in the sediments that cause deleterious effects to the biota should not be allowed, and surface waters should be virtually free from floating nonpetroleum oils of vegetable or animal origin, as well as petroleum-derived oils.

^c In cooler months, maximum plume temperatures must be such that important species will not die if the plume temperature falls to the ambient water temperature. In warmer months, the maximum plume temperature may not exceed the optimum temperature of the most sensitive species by more than one-third of the difference between that species' optimum and ultimate upper incipient lethal temperatures. During reproductive seasons, the plume temperature must permit migration, spawning, egg incubation, fry rearing, and other reproductive functions of important species.

^d The compensation point for photosynthesis may not be reduced by more than 10% of the seasonally adjusted norm.

where H = the mean depth of the lake (m)

J_N = the annual nitrogen load per unit area of lake surface (g N/m² yr)

J_P = the annual phosphorus load per unit area of lake surface (g P/m² yr)

Total Daily Mass Load

The *total daily mass load* (TDML) is the total amount of a specific contaminant that can be discharged by all point and nonpoint sources to a specified receiving water without violating its water quality standards. It must include an allowance for uncertainty. Once the TDML is established for the whole receiving water, it (less the uncertainty allowance) may be allocated to individual point and nonpoint sources.

Nondegradation

The water quality act of 1972 includes a *nondegradation* provision. This means that there should be no measurable increase in contaminant levels, which as a practical matter, means that no contaminant concentration may be increased by more than 10%.

Effluent Standards

Because of the difficulty in assigning legal responsibility for stream standard violations when more than one discharge occurs, it is administratively easier to impose something called an "effluent standard." Each discharger is required to obtain a "National Pollutant Discharge Elimination System (NPDES)" permit from the competent state authority. The permit specifies the location, times, volume, and composition of the permitted discharge. The permit specifications are set by the state agency so as to prevent any violation of stream standards. However, any violation of a permit condition is prosecutable regardless of the impact on stream conditions.

Permit conditions for conservative contaminants, for poisons, for the traditional rules-of-thumb, and for antidegradation conditions are easily established by calculating a steady-state mass balance on the receiving stream at the point where the outfall meets it:

$$\bar{Q}_R \cdot \bar{C}_{QR} = \bar{Q}_W \cdot \bar{C}_{QW} + (\bar{Q}_R + \bar{Q}_W) \cdot \bar{C}_o \quad (8.12)$$

where \bar{C}_o = the stream standard for the most stringent beneficial use, generally a 1-day, 4-day, or 7-day average (kg/m³)

\bar{C}_{QR} = the flow-weighted (flow-composited) contaminant concentration in the river upstream of the outfall (kg/m³)

\bar{C}_{QW} = the flow-weighted (flow-composited) contaminant concentration in the wastewater (kg/m³)

TABLE 8.5 Maximum Contaminant Concentrations Allowable in Irrigation Water^a

Parameter	Concentration
Pathogens and Parasites (no./100mL)	
Total coliform bacteria	5000
Fecal coliform bacteria	1000
Inorganic Poisons (mg/L, except as noted)	
Aluminum	10
Arsenic	10
Beryllium	0.5
Boron	0.75
Cadmium	0.005
Chloride (meq/L)	20
Chromium	5.
Cobalt	0.2
Copper	0.2
Electrical conductivity (mmhos/cm)	1.5–18
Lead	5
Lithium	5
Manganese	2
Molybdenum	0.005
Nickel	0.5
pH (pH units)	4.5–9.0
Selenium	0.05
Vanadium	10
Zinc	5.
Herbicides (mg/L vs. corn)	
Acrolein	60
Amitrol-T	>3.5
Dalapon	<0.35
Dichlobenil	>10
Dimethylamines	>25
Diquat	125
Endothall	25
Fenac	10
Pichloram	>10
Radionuclides (pCi/L)	
Gross beta	1000
Radium-226	3
Strontium-90	10
Soil Deflocculation (units as noted)	
Exchangeable sodium ratio (%)	10–15
Sodium absorption ratio [(meq/L) ^{0.5}]	4–18

^a Ray, H. C., et al. 1968. Water Quality Criteria, Report of the National Technical Advisory Committee to the Secretary of the Interior. U.S. Department of the Interior, Federal Water Pollution Control Administration.

\bar{Q}_R = the time-averaged (steady state) river flow for the critical period, generally the 7-day-average low flow with a 10-year return period (m³/s)

\bar{Q}_W = the time-averaged (steady state) wastewater flow for the critical period, generally the maximum 7-day or 30-day average wastewater flow rate (m³/s)

TABLE 8.6 Maximum Contaminant Concentrations in Surface Waters that have been Used for Cooling Water^a

Parameter	Source			
	Freshwater		Brackish	
	Once Through	Recycle	Once Through	Recycle
Inorganic Substances (mg/L, except as noted)				
Acidity (as CaCO ₃)	0	200	0	0
Alkalinity (as CaCO ₃)	500	500	150	150
Aluminum	3	3	—	—
Bicarbonate	600	600	180	180
Calcium	500	500	1200	1200
Chloride	600	500	22,000	22,000
Hardness (as CaCO ₃)	850	850	7000	7000
Hydrogen sulfide	—	—	4	4
Iron	14	80	1	1
Manganese	2.5	10	0.02	0.02
Nitrate (as NO ₃)	30	30	—	—
pH (pH units)	5–8.9	3–9.1	5–8.4	5–8.4
Phosphate (as PO ₄)	4	4	5	5
Sulfate	680	680	2700	2700
Suspended solids	5000	15,000	250	250
Total dissolved solids	1000	1000	35,000	35,000
Organic Substances (mg/L)				
Carbon-chloroform extract	— ^b	100	— ^b	100
Chemical oxygen demand	—	100	—	200
Methylene blue active substances	1.3	1.3	—	1.3
Miscellaneous (units as noted)				
Color (Pt-Co units)	—	1200	—	—
Temperature (°F)	100	120	100	120

^a Ray, H. C., et al. 1968. Water Quality Criteria, Report of the National Technical Advisory Committee to the Secretary of the Interior. U.S. Department of the Interior, Federal Water Pollution Control Administration.

^b No floating oil.

TABLE 8.7 Total Ammonia Concentrations Corresponding to 0.20 mg/L of Unprotonated (Free) Ammonia

Temp. (°C)	Receiving Water pH								
	6.0	6.5	7.0	7.5	8.0	8.5	9.0	9.5	10.0
0	241	76.2	24.1	7.64	2.43	0.782	0.261	0.0962	0.0441
5	160	50.7	16.0	5.09	1.62	0.527	0.180	0.0707	0.0360
10	108	34.1	10.8	3.42	1.10	0.360	0.128	0.0540	0.0308
15	73.3	23.2	7.35	2.34	0.753	0.252	0.0933	0.0432	0.0273
20	50.3	15.9	5.04	1.61	0.522	0.179	0.0702	0.0359	0.0250
25	32.8	10.4	3.30	1.06	0.348	0.124	0.0528	0.0304	0.0233
30	24.8	7.85	2.50	0.803	0.268	0.0983	0.0448	0.0278	0.0225
35	17.7	5.62	1.79	0.580	0.197	0.0760	0.0377	0.0256	0.0218
40	12.8	4.06	1.30	0.424	0.148	0.0604	0.0328	0.0240	0.0213
45	9.37	2.98	0.955	0.316	0.114	0.0496	0.0294	0.0230	0.0209
50	6.94	2.21	0.712	0.239	0.0892	0.0419	0.0269	0.0222	0.0207

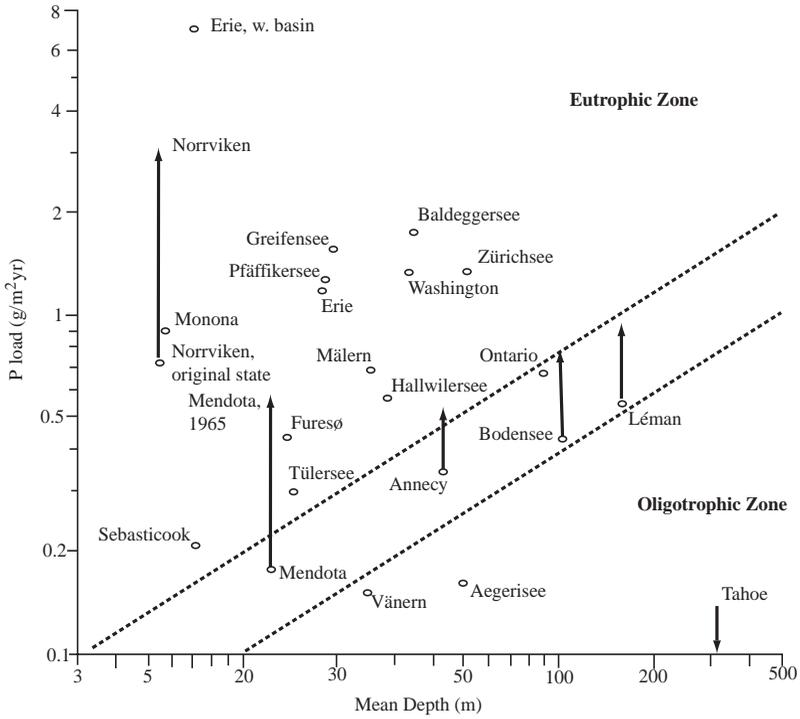


FIGURE 8.1 Classification of European and North American lakes by phosphorus loading and mean depth (International Joint Commission, 1969; Vollenweider, 1970).

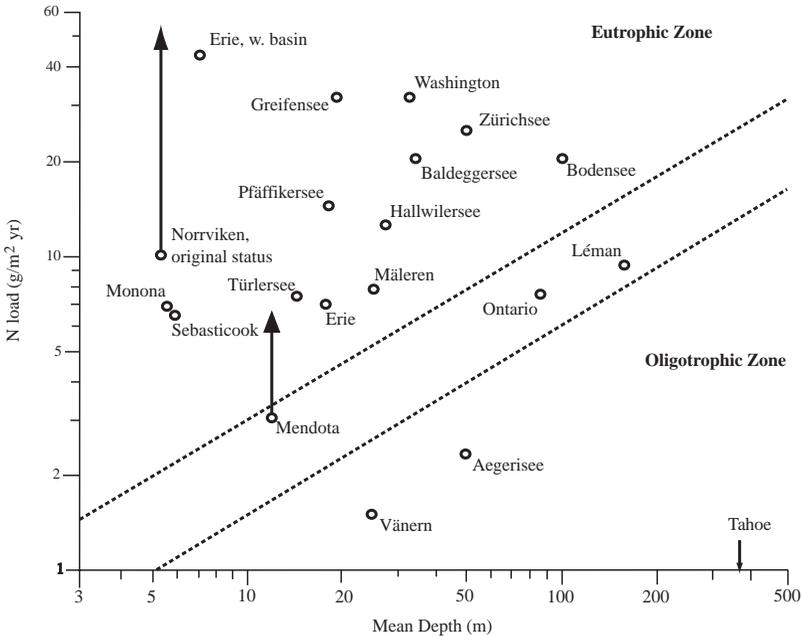


FIGURE 8.2 Classification of European and North American lakes by nitrogen loading and mean depth (International Joint Commission, 1969; Vollenweider, 1970).

The unknown in this case is the contaminant concentration in the treated effluent, \bar{C}_{QW} , and it becomes the NPDES permit condition. Sometimes the competent regulatory agency will “reserve” some stream assimilation capacity by using a \bar{C}_o value that is less than the relevant stream standard.

It should be noted that for the purpose of judging compliance with an NPDES permit, the Water Quality Act defines the effluent load to be the product of the arithmetically averaged contaminant concentration and the arithmetically averaged flow. This product will be larger or smaller than the true value depending on whether concentrations and flows are positively or negatively correlated.

For nonconservative contaminants, the permit conditions must be determined via water quality simulations using calibrated, verified models. The verification requires *in situ* field data that were collected and reduced in conformance with the model variables, such as time-averaged and cross-sectionally average concentrations, and during the appropriate drought flow. The process is time consuming and expensive.

Many substances undergo a simple first-order decay process, and the contaminant concentration downstream of a point source may be modeled as,

$$C(x) = C_o e^{-(kx/u)} + \frac{W}{kA} [1 - e^{-(kx/u)}] \quad (8.13)$$

where A = the cross-sectional area of the receiving stream (m^2)
 $C(x)$ = the contaminant concentration downstream of the outfall (kg/m^3)
 C_o = the initial contaminant concentration at the outfall (kg/m^3)
 k = the contaminant decay rate (per s)
 u = the mean stream velocity (m/s)
 W = the (uniformly) distributed load along the stream reach below the outfall ($kg/m \text{ s}$)
 x = the distance below the outfall (m)

In this case, the effluent permit for C_w (which determines C_o), would be set to prevent the maximum downstream concentration from exceeding the water quality standard. For a very long reach, this maximum would be W/kA .

Some substances like oxygen undergo both sink and source processes, and the models become more elaborate. The simple Streeter-Phelps (1925) model for oxygen includes the effects of both carbonaceous BOD oxidation and reaeration:

$$D(x) = D_o e^{-(k_a x/u)} + \frac{k_d L_o}{k_a - k_d} [e^{-(k_d x/u)} - e^{-(k_a x/u)}] \quad (8.14)$$

where $D(x)$ = the oxygen deficiency downstream of a point source (kg/m^3)
 D_o = the initial oxygen deficit at the outfall (kg/m^3)
 k_d = the carbonaceous BOD decay rate (per s)
 k_a = the oxygen reaeration rate (per s)
 L_o = the initial ultimate carbonaceous BOD at the outfall (kg/m^3)

Thomann and Mueller (1987) and Chapra (1997) present and discuss more detailed and realistic models for a wide variety of receiving waters, contaminants, and processes.

The minimum requirements of a NPDES discharge permit are shown in [Table 8.8](#). These requirements define secondary biological treatment. While such requirements might be imposed on a small discharge to a large body of water, most permits are much more stringent in order to meet relevant water quality standards. More comprehensive permits include:

- Separate restrictions for summer and winter conditions
- Limits on ammonia
- Limits on specific substances known to be in the influent wastewater

TABLE 8.8 Default National Pollution Discharge Elimination System Limits^a

Parameter	Averaging Period	
	7-Days	30-Days
Five-Day Biochemical Oxygen Demand		
Maximum effluent concentration (mg/L)	45	30
Removal efficiency (%)	—	85
Suspended Solids		
Maximum effluent concentration (mg/L)	45	30
Removal Efficiency (%)	—	85
pH (pH Units)		
Minimum	—	6
Maximum	—	9

Note: More stringent limits may be imposed for water quality in limited receiving waters. Other contaminants will be restricted as necessary to maintain water quality standards.

^a Environmental Protection Agency (1976). "Secondary Treatment Information: Biochemical Oxygen Demand, Suspended Solids and pH," Federal Register, 41(144): 30788.

- More stringent sampling and monitoring requirements, perhaps at various points throughout the treatment facility or tributary sewers instead of only the outfall
- Specification of particular treatment processes

The more important parameters, like CBOD₅, are subject to the more stringent sampling programs. Some parameters like flow, oxygen, and chlorine are easily monitored continuously. The discharger may be requested to monitor parameters that have no apparent environmental impact in order to develop databases for future permits.

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8.2 Planning

The problem of projecting future demands may be subdivided into two parts, namely, selection of (1) the planning period and (2) the projection technique.

Selection of Planning Period

In order to assess all the impacts of a project, the planning period should be at least as long as the economic life of the facilities. The U.S. Internal Revenue Service publishes estimates of the economic life of equipment, buildings, etc. This is especially important for long-lived facilities, because they tend to attract additional demand beyond that originally planned for.

Buildings have economic lives on the order of 20 years. For large pipelines, the economic life might be 50 years, and dams might have economic lives as long as 100 years. The U.S. Government usually requires a planning period of 20 years for federally subsidized projects. Usefully accurate projections, however, cannot be made for periods much longer than 10 years. The demands projected for the economic life of a project cannot, therefore, be regarded as likely to occur. Rather, the projected demands set the boundaries of the problem. That is, they provide guides to the maximum plant capacity, storage volume, land area, etc., that might be needed. Preliminary facilities designs are made using these guides, but the actual facilities construction is staged to meet shorter-term projected demands. The longer-term projections and plans serve to guarantee that the various construction stages will produce an integrated, efficient facility, and the staging permits reasonably accurate tracking of the actual demand evolution.

Optimum Construction Staging

The basic question is, how much capacity should be constructed at each stage? This is a problem in cost minimization. Consider Fig. 8.3. The smooth curves represent the projected demand over the economic life of some sort of facility, say a treatment plant, and the stepped lines represent the installed capacity. Note that installed capacity always exceeds projected demand. Public utilities usually set their prices to cover their costs. This means that the consumers generally will be paying for capacity they cannot use, and it is desirable to minimize these excess payments on the grounds of equity and efficiency.

The appropriate procedure is to minimize the present worth of the costs for the entire series of construction stages. This is done as follows. The present worth of any future cost is calculated using the prevailing interest rate and the time elapsed between now and the actual expenditure:

$$PW = \sum_{j=0}^n C_j \exp\{-it_j\} \quad (8.15)$$

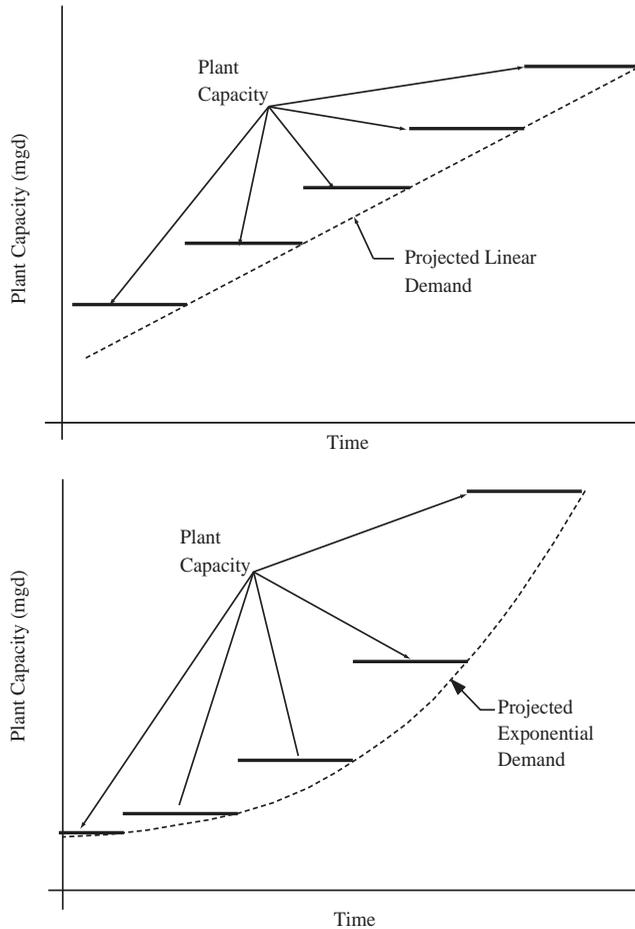


FIGURE 8.3 Capacity construction scheduling for linear and exponential growth.

where PW = the present worth of the j th cost

C_j = the j th cost at the time it is incurred

i = the prevailing interest rate (per year)

t_j = the elapsed time between now and the date of the j th cost

Manne (1967) and Srinivasan (1967) have shown that for either linearly or exponentially increasing demands, the time between capacity additions, Δt , should be constant. The value of Δt is computed by minimizing PW in Eq. (8.15). To do this, it is first necessary to express the cost, C_j , as a function of the added capacity, ΔQ_j . Economies of scale reduce the average cost per unit of capacity as the capacity increases, so the cost-capacity relationship may be approximated functionally as:

$$C = kQ^a \quad (8.16)$$

where a, k = empirical coefficients determined by regression of costs on capacity

C = the cost to construct a facility of capacity Q (\$)

Q = the capacity (m^3/s)

For water treatment plants, a is usually between 0.5 and 0.8 (Clark and Dorsey, 1982).

Suppose that the demand is projected to increase linearly over time:

$$Q = Q_0 + rt \quad (8.17)$$

where Q_0 = the demand at the beginning of the planning period (m³/s)
 r = the linear rate of increase of demand (m³/s per year)

The capacity additions are equal to:

$$\Delta Q = r\Delta t \quad (8.18)$$

and the cost of each addition is:

$$C_j = k(r\Delta t)^a \quad (8.19)$$

Substituting Eq. (8.19) into Eq. (8.15), the present worth of the series of additions becomes:

$$PW = \sum_{j=0}^n k(r\Delta t)^a \exp\{-ij\Delta t\} \quad (8.20)$$

The problem is somewhat simplified if the number of terms in the summation, n , is allowed to become infinite, because for that case, the series converges to a simpler expression:

$$PW = \frac{k(r\Delta t)^a}{1 - \exp\{-i\Delta t\}} \quad (8.21)$$

The value of Δt that yields the minimum value of the present worth is determined by taking logarithms of both sides of Eq. (8.21) and differentiating with respect to Δt :

$$a = \frac{i\Delta t}{\exp\{i\Delta t\} - 1} \quad (8.22)$$

It should be noted that for linearly increasing demand, the optimum time between capacity additions, i.e., the time that minimizes the discounted expansion costs, depends only on the interest rate, i , and the economies of scale factor, a . The annual rate of demand increase, r , does not affect the result, and the coefficient k does not affect the result.

It is more usual to project exponentially increasing demands:

$$Q = Q_0 \exp\{rt\} \quad (8.23)$$

In this case, the capacity additions are still spaced equally in time, but the sizes of the additions increase exponentially (Srinivasan, 1967):

$$PW = \frac{k(Q_0 [\exp\{r\Delta t\} - 1])^a}{1 - \exp\{-(i - ar)\Delta t\}} \quad (8.24)$$

$$a = \frac{(i - ar)[1 - \exp\{-r\Delta t\}]}{r[\exp\{(i - ar)\Delta t\} - 1]} \quad (8.25)$$

In this case, the exponential demand growth rate enters the calculation of Δt .

An exponential growth rate of 2%/year is about twice the national average for the U.S., so Eq. (8.22), which assumes linear growth and is easier to solve, may provide adequate accuracy for most American cities. However, urban areas in some developing countries are growing much faster than 2%/year, and Eq. (8.25) must be used in those cases.

Different components of water supply systems have different cost functions (Rachford, Scarato, and Tchobanoglous, 1969; Clark and Dorsey, 1982), and their capacities should be increased at different time intervals. The analysis just described can also be applied to the system components. In this case, the various component expansions need to be carefully scheduled so that bottlenecks are not created.

Finally, it should be remembered that the future interest rates and growth rates used to estimate the optimum expansion time interval are uncertain. Allowing for these uncertainties by adopting high estimates of these rates in order to calculate “conservative” values for the expansion capacity and costs is not the economically optimum strategy (Berthouex and Polkowski, 1970). Considering first the interest rate, if the range of possible interest rates can be estimated, then the optimum strategy is to adopt the midpoint of the range. The same strategy should be used when selecting growth rates: if the projected growth is exponential, adopt the midrange value of the estimated exponential growth rates. However, for linear growth, a different strategy is indicated: the minimal cost of expansion is achieved by using a linear growth rate somewhat less than the midrange value of the estimated rates. The “under design” capacity increment should be about 5 to 10% less than the capacity increment calculated using the midrange rate.

Population Projections

Because water demand is proportional to population, projections of water demand are reduced to projections of population. Engineers in the U.S. no longer make population projections. Population projection is the responsibility of designated agencies, and any person, company, or agency planning future public works is required to base those plans on the projections provided by the designated agency.

Projection errors of about 10% can be expected for planning periods less than 10 years, but errors in excess of 50% can be expected if the planning period is 20 years or more (McJunkin, 1964). Because nearly all projections are for periods of 20 years or more, all projections are nonsense if one regards them as predictions of future conditions. Their real function is regulatory. They force the engineer to design and build facilities that may be easily modified, either by expansion or decommissioning.

The four most commonly used methods of population projection are as follows:

- Extrapolation of historical census data for the community’s total population
- Analysis of components-of-change (alias cohort analysis, projection matrix, and Leslie matrix)
- Correlation with the total population of larger, surrounding regions
- Estimation of ultimate development

The first method was the principal one employed prior to World War II. Nowadays, cohort analysis is nearly the only method used.

Population Extrapolation Methods

Extrapolation procedures consist of fitting some assumed function to historical population data for the community being studied. The procedures differ in the functions fitted, the method of fitting, and the length of the population record employed. The functions usually fitted are the straight line, the exponential, and the logistic:

$$P = P_o + rt \quad (8.26)$$

$$P = P_o \exp\{rt\} \quad (8.27)$$

$$P = \frac{P_{\max}}{1 + \left(\frac{P_{\max}}{P_0} - 1 \right) \exp\{-rt\}} \quad (8.28)$$

where P = the population at time t (capita)
 P_{\max} = the maximum possible number of people (capita)
 P_0 = the population at the beginning of the fitted record (capita)
 r = the linear or exponential growth rate (number of people per year or per year, respectively)
 t = the elapsed time from the beginning of fitted record (yr)

The preferred method of fitting Eqs. (8.26) through (8.28) to historical population data is least squares regression. Because the exponential and logistic equations are nonlinear, they should be fitted using a nonlinear least squares procedure. Nonlinear least squares procedures are iterative and require a computer. Suitable programs that are available for IBM™-compatible and Macintosh™ personal computers are Minitab™, SAS™, SPSS™, and SYSTAT™.

Components-of-Change

At the present time, the preferred method of population projection is the method of components-of-change. The method is also known as cohort analysis, the projection matrix, and the Leslie (1945) matrix. It is a discretized version of a continuous model originally developed by Lotka (1956).

In this method, the total population is divided into males and females, and the two sexes are subdivided into age groups. For consistency's sake, the time step used in the model must be equal to the age increment. The usual subdivision is a 5-year increment; therefore, the age classes considered are 0/4, 5/9, 10/14, 15/19, etc. This duration is chosen because it is exactly half the interval between censuses, so that every other time step corresponds to a census.

Consider first the female age classes beginning with the second class, i.e., 5/9; the 0/4 class will be considered later. For these higher classes, the only processes affecting the number of females in each age class are survival and migration. The relevant equation is:

$$\begin{aligned} &[\text{number of females in age class "i + 1" at time "t + } \Delta t] = \\ &[\text{number of females in age class "i" at time "t" that survive } \Delta t] + \\ &[\text{number of female migrants during } \Delta t \text{ of age class "i + 1"}] \\ &P_f(i+1, t + \Delta t) = l_f(i, t) \cdot P_f(i, t) + M_f(i+1, t + \Delta t) \end{aligned} \quad (8.29)$$

where $l_f(i, t)$ = the fraction of females in the age class "i" (i.e., aged "5·i" through "5·i + 4" years) at time "t" that survive Δt years (here 5)
 $M_f(i + 1, t + \Delta t)$ = the net number of female migrants (immigrants positive, emigrants negative) that join the age class "i + 1" between times "t" and "t + Δt "
 $P_f(i, t)$ = the number of females in the age class "i" at time "t"
 $P_f(i + 1, t + \Delta t)$ = the number of females in age class "i + 1" (i.e., aged "5·(i + 1)" through "5·(i + 1) + 4" years) at time "t + Δt "

The survival fractions, $l_f(i, t)$, can be calculated independently using the community's death records. The migration rates, however, are calculated using Eq. (8.29). This is done by using census data for $P_f(i + 1, t + \Delta t)$ and $P_f(i, t)$ and the independently calculated $l_f(i, t)$. Consequently, any errors in the census data or death records are absorbed into the migration rate.

It is a matter of convenience how the migration is expressed, and it is most convenient to express it as a correction to the calculated number of survivors:

$$M_f(i+1, t + \Delta t) = m_f(i, t) \cdot l_f(i, t) \cdot P_f(i, t) \quad (8.30)$$

where $m_f(i, t)$ = the female migration rate for age class “ $i + 1$,” in units of number of females joining age class “ $i + 1$ ” during the period “ t ” to “ $t + \Delta t$ ” per survivor of age class “ i ”

With this definition, Eq. (8.23) can be written as follows:

$$\begin{aligned} P_f(i+1, t + \Delta t) &= l_f(i, t) \cdot [1 + m_f(i, t)] \cdot P_f(i, t) \\ &= l_{ef}(i, t) \cdot P_f(i, t) \end{aligned} \quad (8.31)$$

where $l_{ef}(i, t)$ = the effective fraction of females in age class “ i ” that survive from time “ t ” to “ $t + \Delta t$ ”
 $= l_f(i, t) \cdot [1 + m_f(i, t)]$

Now consider the first female age class, which is comprised of individuals aged 0 to 4 years. The processes affecting this class are birth and migration. What is needed is the net result of births and migrations for the 5-year period Δt . However, birth rates are usually given in terms of the number of births per female per year. Therefore, a 1-year rate must be converted into a 5-year rate. Furthermore, over a 5-year period, the number of females in any given age class will change due to death and migration. The birth rate also changes. These changes are handled by averaging the beginning and end of period rates:

$$\begin{aligned} [\text{total births for age class “}i\text{” from “}t\text{” to “}t + \Delta t\text{”}] &= \\ &= \frac{5}{2} \cdot [b_f(i, t) \cdot P_f(i, t) + b_f(i, t + \Delta t) \cdot P_f(i, t + \Delta t)] \end{aligned} \quad (8.32)$$

where $b_f(i, t)$ = the female birth rate for females in age class “ i ” at time “ t ” in units of female babies per female per year.

Note that the factor “5” is needed to convert from 1-year to 5-year rates, and the factor “2” is needed to average the two rates.

The total number of births is obtained by summing Eq. (8.32) over all the age classes. If this is done, and if terms containing like age classes are collected, the total number of births is:

$$\begin{aligned} P_f(0, t + \Delta t) &= \frac{5}{2} \cdot \sum_{i=0}^k [b_f(i, t) + b_f(i+1, t + \Delta t) \cdot l_{ef}(i, t)] \cdot P_f(i, t) \\ &= \sum_{i=0}^k \bar{b}_f(i, t) \cdot P_f(i, t) \end{aligned} \quad (8.33)$$

where $\bar{b}_f(i, t)$ = the average female birth rate for females in the age class “ i ” for the period “ t ” to “ $t + \Delta t$ ”
 $= 5/2 \cdot [b_f(i, t) + b_f(i + 1, t + \Delta t) \cdot l_{ef}(i, t)]$

Equations 8.31 and 8.33 are most conveniently written in matrix form as follows:

$$\begin{bmatrix} P_f(0, t + \Delta t) \\ P_f(1, t + \Delta t) \\ \vdots \\ P_f(k, t + \Delta t) \end{bmatrix} = \begin{bmatrix} \bar{b}_f(0, t) & \bar{b}_f(1, t) & \dots & \bar{b}_f(k, t) \\ l_{ef}(0, t) & 0 & \dots & 0 \\ 0 & l_{ef}(1, t) & \dots & 0 \\ 0 & 0 & l_{ef}(k-1, t) & 0 \end{bmatrix} \begin{bmatrix} P_f(0, t) \\ P_f(1, t) \\ \vdots \\ P_f(k, t) \end{bmatrix} \quad (8.34)$$

The first row in the coefficient matrix has numerous zeros (because very young and very old women are not fertile), and the main diagonal is entirely zeros.

The model for males is slightly more complicated. First, for all age classes other than the first (i.e., for $i = 1, 2, 3, \dots, k$), the number of males depends solely on the fraction of the previous male age class that survives plus any migration that occurs:

$$\begin{aligned} P_m(i+1, t + \Delta t) &= l_m(i, t) \cdot [1 + m_m(i, t)] \cdot P_m(i, t) \\ &= l_{em}(i, t) \cdot P_m(i, t) \end{aligned} \tag{8.35}$$

where $l_{em}(i, t)$ = the effective fraction of males in age class “ i ” that survive from time “ t ” to “ $t + \Delta t$ ”
 $= l_m(i, t) \cdot [1 + m_m(i, t)]$
 $l_m(i, t)$ = the fraction of males in the age class “ i ” (i.e., aged “ $5 \cdot i$ ” through “ $5 \cdot i + 4$ ” years) at time “ t ” that survive Δt years (here 5)
 $m_m(i, t)$ = the male migration rate for age class “ $i + 1$,” in units of number of males joining age class “ $i + 1$ ” during the period “ t ” to “ $t + \Delta t$ ” per survivor of age class “ i ”
 $P_m(i, t)$ = the number of males in the age class “ i ” at time “ t ”
 $P_m(i + 1, t + \Delta t)$ = the number of males in age class “ $i + 1$ ” [i.e., aged “ $5 \cdot (i + 1)$ ” through “ $5 \cdot (i + 1) + 4$ ” years] at time “ $t + \Delta t$ ”

The male birth rate depends on the number of females, not males, and the number of births must be written in terms of the population vector for women. The number of births for men (aged 0 to 4) is:

$$P_m(0, t + \Delta t) = \sum_{i=0}^k \bar{b}_m(i, t) \cdot P_f(i, t) \tag{8.36}$$

where $b_m(i, t)$ = the male birth rate for females in age class “ i ” at time “ t ,” in units of male babies per female per year
 \bar{b}_m = the average male birth rate for females in age class “ i ” at time “ t ,” in units of the total number of male babies born during the interval Δt (here 5 years) per female in age class “ i ”
 $= 5/2 \cdot [b_m(i, t) + b_m(i + 1, t + \Delta t) \cdot l_{em}(i, t)]$
 $P_m(0, t + \Delta t)$ = the number of males in age class “0” (i.e., aged 0 to 4 years) at time “ $t + \Delta t$ ”

Consequently, the projection for males involves the population vectors for both males and females and coefficient matrices for each and is written:

$$\begin{aligned} \begin{bmatrix} P_m(0, t + \Delta t) \\ P_m(1, t + \Delta t) \\ \vdots \\ P_m(k, t + \Delta t) \end{bmatrix} &= \begin{bmatrix} \bar{b}_m(0, t) & \bar{b}_m(1, t) & \dots & \bar{b}_m(k, t) \\ 0 & 0 & \dots & 0 \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & \dots & 0 \end{bmatrix} \begin{bmatrix} P_f(0, t) \\ P_f(1, t) \\ \vdots \\ P_f(k, t) \end{bmatrix} + \dots \\ &\dots + \begin{bmatrix} 0 & 0 & \dots & 0 \\ l_{em}(0, t) & 0 & \dots & 0 \\ 0 & l_{em}(1, t) & \vdots & 0 \\ 0 & 0 & l_{em}(k-1, t) & 0 \end{bmatrix} \begin{bmatrix} P_m(0, t) \\ P_m(1, t) \\ \vdots \\ P_m(k, t) \end{bmatrix} \end{aligned} \tag{8.37}$$

Correlation Methods

In the correlation method, one attempts to use the projected population of a larger, surrounding area, say a county or state, to estimate the future population of the community being studied. This method is sometimes preferable to a direct projection of the community’s population, because there are usually

more accurate and more extensive data available for the larger area. This is especially true for smaller cities. The projected population of the larger, surrounding area is usually obtained by the method of components, which is described above.

The correlation method entails plotting the available population data from the surrounding area and the community against one another. It is assumed that the community is following the pattern of growth exhibited by the surrounding area, or something close to it. The correlation takes the form:

$$P_c(t) = b_1 \cdot P_a(t) + b_0 \quad (8.38)$$

where b_0, b_1 = regression coefficients

$P_a(t)$ = the population of the larger, surrounding area at time t

$P_c(t)$ = the population of the community at time t

Ultimate Development

The concept of ultimate or full development is actually somewhat nebulous, but nonetheless useful, at least for short-term projections. Almost all communities have zoning regulations that control the use of developed and undeveloped areas within the communities' jurisdictions. Consequently, mere inspection of the zoning regulations suffices to determine the ultimate population of the undeveloped areas.

There are several problems with this method. First, it cannot assign a date for full development. In fact, because population trends are not considered, the method cannot determine whether full development will ever occur. Second, the method cannot account for the chance that zoning regulations may change, e.g., an area zoned for single family, detached housing may be rezoned for apartments. Zoning changes are quite common in undeveloped areas, because there is no local population to oppose them. Third, the method cannot account for annexations of additional land into the jurisdiction. However, it should be noted that the second and third problems are under the control of the community, so it can force the actual future population to conform to the full development projection.

Siting and Site Plans

Flood Plains

Water and wastewater treatment plants are frequently built in the flood plain. Of course, the intakes of surface water treatment plants and the outfalls of wastewater treatment plants are necessarily in the flood plain. However, the remainder of the facilities are often built in risky areas either to minimize the travel time of maintenance crews or because — in the case of wastewater plants — the site is the low point of a drainage district.

The Wastewater Committee requires that wastewater treatment plants be fully operational during the 25-year flood and that they be protected from the 100-year flood (Wastewater Committee, 1990). The intake pumping stations of water treatment plants should be elevated at least 3 ft above the highest level of either the largest flood of record or the 100-year flood or should be protected to those levels (Water Supply Committee, 1987).

Permits

Water and wastewater treatment plants require a number of permits and must conform to a variety of design, construction, and operating codes.

Federal Permits

The National Environmental Policy Act of 1969 (PL 91–190) requires an Environmental Impact Statement (EIS) for "... major federal actions significantly affecting the quality of the human environment." The EIS should consider alternatives to the proposed action, both long-term and short-term effects, irreversible and irretrievable commitments of resources, and unavoidable adverse impacts. An EIS may be required even if the only federal action is the issuance of a permit. An EIS is not required if the relevant federal agency (here, the U.S. Army Corps of Engineers or U.S. EPA) certifies that there is no significant environmental impact.

The fundamental federal permit is the National Pollution Discharge Elimination System (NPDES) permit, which the Federal Water Pollution Control Act of 1972 requires for each discharge to a navigable waterway. Authority to grant permits is vested in the U.S. EPA, however, it has delegated that authority to the various states, and as a practical matter, dischargers apply to the relevant state agency.

Treatment plants may also be required to obtain a 404 permit from the U.S. Army Corps of Engineers if the proposed facility creates an obstruction in or over a navigable waterway.

A broadcasting license may be required by the Federal Communications Commission if radio signals are used for telemetry and remote control.

Besides these permits and licenses, treatment plants must be designed in conformance with a number of federal regulations, including:

- Clean Air Act requirements governing emissions from storage tanks for chemicals and fuels (Title 40, Parts 50 to 99 and 280 to 281)
- Americans with Disabilities Act of 1990 (PL 101–336) requirements regarding access for handicapped persons
- Occupational Health and Safety Act requirements regarding workplace safety

State and Local Permits

The design of water and wastewater treatment plants is subject to review and regulation by state agencies. Such reviews are normally conducted upon receipt of an application for a NPDES permit. The states are generally required to make use of U.S. EPA guidance documents regarding the suitability of various treatment processes and recommended design criteria. Some states also explicitly require adherence to the so-called “Ten States Standards,” which are more correctly referred to as Great Lakes-Upper Mississippi River Board of State Public Health and Environmental Managers’ “Recommended Standards for Water Works” and “Recommended Standards for Wastewater Facilities.”

States also require permits for construction in floodplains, because large facilities will alter flood heights and durations.

Highway departments require permits for pipelines that cross state roads. Generally, such crossings must be bored or jacked so that traffic is not disrupted.

Facilities must also conform to local codes, including:

- State building codes [usually based on the Building Official and Code Administrators (BOCA) Building Code]
- State electrical codes [usually based on the National Electrical Code as specified by the National Fire Protection Association (NFPA-70)]
- State plumbing code

These are usually administered locally by county, municipal, or township agencies. Most commonly, the agencies require a permit to build (which must be obtained before construction and which requires submission of plans and specifications to the appropriate agencies) and a permit to occupy (which requires a postconstruction inspection).

In many cases, design engineers voluntarily adhere to or local regulations require adherence to other specifications, e.g.:

- General materials specifications, sampling procedures, and analytical methods — American Society for Testing and Materials (1916 Race Street, Philadelphia, PA 19103–1187)
- Specifications for water treatment chemicals and equipment — American Water Works Association (6666 West Quincy Avenue, Denver, CO 80235)
- Specifications for fire control systems and chemical storage facilities — National Fire Protection Association (1 Batterymarch Park, PO Box 9101, Quincy, MA 02269–9191)
- Specifications for chemical analytical methods — Water Environment Federation (601 Wythe Street, Alexandria, VA 22314–1994) and American Water Works Association (6666 West Quincy Avenue, Denver, CO 80235)

Other Permits

Railroad crossings require the permission of the railroad. As with highways, pipelines must be bored or jacked under the roadbed so that traffic is not disrupted.

Excavation almost always occurs near other utilities, and this should be coordinated with them.

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8.3 Design Flows and Loads

Flow and Load Averaging

Designers must account for the inherent variability in flows and loads. This is done by defining various duration averages, maxima, and minima. The average flow for some specified time period, T , is for continuous and discrete data, respectively,

$$\bar{Q} = \frac{1}{T} \int_0^T Q(t) \cdot dt \quad (8.39)$$

$$\bar{Q} = \frac{\sum_i Q_i}{n} \quad (8.40)$$

where n = the number of discrete flow measurements during the specified period (dimensionless)
 $Q(t)$ = the flow at time instant t (m^3/s)
 \bar{Q} = the arithmetic mean volumetric flow for the specified period (m^3/s)

Q_i = the "ith" flow measurement during the specified period (m^3/s)
 T = the duration of the specified period (s)
 t = the time elapsed since the beginning of the period (s)

The usual averaging periods (T) are:

- The annual average, alias average day
- The maximum (or minimum) 1-h average
- The maximum (or minimum) 24-h average
- The maximum (or minimum) 3-day average
- The maximum (or minimum) 7-day average
- The maximum (or minimum) 30-day average

In each case, the database consists of 12 consecutive months of flow and load records. A maximum average flow or load for some specified period is computed by identifying the continuous time interval in the annual record that is equal in duration to the specified period and that produces the maximum total volume of flow or mass of contaminant load. The minimum averages are determined similarly. If the NPDES permit is written with seasonal limits, the maxima and minima should be determined using seasonal rather than annual data.

The *instantaneous* contaminant load is the product of the instantaneous contaminant concentration and the instantaneous wastewater volumetric flow rate:

$$W(t) = Q(t) \cdot C(t) \quad (8.41)$$

$$W_i = Q_i \cdot C_i \quad (8.42)$$

where $C(t)$ = the contaminant concentration at time instant t (kg/m^3)

C_i = the "ith" measurement of the contaminant concentration (kg/m^3)

$W(t)$ = the load at time instant t , (kg/s)

W_i = the "ith" load measurement (kg/s)

The *average* contaminant load during some interval T is calculated as,

$$\bar{W} = \frac{1}{T} \int_0^T W(t) \cdot dt \quad (8.43)$$

$$\bar{W} = \frac{\sum_i W_i}{n} \quad (8.44)$$

where \bar{W} = the average contaminant load for the specified period (kg/s).

The Average Concentration

The most natural and useful definition of an average concentration is the *flow-weighted* (*flow-composited*) concentration:

$$\bar{C}_Q = \frac{\int_0^T Q(t) \cdot C(t) \cdot dt}{\int_0^T Q(t) \cdot dt} = \frac{\bar{W}}{\bar{Q}} \quad (8.45)$$

$$\bar{C}_Q = \frac{\sum_i Q_i \cdot C_i}{\sum_i Q_i} = \frac{\bar{W}}{\bar{Q}} \quad (8.46)$$

where \bar{C}_Q = the flow-weighted (flow-composited) average concentration (kg/m³).

Note that the (arithmetic) average load is the product of the (arithmetic) average flow and the flow-weighted average concentration.

The simple arithmetic average concentration is frequently encountered,

$$\bar{C} = \frac{1}{T} \int_0^T C(t) \cdot dt \quad (8.47)$$

$$\bar{C} = \frac{\sum_i C_i}{n} \quad (8.48)$$

where \bar{C} = the arithmetic average contaminant concentration (kg/m³).

The general relationship between the average load and the arithmetic average concentration is,

$$\bar{W} = \bar{Q} \cdot \bar{C} + \text{CoVar}(Q, C) = \bar{Q} \cdot \bar{C} + \rho_{QC} \cdot \sigma_Q \cdot \sigma_C \quad (8.49)$$

where $\text{CoVar}(Q, C)$ = the covariance of Q and C (kg/s)

$$= \frac{\sum_i (Q_i - \bar{Q})(C_i - \bar{C})}{n}$$

ρ_{QC} = the Pearson product-moment correlation coefficient (dimensionless)

$$= \frac{\sum_i (Q_i - \bar{Q})(C_i - \bar{C})}{\sqrt{\sum_i (Q_i - \bar{Q})^2} \cdot \sqrt{\sum_i (C_i - \bar{C})^2}}$$

σ_C = the standard deviation of the contaminant concentration measurements (kg/m³)

$$= \sqrt{\frac{\sum_i (C_i - \bar{C})^2}{n}}$$

σ_Q = the standard deviation of the flow measurements (m³/s)

$$= \sqrt{\frac{\sum_i (Q_i - \bar{Q})^2}{n}}$$

Note that the average load is equal to the product of the average flow and arithmetic average concentration only if the flows and concentrations are uncorrelated ($\rho_{QC} = 0$). This is generally false for raw wastewaters, but it may be true for biologically treated effluents. In most raw wastewaters, the flows and concentrations are positively correlated. Therefore, the product $\bar{Q} \cdot \bar{C}$ underestimates the mean load. The underestimate can be substantial.

For NPDES permits, the U.S. EPA defines the average load to be the product of the arithmetic average flow and the arithmetic average concentration, $\bar{Q} \cdot \bar{C}$. The great majority of discharges are biologically treated effluents, so there is no significant error in the EPA's definition.

Annual Average per Caput Flows and Loads

The annual average per capita water demand is the total volume of water supplied during a complete year divided by the mid-year population. The usual units are either gallons per caput per day (gpcd) or liters per caput per day (Lpcd). Sometimes the volumes are expressed as cubic feet or cubic meters. The calculation proceeds as follows:

$$\bar{Q}_{pc} = \frac{1}{T} \int_0^T \frac{Q(t) \cdot dt}{P(t)} = \frac{1}{n} \sum_{i=1}^n \frac{Q_i}{P_i} \quad (8.50)$$

$$\bar{W}_{pc} = \frac{1}{T} \int_0^T \frac{Q(t) \cdot C(t) \cdot dt}{P(t)} = \frac{1}{n} \sum_{i=1}^n \frac{Q_i \cdot C_i}{P_i} \quad (8.51)$$

where \bar{Q}_{pc} = the annual average per caput water demand (m³/caps)
 \bar{W}_{pc} = the annual average per caput load (kg/caps)

Because the rate of population increase is usually only a few percent per year, or less, the integral and sum are evaluated by assuming P is constant and equal to the mid-year population. P is then factored out, and the integral and sum become the total volume of water supplied in gallons or liters during the year.

Automatic Samplers

Some automatic samplers capture a constant sample volume at uniformly spaced time increments: the data from these samplers are arithmetically averaged concentrations. Other samplers capture sample volumes at uniformly spaced time increments, but the sample volumes are proportional to the simultaneous flow rate: the data from these samplers are flow-weighted average concentrations. Flow-weighted concentrations are also produced by samplers that collect constant sample volumes but adjust the sampling time increments so that each sample represents the same total flow volume.

Water Treatment

The flow through water treatment plants is controlled by the operator, and this simplifies the design process. The designer must still allow for variations in consumption, but these can be satisfied by system storage of finished water.

Annual Average per Caput Flows and Loads

In public water supplies, the water demand is the sum of the volumes supplied to households, commercial and industrial enterprises, and public agencies. An approximate breakdown by use of the water withdrawn for public supplies in 1980 is given in [Table 8.9](#). The estimates were compiled from several independent sources, and they may each be in error by several gpcd. The data are national averages, and particular cities will exhibit substantial divergences from them. In 1980, public water supplies in the U.S. served 186 million people (81% of the total population), and their total withdrawal from surface and ground waters amounted to 183 gpcd (Solley, Chase, and Mann, 1983). Of the total withdrawal, 66% was obtained from surface waters, and 34% came from ground waters. And, 21% of the withdrawal went to consumptive uses, most of which was lawn sprinkling.

These national averages mask a wide range of local variations (Solley, Chase, and Mann, 1983). For example, the amount of water withdrawn in 1980 varied from 63 gpcd in the Virgin Islands to 575 gpcd in Utah, and the consumptive use varied from none in Delaware to 71% in Wyoming. In general, per caput water withdrawals are greatest in the hot, arid Colorado River valley and the Great Basin (341 gpcd, 45% consumptive) and least in the wetter, cooler New England states (148 gpcd, 10% consumptive).

TABLE 8.9 Estimate of Uses of the Annual Average Withdrawal by Public Supplies in 1980

Use	Rate (gpcd)	Percent of Total Withdrawal
Total withdrawal	183	100 ^a
Commercial/industrial	63	34 ^a
Public	35	19 ^b
Unaccounted for	20	11 ^{c,d}
Treatment losses	8	4 ^e
Metered agencies	7	4 ^b
Households	85	46 ^{f,g}
Lawn sprinkling	25	14 ^{f,g}
Toilet flushing	25	14 ^h
Bathing	20	11 ^h
Laundry	8	4 ^h
Kitchen	6	3 ^h
Drinking	0.5	0.2 ^h

^a Solley, W. B., Chase, E. B., and Mann, W. B., IV. 1983. Estimated Water Use in the United States in 1980, Circular No. 1001. U.S. Geological Survey, Distribution Branch, Text Products Division, Alexandria, VA.

^b By difference, but see: Schneider, M. L. 1982. Projections of Water Usage and Water Demand in Columbus, Ohio: Implications for Demand Management, M.S. Thesis. The Ohio State University, Columbus.

^c Seidel, H. F. 1985. "Water Utility Operating Data: An Analysis," *Journal of the American Water Works Association*, 77(5): 34.

^d Keller, C. W. 1976. "Analysis of Unaccounted-for Water," *Journal of the American Water Works Association*, 68(3): 159.

^e Lin, S., Evans, R. L., Schnepfer, D., and Hill, T. 1984. Evaluation of Wastes from the East St. Louis Water Treatment Plant and Their Impact on the Mississippi River, Circular No. 160. State of Illinois, Department of Energy and Natural Resources, State Water Survey, Champaign.

^f Linaweaver, F. P., Jr., Geyer, J. C., and Wolff, J. B. 1966. Final and Summary Report on the Residential Water Use Research Project, Report V, Phase Two. The Johns Hopkins University, Department of Environmental Engineering Science, Baltimore, MD.

^g King, G. W., et al., eds. 1984. Statistical Abstract of the United States 1985, 105 ed. U.S. Government Printing Office, Washington, DC.

^h Bailey, J. R., Benoit, R. J., Dodson, J. L., Robb, J. M., and Wallman, H. 1969. A Study of Flow Reduction and Treatment of Waste Water from Households, Water Pollution Control Research Series 11050FKE 12/69. U.S. Department of the Interior, Federal Water Quality Administration, Washington, DC.

About one-eighth of the withdrawal is lost either as treatment byproducts (i.e., sludges and brines) or as unaccounted for water. The latter category includes such things as unmetered (i.e., free) water, leakage, fire control, and metering errors.

One-third of the withdrawal is supplied to commercial and industrial enterprises, although many industrial plants have independent supplies.

Somewhat less than half the withdrawal is supplied to households. The principal uses there are lawn sprinkling, toilet flushing, and bathing. The particular estimates of aggregate household use cited in [Table 8.9](#) are the weighted average of the demands reported by Linaweaver, Geyer, and Wolff (1966) for detached, single-family houses and apartments that had metered water and sewerage. It was assumed that detached, single-family homes comprised 67% of the total number of housing units, and that the

TABLE 8.10 Per Caput Rates of Withdrawal by Public Water Supplies in the U.S. from 1870 to 1980

Year	Withdrawal (gpcd)
1870	55 ^a
1907	121 ^b
1950	145 ^c
1960	151 ^c
1970	166 ^c
1980	183 ^c

^a Amount supplied, not withdrawn. Fanning, J. T. 1882. *A Practical Treatise on Hydraulic and Water-Supply Engineering: Relating to the Hydrology, Hydrodynamics, and Practical Construction of Water-Works in North America*, 3rd ed. D. Van Nostrand, New York.

^b Amount supplied, not withdrawn. Metcalf, L., Gifford, F. J., and Sullivan, W. F. 1913. "Report of Committee on Water Consumption Statistics and Records," *Journal of the New England Water Works Association*, 27(1): 29.

^c King, G. W., et al., ed. 1984. *Statistical Abstract of the United States 1985*, 105 ed. U.S. Government Printing Office, Washington, DC.

remainder were apartments. For detached, single-family homes with metered water and septic tanks, a better estimate of usage exclusive of lawn sprinkling is 44 gpcd (Schmidt et al., 1980), and lawn sprinkling might be reduced by two-thirds (Linaweaver, Geyer, and Wolff, 1966). For high-income areas, the lawn sprinkling usage might be doubled, but in-house usage would only increase by about 10%.

The per caput annual average withdrawal rate for public supplies rose steadily at about 1%/year for over 100 years. Some statistics are given in Table 8.10. The rate of increase from 1870 is 1.09%/year, and the rate since 1950 is 0.78%/year. The annual increase in withdrawals may have declined or even ceased since the mid-1980s because of conservation efforts.

Because it is probable that no city in the U.S. has a usage breakdown like that shown in Table 8.9, local usage projections should always be based on local data. This is not a very severe requirement, because all American cities already have water supply systems, and the records maintained by these systems can be used as the basis of design. National average data is more useful for making adjustments to local usage projections to account for factors like income levels, water prices, and climate.

Peak Demand

Some reported peaking factors for entire cities are given in Table 8.11. The 1882 data given by Fanning are for large cities in the Northeast, and the 1913 data given by Metcalf et al. are for cities and towns in Massachusetts. The data for 1932 and 1936 were obtained from numerous cities of all sizes throughout the country. The peak data for 1936 were obtained during a severe drought, and the peak-to-average ratio was calculated using the average demand for June of 1935. Wolff's data were obtained from cities in the Northeast.

The detailed data reported in the cited papers indicate that the peak demands for the 4-h averaging period are only slightly smaller than those for the 1-h averaging period. The 3-day averages are also nearly equal to the 1-day averages. However, the ratio of peak-to-average demands generally declines as the peak averaging period increases, as it must.

The detailed data also show a weak tendency for the peak-to-average ratio to decline as the population rises. This tendency is not apparent in the data for 1932 and 1936, which represent about 400 cities nationwide. However, for 70 cities in the Northeast, Wolff (1957) reported the following ratios of peak 1-day to average day demands:

TABLE 8.11 Reported Ratios of Peak to Average per Capita Demands for American Cities

	Averaging Period					
	30 days	7 days	3 days	1 day	4 h	1 h
1882 ^a	1.017	1.027	—	—	—	1.731
1913 ^b	1.28	1.47	—	1.98	—	—
1932 ^c						
<10,000 cap.	—	—	—	—	—	3.30
>10,000 cap.	—	—	—	—	—	2.98
5000/20,000 cap.	—	—	—	1.66	—	—
>122,000 cap.	—	—	—	1.53	—	2.34
1936 ^d	—	—	1.75	1.82	2.40	2.79
1957 ^e	—	—	—	≤2.09	—	3.30

^a Fanning, J. T. 1882. *A Practical Treatise on Hydraulic and Water-Supply Engineering: Relating to the Hydrology, Hydrodynamics, and Practical Construction of Water-Works in North America*, 3rd ed. D. Van Nostrand, New York.

^b Metcalf, L., Gifford, F. J., and Sullivan, W. F. 1913. "Report of Committee on Water Consumption Statistics and Records," *Journal of the New England Water Works Association*, 27(1): 29.

^c Folwell, A. P. 1932. "Maximum Daily and Hourly Water Consumption in American Cities," *Public Works*, 63(10): 13.

^d Anonymous. 1936. "Record-Breaking Consumptions: Established by Severe Drought and High Summer Temperatures," *Water Works Engineering*, 89(19): 1236.

^e Wolff, J. B. 1957. "Forecasting Residential Requirements," *Journal of the American Water Works Association*, 49(3): 225.

For purely residential areas:

$$\frac{\text{maximum 1-day average demand}}{\text{annual average demand}} = 2.51P^{-0.0855} \quad (8.52)$$

For combined residential and industrial areas:

$$\frac{\text{maximum 1-day average demand}}{\text{annual average demand}} = 2.09P^{-0.0574} \quad (8.53)$$

where P = the service population in thousands.

In view of the insensitivity of peaking factors to population, the peaking factors in Table 8.12 are recommended for American communities. These ratios are somewhat higher than those recommended a generation ago (e.g., Fair, Geyer, and Morris, 1954), but the amount of lawn sprinkling has increased substantially since then, and lawn sprinkling is a major component of the peak demands. The same ratios may be used for residential neighborhoods, with two exceptions (Linaweaver, Geyer, and Wolff, 1966).

TABLE 8.12 Recommended Peaking Factors for Water Demand

Type of Community	Averaging Period					
	30 days	7 days	3 days	1 day	4 h	1 h
Population greater than 5000 with commerce and industry	1.3	1.5	2	2	3.3	3.3
Purely residential and resort ^{a,b}	1.9	2.2	—	2.9	5	6.2

^a Henderson, A. D. 1956. "The Sprinkling Load—Long Island, NY, and Levittown, PA," *Journal of the American Water Works Association*, 48(4): 361.

^b Critchlow, H. T. 1951. "Discussion: S.K. Keller (1951), 'Seasonal Water Demands in Vacation Areas,'" *Journal of the American Water Works Association*, 43(9): 701.

The exceptions are newly developed lots where new lawns are being established and very large lots in high-income areas. In these areas, the ratio of peak hour to average day demands reaches 15 to 20.

In the late 19th Century, peak demands in northern cities usually occurred in the winter. This was a result of the need to discharge water through hydrants and household taps to prevent freezing, the water mains often being shallow and the houses poorly heated. By the early 20th Century, lawn sprinkling had shifted the peak demand to summer. By 1930, 55% of American cities reported that peak demand occurred in July, and 25% reported that it occurred in August (Folwell, 1932). Only 15% of the cities experienced peak demands in the winter.

The peaking factors cited above are weighted toward Eastern cities, which receive significant rainfall throughout the summer. This somewhat mitigates lawn sprinkling. Many Western cities, however, receive little or no rain in the summer, and residents often water their lawns several times a week. The practical result of this practice is that the so-called maximum daily average demand occurs nearly every day during the Western summer (Linaweaver, Geyer, and Wolff, 1966).

Finally, there is the issue of fire demands. The amount of water used for fire control over an entire year is small, and it might not affect the peak demand statistics for a whole city. However, in the neighborhood of an ongoing fire, the fire demand dominates the local water requirements and controls the design of the local storage and distribution systems. In the U.S., water supply systems are usually designed in accordance with the recommendations of the Insurance Services Office (ISO). These recommendations are used to set fire insurance premiums for new cities and for small cities for which there is little data on fire insurance claims. For old, large cities that have ample statistics on claims, premiums are usually set on an actuarial basis.

The ISO calculates a "Needed Fire Flow" (NFF_i) at each of several representative locations in the city. It is assumed that only one building is on fire at any time. The formula for the NFF_i is (Anonymous, 1980):

$$NFF_i = C_i \cdot O_i \cdot (X + P)_i \quad (8.54)$$

where NFF_i = the needed fire flow (gpm)

C_i = the fire flow based on the size of the building and its construction (ranging from 500 to 8000 gpm)

O_i = a correction factor for the kinds of materials stored in the building (ranging from 0.75 to 1.25)

$(X + P)_i$ = a correction factor for the proximity of other buildings (ranging from 0.0 to 1.75)

Criteria for assigning values to the various factors in Eq. (8.54) are given in the ISO recommendations. In any case, the NFF_i must be not be greater than 12,000 gpm or less than 500 gpm. For one- and two-family dwellings, the NFF_i ranges from 500 to 1500 gpm; for larger dwellings, NFF_i is increased up to a maximum of 3500 gpm. The water supply system should be able to deliver to the fire location the total of the NFF_i and the maximum 24-h average demand, while maintaining a system pressure of at least 20 psig everywhere. For an NFF_i of 2500 gpm or less, the total flow must be sustainable for 2 h; for an NFF_i between 3000 and 3500 gpm, it must be sustainable for 3 h; and for an NFF_i greater than 3500 gpm, it must be sustainable for 4 h.

These recommendations refer to the water distribution system. The treatment plant design flow need not be the total of the NFF_i and the maximum 24-h average demand, so long as there is sufficient storage capacity to make up the difference between plant capacity and the total flow.

Factors Affecting Household Demand

The principal factors affecting household water demand are household income, water price, climate, and conservation regulations. They primarily affect lawn sprinkling, and there is a strong climatic effect that distinguishes the humid East from the arid West.

In general, lawn sprinkling demand amounts to about 60% of the net evapotranspiration rate (Linaweaver, Geyer, and Wolff, 1966):

$$Q_{ls} = 0.60 \cdot A_l \cdot (E_{pot} - P_{eff}) \quad (8.55)$$

where Q_{ls} = the lawn sprinkling demand (annual, monthly, or daily average, as appropriate, in m^3/s)
 A_l = the lawn area (hectares)
 E_{pot} = the potential evapotranspiration rate (annual, monthly, or daily average to conform to Q_{ls} in $m^3/ha \cdot s$)
 P_{eff} = the effective precipitation rate (annual, monthly, or daily average to conform to Q_{ls} in $m^3/ha \cdot s$)

Lawn sprinkling demand is modified by household income and water price. Lawn sprinkling demand elasticities for income and price are calculated as:

$$\eta_I = \frac{dQ_{ls}/Q_{ls}}{dI/I} \quad (8.56)$$

$$\eta_C = \frac{dQ_{ls}/Q_{ls}}{dC/C} \quad (8.57)$$

where C = water cost ($$/m³)
 I = household income ($$/yr)
 Q_{ls} = lawn sprinkling demand (m^3/s)
 η_C = the elasticity of lawn sprinkling demand with respect to water cost (dimensionless)
 η_I = the elasticity of lawn sprinkling demand with respect to household income (dimensionless)$$

In metered, sewered Eastern communities, the income and price elasticities for the summer average lawn sprinkling demand are about 1.5 and -1.6, respectively (Howe and Linaweaver, 1967). This means that if household income were increased by 10%, lawn sprinkling demand would be increased by 15%; if water prices were increased by 10%, lawn sprinkling demand would be reduced by 16%. In metered, sewered Western communities, however, the income and price elasticities are only 0.4 and -0.7, respectively.

These differences in response are due to climate. In the humid East, there are frequent summer rains, and reductions in lawn sprinkling will not result in loss of lawns. In the arid West, however, summer rains are infrequent or nonexistent, and lawns will die without regular irrigation.

Prices and incomes have less effect on peak daily lawn sprinkling demands than on the summer average. This is an artifact of utility billing practices (Howe and Linaweaver, 1967). Because consumers are billed only monthly or quarterly for the total amount of water used, the cost of a single afternoon's watering is not a major portion of the bill and does not deter water use. If necessary, peak daily demands can be curtailed by use prohibitions (Heggie, 1957).

Wastewater Treatment

It has been the custom in the U.S. to use the annual average flows and loads as the basis of process design, exceptions being made for process components that are especially sensitive to maximum and/or minimum flows and loads. However, nowadays, NPDES permits generally are written in terms of 30-day and 7-day average limits, and this suggests that the design basis should be a maximum 30-day average or maximum 7-day average load or flow (Joint Task Force, 1992). If other averages are written into the permit, they should become the design basis. If the permit conditions are seasonal, then the averages should be calculated on a seasonal basis, e.g., a maximum 30-day average load for the winter, a maximum 30-day average load for the summer, etc. Consequently, the design of each component in a treatment plant must consider several different loading and flow conditions, and the most severe condition will control its design.

Table 8.13 is a compilation of recommended design flows and loads for various treatment plant components. The basic design period is assumed to be either the maximum 7-day or 30-day average flow or load, depending on which is most severe given the associated temperature and composition. Most regulatory authorities still use the annual average load or flow in their regulations.

TABLE 8.13 Recommended Design Flows and Loads for Wastewater Treatment Plant Components^a

Process or Material	Design Period	Peaking Factor Peak Value/Annual Average
Operating costs		
Chemicals	Annual average	1
Electricity	Annual average	1
Ultimate hydraulic capacity of tanks, conduits, pumps, weirs, screens, etc.	Transmit the instantaneous annual maximum flow with one of each parallel units out of service; check solids deposition at minimum flows	Maximum and minimum rated capacities of influent sewer or pumping station
Preliminary treatment		
Coarse screens size	Max. 7 or 30 day ave. flow*	2 or 1.4
Screenings storage volume	Max. 3-D ave. load	4
Grit chambers size	Max. 1-h ave flow	3
Grit storage volume	Max. 3-D ave. load	4
Comminutor size	Max. 1-h ave. flow	3
Influent pumping station	Max. 1-h ave. flow <u>and</u> min. 1-h ave. flow	3 and 0.33
Influent sewer	Max. 1-h ave. flow <u>and</u> min. 1-h ave. flow	3 and 0.33
Flow meters	Max. 1-h ave. flow <u>and</u> min. 1-h ave. flow	3 and 0.33
Primary treatment		
Primary clarifier	Max. 1-h ave. flow <u>and</u> max. 7-day or 30-day ave. flow*	3 and 2 or 1.4
Outlet weir length	Max. 1-h ave. flow	3
Outlet trough capacity	Max. 1-h ave. flow <u>and</u> ½ max. 7-day or 30-day max. ave. flow	3 and 2 or 1.4
Sludge volume	Max. 30-day ave. load	1.3
Scum volume	Max. 7-day or 30-day ave. load	1.6 or 1.3
Activated sludge		
FM or SRT	50 to 150% of value for max 7-day or max 30-day ave load*	2.4 or 1.6
Aeration volume	Max. SRT or min. F/M at max. 7-day or 30-day ave. BOD load*	2.4 or 1.6
Aeration tank weirs and troughs	Max. instantaneous flow plus max. recycle	3
Air supply	Max. 1-h ave. BOD load; include NOD for nitrifying systems	2.8
Temperature (nitrification)	30-day ave. preceding critical period	—
Alkalinity (nitrification)	Min. 7-day or 30-day ave. load	—
Waste sludge volume and dry solids	Min. SRT or max F/M at max 7-day or 30-day ave. BOD load*	1.6 or 1.3
Return sludge pumps	Max. 7-day or 30-day ave. flow*	2.0 or 1.4
Final clarifier size	Max. 1-h ave. and max. 7-day or 30-day ave. flow * and max. 24-h ave. solids load	3.0 and 2.0 or 1.4 and 2.8(Q + Q _r)/MLSS
Trickling filters		
Media volume	Max. 24-h BOD load	2.4
Distributor and recirculation system	Max. 7-day or 30-day ave. flow* and min. 1-h ave. flow	2 or 1.4 and 0.33
Underdrains	Max. 7-day or 30-day ave. flow* plus recirculation	(2 or 1.4)Q + Q _r
Intermediate clarifiers	Max. 1-h ave. flow	3
Final clarifier size	Max. 1-h ave. flow	3
Wastewater ponds		
Facultative pond area	Max. 7-day or 30-day ave. BOD load	1.6 or 1.3
Facultative pond volume	Ann. ave. flow	1
Aerated pond volume	Ann. ave. flow	1
Aerated pond air supply	Max. 7-day or 30-day ave. BOD load	1.6 or 1.3
Effluent sand filters		
Filter area	Max. 1-h ave. flow	3
Disinfection		
Chlorine contactor volume	Max. 1-h ave. flow	3

TABLE 8.13 Recommended Design Flows and Loads for Wastewater Treatment Plant Components^a

*Note: The Wastewater Committee (1990) uses the annual average flow wherever the table recommends the maximum 7-day or 30-day average flow.

^a Compiled from the following sources:

- Fair, G. M., Geyer, J. C., and Morris, J. C. 1954. *Water Supply and Waste-Water Disposal*. John Wiley & Sons, Inc., New York.
- Joint Task Force. 1992. *Design of Municipal Wastewater Treatment Plants: Volume 1—Chapters 1–12*. Water Environment Federation, Alexandria, VA; American Society of Civil Engineers, New York.
- Metcalf & Eddy, Inc. 1991. *Wastewater Engineering: Treatment, Disposal, and Reuse*, 3rd ed., revised by G. Tchobanoglous and F. L. Burton. McGraw-Hill, Inc., New York.
- Mohlman, F. W., Chairman, Thomas, H. A. Jr., Secretary, Fair, G. M., Fuhrman, R. E., Gilbert, J. J., Heacox, R. E., Norgaard, J. C., and Ruchhoft, C. C. 1946. "Sewage Treatment at Military Installations, Report of the Subcommittee on Sewage Treatment of the Committee on Sanitary Engineering, National Research Council Division of Medical Science, Washington, DC," *Sewage Works Journal*, 18(5): 791.
- Wastewater Committee of the Great Lakes-Upper Mississippi River Board of State Public Health and Environment Managers. 1990. Recommended Standards for Wastewater Facilities, 1990 Edition. Health Education Services, Albany, NY.

Typical peaking factors are included in [Table 8.13](#). There is substantial variation in peaking factors between communities, especially between large and small communities, and designs should be based on local data.

Flow and Load Equalization

The design flows and load recommendations in [Table 8.13](#) generally do not account for hourly flow variations within a 24-h period. Normal daily peak flows and loads do not affect average activated sludge process performance unless the daily 3-h average maximum flow exceeds 160% of the 24-h average flow (140% of the average for the 6-h average maximum flow). If higher peak flows occur, they can be accounted for by increasing the size of the aeration tank. An approximate rule for the volume increase is (Joint Committee, 1977):

$$\Delta V = 60(PF_{3h} - 1.6) \quad (8.58)$$

where ΔV = the additional aeration tank volume needed to account for excessive peak flows (%)
 PF_{3h} = the peaking factor for the daily 3-h average maximum flow (decimal fraction)

For the daily 6-h average maximum flow, the rule is,

$$\Delta V = 60(PF_{6h} - 1.4) \quad (8.59)$$

where PF_{6h} = the peaking factor for the daily 6-h average maximum flow (decimal fraction).

Flow equalization is generally not recommended at municipal wastewater treatment plants unless the daily 3-h average maximum flow exceeds 210% of the average (180% of the average for the daily 6-h average maximum flow). The principal cause of performance degradation during peak hydraulic loads is secondary clarifier failure. In activated sludge plants, this seems to become a problem at overflow rates of about 1000 gpd/ft² (Ongerth, 1979).

Population Equivalents

The concentrations of various substances in raw sewage varies widely, because (1) water consumption rates vary widely, yielding differing degrees of dilution, and (2) potable waters start out with different amounts of minerals depending on local geology. However, the rates of contaminant mass generation per capita are fairly uniform, even across cultures. [Table 8.14](#) presents a summary of American and some European data.

The organic fraction is reported variously as 5-day biochemical oxygen demand (BOD), (dichromate) chemical oxygen demand (COD), volatile solids (VS), and total organic carbon (TOC). Approximate ratios among some of these parameters are given in [Table 8.15](#). The tabulated ratios are weighted toward the results of the U.S. EPA study (Burns and Roe, Inc., 1977), because of the large number of plants included.

TABLE 8.14 Population Equivalents for Municipal Sewage

Source	Volume (L/cap d)	TSS [VSS] (g/cap d)	CBOD ₅ (g/cap d)	Total N (g/cap d)	Total P (g/cap d)
Households ^{c,d,e,f}	220	38 [34]	45	15	4
Sanitary sewage, no industry ^b	—	—	54	—	—
Combined sewage, little industry ^b	—	—	73	—	—
Average of 31 cities ^a	400	95 [64]	100	14	—
Combined sewage, much industry ^b	—	—	150	—	—

^a Committee of the Sanitary Engineering Division on Sludge Digestion. 1937. "Report of Committee," *Proceedings of the American Society of Civil Engineers*,

^b Theriault, E. J. 1927. *The Oxygen Demand of Polluted Waters*, Bulletin No. 173. U.S. Public Health Service, Washington, DC.

^c Vollenweider, R. A. 1968. *Scientific Fundamentals of the Eutrophication of Lakes and Flowing Waters, with Particular Reference to Nitrogen and Phosphorus as Factors in Eutrophication*, OECD Report No. DAS/CSI/68.27. Organization for Economic Cooperation and Development, Directorate for Scientific Affairs, Paris, France.

^d Watson, K. S., Farrel, R. P., and Anderson, J. S. 1967. "The Contribution from the Individual Home to the Sewer System," *Journal of the Water Pollution Control Federation*, 39(12): 2039.

^e Webb, P., ed. 1964. *Bioastronautic Data Book*, NASA SP-3006. National Aeronautics and Space Administration, Scientific and Technical Information Division, Washington, DC.

^f Zaroni, A. E. and Rutkowski, R. J. 1972. "Per Capita Loadings of Domestic Wastewater," *Journal of the Water Pollution Control Federation*, 44(9): 1756.

TABLE 8.15 Relationships Among Organic Composition Indicators for Municipal Sewage

Parameter Ratio	Raw or Settled Sewage	Conventional,	
		Nonnitrified Effluent	Nitrified Effluent
BOD/COD	0.5	0.3	0.15
TOC/COD	0.4	0.4	—
COD/VSS	2.0?	1.4	1.4
Suspended BOD/VSS	—	0.5	—
Suspended BOD/VSS	—	0.4	—

Sources:

Austin, S., Yunt, F., and Wuerdeman, D. 1981. *Parallel Evaluation of Air- and Oxygen-Activated Sludge*, Order No. PB 81-246 712, National Technical Information Service, Springfield, VA.

Bishop, D. F., Heidman, J. A., and Stamberg, J. B. 1976. "Single-Stage Nitrification-Denitrification," *Journal of the Water Pollution Control Federation*, 48(3): 520.

Boon, A. G. and Burgess, D. R. 1972. "Effects of Diurnal Variations in Flow of Settled Sewage on the Performance of High-Rate Activated-Sludge Plants," *Water Pollution Control*, 71: 493.

Burns and Roe, Inc. 1977. *Federal Guidelines—State Pretreatment Programs—Vol. II—Appendixes 1–7*, EPA-430/9–76–017b, U.S. Environmental Protection Agency, Office of Water Program Operations, Municipal Construction Division, Washington, DC.

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Hoover, S. R. and Porges, N. 1952. "Assimilation of Dairy Wastes by Activated Sludge: II. The Equation of Synthesis and Rate of Oxygen Consumption," *Sewage and Industrial Wastes*, 24(3): 306.

Hunter, J. V. and Heukelekian, H. 1965. "The Composition of Domestic Sewage Fractions," *Journal of the Water Pollution Control Federation*, 37(8): 1142.

TABLE 8.15 (continued) Relationships Among Organic Composition Indicators for Municipal Sewage

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8.4 Intakes and Wells

Natural waters are almost never homogeneous, either in space or in time. Even aquifers often show some vertical and/or seasonal variation in water composition. There are, however, patterns to the variations, and a judicious selection of the exact point and timing of water withdrawals can simplify and improve treatment plant design, operation, and performance.

The factors requiring consideration by the designer of an intake or well field are as follows (Babbitt, Doland, and Cleasby, 1967; Burdick, 1930):

- Source of supply — e.g., river, lake, impounding reservoir, spring, or well
- Nature of the withdrawal site — e.g., depth of water, character of bottom (silt, gravel, cobbles, etc.), wind fetch and wave protection, flow stratification and seasonal turnover, salt fronts and intrusion, consolidated or unconsolidated aquifer materials, artesian or nonartesian aquifer
- Relationship of site to possible or actual sources of contamination
- Permanence of existing conditions — e.g., to what extent floods, droughts, other development, etc., might change the quantity and quality of the water, the site topography, or access to the water source
- Special requirements of the site — e.g., the need to provide special facilities to cope with moving sand bars, fish and wildlife, seasonal debris, ice, navigation, violent storms, high flood stages and low drought stages, accessibility of site, availability and reliability of electrical power, and distance to pumping station
- Site properties that affect the stability of the proposed structures — e.g., the character of the foundation soils

The source of supply, nature of the withdrawal site, special requirements of the site, and the relationship of the site to sources of contamination have direct impacts on the quality of the raw water and the treatment it requires. Each of these is discussed below.

River Intakes

Hazards

River intakes are exposed to a variety of hazards, including floating and suspended debris, fish and wildlife, ice, variable water stages due to floods and droughts, shipping, and pleasure boating. Although these hazards primarily affect the longevity and reliability of the structures involved, there are some indirect effects on water quality.

Debris and Wildlife

The principal concern raised by debris and wildlife is that they might enter the plant piping and damage mechanical equipment, such as pumps and valves. Small and fragile debris like fish, leaves, floating oils, cloth, and paper may pass through pumps and valves without damaging them. Nevertheless, such materials are objectionable, because they are macerated by the mechanical equipment and increase the amount of organic matter dissolved in the raw water. High concentrations of dissolved organic matter produce tastes, odors, colors, and turbidity; reduce the efficiency of disinfection; and promote the regrowth of microbes in the distribution system. This regrowth increases the threat of disease transmission and system corrosion.

The defense against debris and wildlife is screens. For intakes, these come in two general classes: bar racks and traveling screens. Screen design is discussed in Chapter 9.

The screenings collected by manual and/or mechanical cleaning undergo some compaction and decay during collection and storage, and they should not be returned to the raw water source. The debris has been concentrated from a large volume of water, and its physical, chemical, and biological properties may have been changed. The mere fact of concentration means that if the debris is returned to the raw

water source, a high local debris concentration will be created, which may be a nuisance. The changes in properties may cause the debris to settle in the water source, forming sludge deposits, or the dissolved organic matter concentration in the source may be increased. The preferred route of disposal is incineration or landfill.

Fish require special considerations, which are listed in Chapter 9.

Ice

A major hazard to all surface water intakes in the temperate zone is ice. The principal ice forms are surface, anchor, frazil, and slush (Baylis and Gerstein, 1948; Gisiger, 1947).

Frazil ice consists of fine needle-like ice crystals that form suspended in the water and later deposit on various surfaces. It sometimes accumulates in the water as slushy masses. Because the crystals are small and the slush is fragile, they can penetrate deeply into the intake piping system, and large amounts of ice may coat intake screens, the inside of conduits, and even pump impellers. This generally results in serious clogging of the intake system. Frazil ice occurs under turbulent conditions in a narrow temperature range, maybe only -0.05 to $+0.10^{\circ}\text{C}$. The condition is usually temporary, starting in the late evening to early morning and lasting until noon. Its formation is prevented by restricting the velocity through intake screens and conduits to less than 30 fpm and by mild heating of the flow to about 2°F above the ambient water temperature (Pankratz, 1988).

Anchor ice forms on submerged objects. This is especially troublesome in the guideways for slidegates. The usual preventive measure includes some sort of heating device, which may be as simple as an air bubble curtain or as involved as hot steam lines.

Surface ice damages structures by impact, static horizontal pressure, and static lifting. The last occurs when ice forms on the structure during low water stages, and the water levels subsequently rise. Drifting surface ice and ice floes may also enter the intakes, clogging them. In large lakes and rivers, ice floes may pile up against structures or be stranded in shallow water. At the Chicago intakes in Lake Michigan, stranded ice has been observed reaching from the Lake bottom some 37 ft below the water surface to 25 ft above the water surface (Burdick, 1946). Consequently, even bottom intakes are threatened. Because of these possibilities, intakes are usually designed to incorporate barriers and deflectors to keep ice away from the exposed structures.

Floods and Droughts

Floods threaten intakes by simple submergence, which may damage electric equipment and furnishings, and by structural damage due to hydraulic forces and the impacts of entrained debris, like logs and large rocks. Floods also move sand bars and may bury intakes or isolate them from the water flow. Droughts can lower water stages to the point that the intakes no longer contact the water. Very low water stages, if they happen fast enough, may destabilize river banks, causing the banks to slump and threaten intake foundations and screens.

Flood protection consists of barriers and deflectors like pilings, river training to stabilize the banks, and levees to protect buildings and equipment. Regulatory authorities may require that pumps, controls, and electrical switching equipment be installed above the expected maximum flood stage for some rare design flood. Drought protection generally involves the capability to withdraw water from several different elevations. This may be done by constructing multiple intakes or by constructing floating or moving intakes. The former are built on moored barges; the latter are built on rails that permit the pump suction bell to be moved up and down the bank as the river stage changes.

Floods and droughts exacerbate the problem of suspended debris, especially silts and sands. Floods resuspend stream bottoms and carry increased land erosion, and droughts often force water intakes to be placed so close to the bottom that the intake suction resuspends some sediments.

All of these problems require estimates of flood and drought stages. First, each time series (not the ranked values) is analyzed for trends and serial correlation. If none are found, the series may be fitted to some extreme value distribution, usually either Gumbel's distribution or the log Pearson Type III distribution.

Next, the maximum annual flood and minimum annual drought water surface elevations are ranked according to severity. For floods, this means highest ($m = 1$) to lowest ($m = n$), but for droughts, it means lowest ($m = 1$) to highest ($m = n$). Each elevation is then assigned an “exceedance” probability and a return period based on its rank:

$$P = \frac{m}{n+1} = \frac{1}{T} \quad (8.60)$$

where m = the rank of a flood or a drought beginning with most severe (highest stage for flood, lowest for drought) and ending with least severe (lowest stage for flood, highest for drought) (dimensionless)
 n = the number of records of maximum annual flood or minimum annual drought (dimensionless)
 P = the “exceedance” probability of either (1) the probability of a flood stage greater than or equal to the given elevation or (2) the probability of a drought stage less than or equal to the given elevation (per year)
 T = the return period of either (1) a flood stage equal to or larger than the given elevation or (2) a drought stage less than or equal to the given elevation (year)

For small data sets, the probabilities are sometimes assigned according to Eq. (8.61) (Taylor, 1990):

$$P = \frac{m-0.5}{n} = \frac{1}{T} \quad (8.61)$$

The exceedance probability is the probability that a flood or drought equal to or worse than the given event will occur in any randomly chosen year. The return period is the average length of time between such occurrences.

Each elevation can also be assigned a nonexceedance probability and an associated return period:

$$P + P_o = 1 \quad (8.62)$$

$$\frac{1}{T} + \frac{1}{T_o} = 1 \quad (8.63)$$

where P_o = the “nonexceedance” probability — either (1) the probability of a flood stage elevation less than the given elevation or (2) the probability of a drought stage greater than the given elevation (per year)
 T_o = the return period of either (1) a flood stage less than the given elevation or (2) a drought stage greater than the given elevation (year)

If the Gumbel equation gives an adequate fit (a plot of the flows vs. the equivalent flood return period on Gumbel probability paper should be linear), the design flood and drought stages for some design return period, T , may be calculated from Eqs. (8.58) and (8.59), respectively:

$$z_F(T) = \bar{z}_F - s_F \frac{\sqrt{6}}{\pi} [\gamma + \ln(\ln T_o)] \quad (8.64)$$

$$z_D(T) = \bar{z}_D - s_D \frac{\sqrt{6}}{\pi} [\gamma + \ln(\ln T)] \quad (8.65)$$

where s_D = the standard deviation of the observed minimum annual drought stages (m)
 s_F = the standard deviation of the observed maximum annual flood stages (m)
 \bar{z}_D = the arithmetic average of the observed minimum annual drought stages (m)

- $z_D(T)$ = the estimated drought stage elevation with a return period of T years (m)
- \bar{z}_F = the arithmetic average of the observed maximum annual flood stages (m)
- $z_F(T)$ = the estimated flood stage elevation with a return period of T years (m)
- $\gamma = 0.57722\dots$, Euler's constant

Although the Gumbel distribution often provides a satisfactory fit, other distributions may be needed in other cases. A collection of these may be found in Chow (1964). The U.S. Water Resources Council adopted the log Pearson Type III distribution for flood analyses on federally funded projects (Hydrology Committee, 1977).

Navigation

The principal hazards due to navigation are collisions between boats and the intake structure, anchors snagging parts of the intake, and contaminants discharged accidentally or deliberately. The recommended defense against these hazards is avoidance of navigation channels. However, this conflicts with the need for access to deep water during droughts and cannot always be achieved. Once again, various kinds of barriers and deflectors are constructed, and intake depths are selected to avoid water contaminated by waste discharges from boats. This normally means selection of deeper waters.

In the U.S., all navigable waters are under the jurisdiction of the U.S. Army Corps of Engineers, and construction within navigable waters requires a Corps-issued permit.

Contaminant Distribution

Rivers are frequently stratified horizontally and vertically. This is important for two reasons:

- First, water quality in rivers can be expected to vary significantly with the distance from the bank. Most importantly, contaminants will tend to cling to the bank from which they are discharged, so that water intakes are best placed on the opposite bank.
- Second, contaminant dilution does not always or even usually occur. What is needed is a method to predict when and where contaminants will occur. It is especially desirable to locate zones of low contamination and to predict when known spills will arrive at the intake.

In order to predict where contaminants are at any time, it is necessary to consider how they are transported by the water source. There are four mechanisms that need to be accounted for: advection (convection), shear flow dispersion, turbulent diffusion, and molecular (Fickian) diffusion. Advection is the transport achieved by the average motion of the whole body of water. In the case of rivers, this is the transport due to the mean velocity. Shear flow dispersion is caused by the vertical and horizontal variations in velocity over the cross section. If velocity measurements are taken at small time intervals at a fixed point in the cross section, it will be discovered that the velocity there fluctuates. These short period fluctuations about the predicted velocity distribution give rise to another transport mode — turbulent diffusion. Finally, there is molecular diffusion, which arises because of the random thermal motions of contaminant molecules.

These four transport mechanisms are additive. However, one or more of them may sometimes be ignored. For example, by definition, there is no advection or shear flow dispersion vertically or laterally in a river; transport in these directions is due entirely to turbulent diffusion and molecular diffusion. In river mouths and in estuaries and lakes, the mean velocity is often small, and advection is negligible, even in the “downstream” direction. In these cases, only shear flow dispersion, turbulent diffusion, and molecular diffusion are active. But, in the main stem of most rivers, only advection need be considered.

Longitudinal Transport

Longitudinal transport in rivers is dominated by advection. Shear flow dispersion is a minor component, but it becomes important during unsteady phenomena like spills. Turbulent diffusion and molecular diffusion are negligible contributors to longitudinal transport.

Consider a spill of a conservation contaminant some distance upstream of an intake. If the flow is uniform and steady, the predicted arealy-weighted cross-sectional average contaminant concentration at the intake is (Crank, 1975; Fischer et al., 1979),

$$\bar{C}_A(x,t) = \frac{M/A}{\sqrt{4\pi Kt}} \cdot \exp\left[-\frac{(x-Ut)^2}{4Kt}\right] \quad (8.66)$$

where A = the cross-sectional area of the stream (m^2)
 $\bar{C}_A(x,t)$ = the cross-sectional average contaminant concentration at x downstream from the apparent origin at time t after the spill (kg/m^3)
 K = the longitudinal shear flow diffusivity (m^2/s)
 M = the mass of the contaminant spilled (kg)
 t = the time elapsed since the spill (s)
 U = the (uniform flow) areally-weighted, cross-sectional average stream velocity between the spill and the intake (m/s)
 x = the effective distance the spill has traveled from its apparent origin (m)

A commonly used semiempirical formula for K is (Fischer et al., 1979),

$$K = \frac{0.011U^2W^2}{Hv_*} \quad (8.67)$$

where g = the acceleration due to gravity (9.806650 m/s^2)
 H = the mean depth (m)
 R = the hydraulic radius (m)
 S = the energy gradient (m/m)
 v_* = the shear velocity (m/s)
 $= \sqrt{gRS}$
 W = the stream (top) width (m)

The minimum distance downstream of a spill or an outfall that the water must travel before the contaminant has spread sufficiently for Eq. (8.66) to apply is called the “initial period” and is given approximately by (Fischer et al., 1979),

$$L_{ip} = \frac{0.67UW^2}{Hv_*} \quad (8.68)$$

where L_{ip} = the initial period (m).

The error in Eq. (8.62) is on the order of 50%. At L_{ip} from the spill, the contaminant cloud looks as if it had traveled a shorter distance.

The apparent origin of the spill is L_{ao} downstream from its actual location (Fischer et al., 1979):

$$L_{ao} = \frac{0.12UW^2}{Hv_*} \quad (8.69)$$

where L_{ao} = the distance from the actual spill to its apparent origin (m). This equation also has an error of about 50%.

Lateral Distribution

If a spill occurs within L_{ip} of an intake, the effect of lateral spreading of the contaminant will be important. The transport problem is now two-dimensional, and the important processes are advection and turbulent diffusion. Vertical mixing can be ignored, because streams are very much wider than they are deep, and the vertical velocity gradients are steeper than the horizontal velocity gradients, so they become vertically mixed long before they are laterally mixed. For steady, uniform flow with a uniform velocity profile and

isotropic turbulence, the longitudinal and lateral contaminant concentrations due to a spill can be estimated from,

$$\begin{aligned} \bar{C}_z(x, y, t) = & \frac{M/H}{4\pi t \sqrt{KE}} \cdot \exp\left[-\frac{(x-Ut)^2}{4Kt}\right] \times \dots \\ & \dots \times \sum_{j=-\infty}^{\infty} \left\{ \exp\left[-\frac{(y-2jW-y_o)^2}{4Et}\right] + \exp\left[-\frac{(y-2jW+y_o)^2}{4Et}\right] \right\} \end{aligned} \quad (8.70)$$

where \bar{C} = the depth-averaged contaminant concentration at x downstream from the apparent origin, y from the right bank, and t after the spill (kg/m^3)
 E = the turbulent diffusivity (m^2/s)
 H = the mean depth (m)
 M = the mass of the contaminant spilled (kg)
 t = the time elapsed since the spill (s)
 U = the (uniform flow) arealy-weighted, cross-sectional average stream velocity between the spill and the intake (m/s)
 x = the effective distance the spill has traveled from its apparent origin (m)
 y = the distance across a stream measured from the right bank (m)
 y_o = the location of a spill measured from the right bank (m)

The error in this equation is approximately 50%. As a practical matter, we are interested in peak concentrations, which occur when $x = Ut$; the first exponential term is unity. As for the summation, only the few terms centered on $j = 0$ need to be computed.

A commonly used estimate of E is (Fischer et al., 1979),

$$E = 0.6Hv_* \quad (8.71)$$

If the source of the contaminant is a steady discharge rather than a spill, then the steady state, depth-averaged prediction is (Fischer et al., 1979),

$$\begin{aligned} \bar{C}_{z,t}(x, y) = & \frac{W\bar{Q}_w\bar{C}_{QW}}{\sqrt{4\pi E\tau}} \left/ \left(\bar{Q}_R + \bar{Q}_w \right) \right. \times \dots \\ & \dots \times \sum_{j=-\infty}^{\infty} \left\{ \exp\left[-\frac{(y-2jW-y_o)^2}{4E\tau}\right] + \exp\left[-\frac{(y-2jW+y_o)^2}{4E\tau}\right] \right\} \end{aligned} \quad (8.72)$$

where \bar{C}_{QR} = the flow-weighted (flow-composited) contaminant concentration in the river upstream of the outfall (kg/m^3)
 \bar{C}_{QW} = the flow-weighted (flow-composited) contaminant concentration in the wastewater (kg/m^3)
 $\bar{C}_{z,t}(x, y)$ = the depth- and time-averaged concentration at distance x from the apparent origin and y from the right bank (kg/m^3)
 \bar{Q}_R = the time-averaged (steady state) river flow for the critical period, generally the 7-day-average low flow with a 10-year return period (m^3/s)
 \bar{Q}_W = the time-averaged (steady state) wastewater flow for the critical period, generally the maximum 7-day or 30-day average wastewater flow rate (m^3/s)
 τ = the time-of-travel from the apparent origin (s)

Note that the time-of-travel from the source, τ , replaces the elapsed time, t .

Adsorbed Contaminants

Because streams mix vertically more quickly than they mix laterally, it is usually assumed that contaminant concentrations are uniform with respect to depth. There are, however, important exceptions to this rule: settleable solids — especially sand, silt, and clay — and floatable solids like grease.

Sands, silts, and clays, are important for two reasons. First, they are contaminants, causing turbidity. However, this is not a major issue. Sands, silts, and clays are quite reactive, and they adsorb a large number of other contaminants, particularly pesticides, volatile organic substances, and heavy metals. Consequently, the presence of sand, silt, and clay in a finished water raises the possibility that other, more serious contaminants are present. There is one advantage in all this: they are easily removed and so, consequently, are the poisons adsorbed to them.

The amount of contaminant sorbed onto silt and clay particles can be expressed in terms of a partition coefficient (Thomann and Mueller, 1987):

$$\Pi = \frac{C_p}{C_d} \quad (8.73)$$

where C_d = the dissolved contaminant concentration based on total volume of liquid and solids (kg/m³)
 C_p = the concentration of contaminant adsorbed to suspended solids (kg contaminant/kg clay)
 Π = the partition coefficient (m³/kg)

The partition coefficient may be used to calculate the fraction of the contaminant that is dissolved or adsorbed (Thomann and Mueller, 1987):

$$\frac{C_d}{C_t} = \frac{1}{1 + \Pi C_c} \quad (8.74)$$

$$\frac{C_p}{C_t} = \frac{\Pi C_c}{1 + \Pi C_c} \quad (8.75)$$

where C_t = the total concentration of contaminant based on the total volume of liquid and solids (kg/m³)
 $= C_d + C_p C_c$

If there is no resuspension of sediment, then a simple steady, uniform flow model for contaminant loss due to sedimentation of silts and clays is,

$$C_t(x) = C_{t0} \frac{1 + \Pi C_{c0} \exp\left(-\frac{v_s x}{HU}\right)}{1 + \Pi C_{c0}} \quad (8.76)$$

where C_{c0} = the initial silt and clay concentration at a point source of sorbable contaminants (kg/m³)
 C_{t0} = the initial total concentration of contaminant at a point source (kg/m³)
 $C_t(x)$ = the total concentration of contaminant at distance x from a point source (kg/m³)
 v_s = the settling velocity of the sediment (m/s)
 x = the distance from the point source (m)

Lake and Reservoir Intakes

Hazards

All of the hazards that threaten river intakes also threaten lake intakes: namely, debris and wildlife, ice, floods and droughts, and shipping. There are, of course, differences. Ice and debris in rivers are often

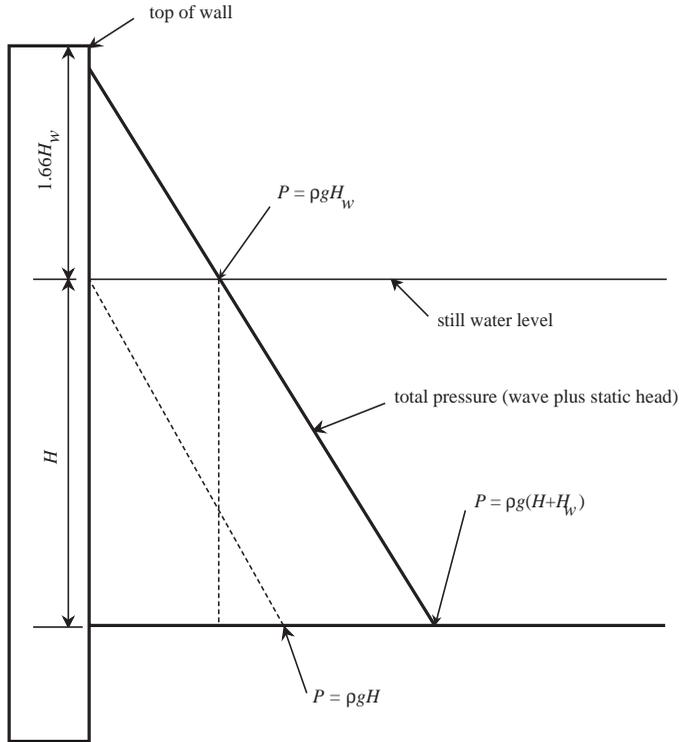


FIGURE 8.4 Reflected wave pressure on vertical wall (Minikin, 1950).

carried by strong currents that may produce strong impacts. In lakes, the currents and impacts are less severe. Rivers in flood can move sizeable boulders, but the debris in lakes is limited to floatable objects. Floods and droughts in rivers are generally short-term phenomena, driven by recent weather and lasting a few days to a season or two, whereas floods and droughts in lakes often last for years and are driven by long-cycle weather patterns. There are, however, two hazards that are peculiar to lakes: surface waves and seiches. Both are wind-driven phenomena.

Surface Waves

Waves are formed initially by fluctuations in the wind stress on the water surface, but once formed, they interact with each other and the wind to form a large variety of wave sizes. Wave power is transmitted to structures when the waves (1) are reflected by the structure, (2) break on the structure, or (3), if it is submerged, pass over the structure.

For reflected, nonbreaking waves, the simplest analysis is Minikin's (1950). If a wave is reflected by a vertical wall, it is supposed to rise up the wall above the still water level a distance equal to 1.66 times the wave height. This produces a momentary pressure gradient like that shown in Fig. 8.4, with a wave pressure at the still water elevation of $\rho g H_w$. The total horizontal force due to the wave of,

$$F = \underbrace{\rho g W H H_w}_{\text{wave force below still water depth}} + \underbrace{\rho g W \frac{H_w^2}{2}}_{\text{wave force above still water depth}} \quad (8.77)$$

where F = the total wave force (N)
 g = the acceleration due to gravity (9.80665 m/s²)
 H = the still water depth (m)
 H_w = the wave height measured crest to trough (m)

W = the width of the wall (m)
 ρ = the density of water (kg/m³)

and a wave moment of

$$M = \rho g W \left[\frac{1.66 H_w^2}{2} \left(H + \frac{1.66 H_w}{3} \right) + \frac{H^2 H_w}{2} \right] \quad (8.78)$$

where M = the turning moment (Nm).

These estimates should be doubled for ratios of water depth to wave length less than about 0.20 (King and Brater, 1963). The forces and moments due to the still water level should be added to the wave forces and moments.

Breaking waves would develop horizontal forces on the order of (King and Brater, 1963),

$$F = 87.8 W H_w^2 \quad (8.79)$$

where F = the horizontal wave force (lbf)
 H_w = the wave height (ft)
 W = the width of the structure (ft)

Note that this is an empirical equation written in customary American units. The force would be centered at the still water level and be distributed approximately $\frac{1}{2} H_w$ above and below the still water level.

Forces on submerged structures depend upon the depth of submergence and the dimensions of the structure relative to the wave length of the waves. Horizontal and vertical forces can be developed.

In many locales, wave heights are routinely recorded, and the needed information can be obtained from the records. Designs will generally be based on some rare, large wave, and it will be necessary to interpolate within or extrapolate from the recorded waves to get the design wave height, H_{wd} . Interpolation/extrapolation is normally done using the Rayleigh distribution (Coastal Engineering Research Center, 1984):

$$P = \text{Prob}(H_w \geq H_{wd}) = \exp\left(-\frac{H_{wd}^2}{H_{rms}^2}\right) \quad (8.80)$$

$$H_{rms} = \sqrt{\frac{\sum_{i=1}^n H_w^2}{n}} \quad (8.81)$$

where H_{wd} = the design wave height (m) and H_{rms} = the root-mean-square wave height (m).

A semilog plot of the exceedance vs. the square of the wave heights is a straight line, so linear regression techniques may be used to make the interpolation/extrapolation unbiased and objective. The wave heights are rank ordered from largest to smallest, and the exceedance probability is calculated for each recorded height using $m/(n + 1)$, where m is the rank of the recorded height.

If wave records are unavailable, then wave heights are estimated using the dimensions of the lake and wind speed records. The simplest procedure uses an empirical equation for the highest waves given by Stevenson (Hutchinson, 1975):

$$H_w = 0.105 \sqrt{L_f} \quad (8.82)$$

where H_w = the highest wave height (cm)
 L_f = the fetch, i.e., the distance across the lake measured from the point of interest upwind (cm)

The wind is supposed to be steady over the fetch length, and it is supposed to last long enough to fully develop the waves. In large lakes, this may not be the case. Methods for handling these cases and others are described in the *Shore Protection Manual* (Coastal Engineering Research Center, 1984).

Seiches

If the wind blows long enough, water will tend to pile up on the lee side of the lake, and the entire lake surface will be tilted upwards downwind. This phenomenon is called a “seiche,” and the difference in water surface elevations from one end of the lake to the other is called the “set-up.”

An estimate for the set-up can be derived from a simple, steady state momentum balance on an arbitrary control volume (Hutchinson, 1975):

$$H_{su} = \frac{\tau_w L}{\rho g H} \quad (8.83)$$

As a first-order approximation, the distance L may be identified as the length of the lake, H as its mean depth, and H_{su} its setup. This implies that the sloping lake surface is planar; it is usually curved. The effects of varying lake depth and thermal stratification have also been ignored, but their incorporation requires a complete two- or three-dimensional hydrodynamic model, and what is wanted here is an order of magnitude estimate of the setup.

The wind shear stress may be estimated by a variety of semiempirical formulas, but a common estimate is (Hutchinson, 1975):

$$\tau_w = 3.5 \times 10^{-6} v_{10}^2 \quad (8.84)$$

where v_{10} = the wind velocity 10 m above the surface (cm/s)
 τ_w = the wind shear stress (dynes/cm²)

Note that Eq. (8.84) gives the stress in cgs units, so if it is used in Eq. (8.83), cgs units must be used there, too.

Another empirical equation for the set-up is (King and Brater, 1963):

$$\frac{H_{su}}{H} = 4.88 \times 10^{-5} \left(\frac{L_f}{H} \right)^{1.66} \left(\frac{v_{10}^2}{gL_f} \right)^{2.20(L_f/H)^{-0.0768}} \quad (8.85)$$

Note that this equation is written in dimensionless ratios.

Lake Stratification

During the summer, temperate zone lakes tend to be stratified vertically. The stratified lake may be divided into three zones: an epilimnion, a mesolimnion (alias thermocline and metalimnion), and a hypolimnion. The epilimnion is the top portion of the lake. It is generally well mixed and somewhat turbulent, because of the wind. It is also warm. The bottom part of the lake is the hypolimnion. It, too, is homogeneous, but it is poorly mixed and nonturbulent or even laminar. It is cool. The mesolimnion is the transition zone between the other layers. It is normally defined as the region in which the temperature changes.

The stratification is due to the density differences between the warm, light surface water and the cold, heavy bottom water. It develops as follows: In the early spring, the lake is still cold from the previous winter, and it is well mixed from top to bottom, because of the seasonally strong winds. As the spring and early summer progress, solar heating increases, and the winds die down. The heating tends to produce a warm surface layer that is lighter than the cold, deep waters. If the winds are strong enough, they will mix the warm and cold waters, and the whole lake will warm uniformly. However, at some point, the increasing heating rate will overcome the decreasing wind shear, and the warm surface waters will not be mixed down into the whole lake. The lake is then stratified. The epilimnion will continue to warm during the summer, but the hypolimnion will remain cool.

The stratification breaks down in the fall as solar heating declines and the winds pick up. This generally results in complete mixing of the lake. If the lake surface does not freeze during the winter, the mixing will continue until the next spring, when the lake again stratifies. Such lakes are called “monomictic,” meaning one mixing per year. If the lake surface freezes, wind mixing is impossible, and the lake becomes stagnant. It may exhibit an inverse stratification with near 0°C water just under the ice and denser 4°C water below. Such lakes are called “dimictic,” meaning two mixings per year.

Many variations are possible. First, lakes might not stratify. High-latitude and high-altitude lakes may cycle between frozen, stagnant periods and well-mixed periods, and equatorial lakes may be mixed continuously throughout the year. Second, in temperate lakes, local conditions may result in weak stratification that comes and goes during the summer, or the lake may stratify into more than three zones. Third, deep bottom waters may remain permanently segregated from the rest of the lake, not participating in the general mixing. Such lakes are called “meromictic.” The bottom waters of these lakes are always significantly denser than the remainder of the lake, especially the hypolimnion, because they contain elevated concentrations of dissolved solids.

Intake Location

Because lakes and reservoirs stratify, the depth of intakes requires special consideration. The temperature difference between the epilimnion and hypolimnion is usually large enough to affect the kinetics and hydraulics of the various treatment processes, and tank volumes, chemical dosages, and power consumption will depend on whether upper or lower waters are withdrawn. Particularly in the cases of tastes and odors and turbidity, there may be major differences between the epilimnion and hypolimnion, and these differences may dictate the kinds of treatment processes used, not merely their sizing. Many of the characteristics of the epilimnion and hypolimnion change during the year, and a single depth of withdrawal will not always be preferred. Consequently, the intakes should be designed with multiple inlet ports at various depths.

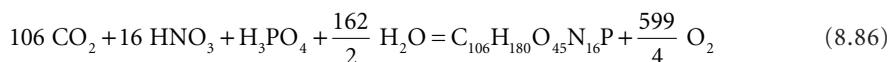
The effects of seiches also deserve consideration. When strong winds set-up a lake, it is the epilimnion waters that pile up on the lee end. This means that at the lee end, the epilimnion becomes deeper, and the hypolimnion is compressed and becomes shallower. In fact, the epilimnion may become deep enough that the hypolimnion is completely displaced. The opposite occurs at the windward end of the lake: the epilimnion may be completely blown away, and the hypolimnion may surface. The result of these effects is that the plant operator may not be able to control the kinds of water withdrawn from the lake, and the plant must be designed to accommodate any raw water.

Stratification also affects the fate of contaminant discharges and spills. These inputs will tend to seek water of their own density rather than simply disperse any that may be found at any elevation in the lake. The elevation may vary seasonally as the input’s density varies and the lake temperatures change.

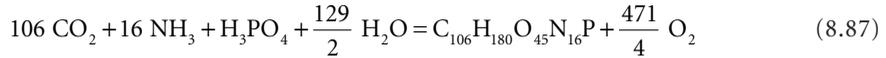
Algal Growth in the Epilimnion

The principal water quality problems associated with stratification are (1) the presence of algae and their metabolic products in the epilimnion and (2) reduced oxygen concentrations and algal decay products in the hypolimnion. These problems are connected.

Algae grow by absorbing nutrients and water and using light energy to convert them to more algae. Except for diatoms, the synthesis stoichiometry for algae growing on nitrate and phosphorus is approximately as follows (Jewell, 1968):



The algae have a formula weight of 2428, and they are, on a dry weight basis, 52% carbon, 30% oxygen, 9.2% nitrogen, 7.4% hydrogen, and 1.3% phosphorus. Diatoms have an exterior shell of silica, which comprises about one-fourth of the dry weight. However, the material inside the shell has the composition given above. If the synthesis begins from ammonia rather than nitrate, the stoichiometry is:



Note that the formation of 1 g of algae results in the release of either 1.97 g of oxygen or 1.55 g, depending on whether the synthesis starts with nitrate or ammonia, respectively. Conversely, when the algae decay, the same amount of oxygen is consumed. Algae grow in the warm, illuminated epilimnion and settle into the cool, dark hypolimnion. This leads to the fundamental problem of lakes: oxygen release occurs in the epilimnion, whereas oxygen consumption occurs in the hypolimnion, and because the two do not mix, the hypolimnion can become anoxic.

The effects of algae may be estimated by a somewhat simplified analysis of the spring bloom. The algae growing in the epilimnion may be either nutrient-limited or light-limited. They are most likely nutrient limited, if the epilimnion is clear and shallow. In this case, the algae grow until some nutrient is exhausted. Supposing that nutrient is phosphorus, the maximum possible algal concentration is:

$$X_{\max} = Y_p P_o \quad (8.88)$$

where P_o = the initial available orthophosphate concentration (kg/m³)

X_{\max} = the maximum possible algal concentration in the epilimnion due to nutrient limitation (kg/m³)

Y_p = the algal true growth yield coefficient on phosphorus (kg algae/kg P)

Other nutrients can be limiting. Diatoms are often limited by silica.

Limnologists prefer to work in terms of the areal concentration rather than the volumetric concentration. This can be calculated by multiplying X_{\max} by the volume of the epilimnion and dividing it by the surface area of the lake:

$$\frac{X_{\max} V_e}{A} = Y_p P_o H_e \quad (8.89)$$

where A = the epilimnion plan area (m²)

H_e = the epilimnion depth (m)

V_e = the epilimnion volume (m³)

Note that the areal concentration is simply proportional to the epilimnion depth and the initial phosphorus concentration. Of course, if nitrogen was the limiting nutrient, the slope would depend on nitrogen rather than phosphorus.

The light-limited case is more complicated, because the maximum algal concentration is the result of a balancing of rates rather than simple nutrient exhaustion. The simplest mass balance for the algae in the epilimnion is:

$$\underbrace{V_e \frac{dX}{dt}}_{\text{rate of algal accumulation}} = \underbrace{\mu X V_e}_{\text{rate of algal growth}} - \underbrace{k_d X V_e}_{\text{rate of algal loss}} \quad (8.90)$$

where k_d = the algal decay rate (per sec)

t = elapsed time (sec)

X = the algal concentration (kg/m³)

μ = the algal specific growth rate (per sec)

In this simplified model, the specific growth rate is supposed to depend only on light intensity and temperature, and the decay rate is supposed to depend only on temperature.

The dependency of the specific growth rate on light can be represented by Steele's (1965) equation:

$$\mu(z) = \mu_{\max} \frac{I(z)}{I_{\text{sat}}} \exp\left(1 - \frac{I(z)}{I_{\text{sat}}}\right) \quad (8.91)$$

where I_{sat} = the saturation light intensity for the algae (W/m^2)
 $I(z)$ = the light intensity at depth z from the surface (W/m^2)
 μ_{\max} = the maximum algal specific growth rate (per sec)
 $\mu(z)$ = the algal specific growth rate at depth z (per sec)
 z = the depth from the surface (m or ft)

The light intensity declines exponentially with depth according to the Beer-Lambert Law:

$$I(z) = I_o \exp[-(a + bX)z] \quad (8.92)$$

where a = the water extinction coefficient (per m)
 b = the algal extinction coefficient (m^2/kg)
 I_o = the surface light intensity (W/m^2)

Substituting Eq. (8.92) into Eq. (8.95) and averaging over depth and time yields (Thomann and Mueller, 1987):

$$\bar{\mu}_{zt} = \frac{ef\mu_{\max}}{(a + bX)H_e} \left\{ \exp\left[-\frac{\bar{I}_o}{I_{\text{sat}}} \exp(-[a + bX]H_e)\right] - \exp\left(-\frac{\bar{I}_o}{I_{\text{sat}}}\right) \right\} \quad (8.93)$$

where $e = 2.7183\dots$, the base of the natural logarithms
 f = the fraction of the 24-h day that is sunlit (dimensionless)
 \bar{I}_o = the average sunlight intensity during daylight (W/m^2)
 $\bar{\mu}_{zt}$ = the depth- and time-averaged algal specific growth rate (per sec)

During the summer, the average surface light intensity is usually much larger than the saturation light intensity, and the exponential of their ratio is nearly zero. Also, the light intensity at the bottom of the epilimnion will be close to zero, because the algal population will increase until nearly all the light has been absorbed. The exponential of zero is one, so Eq. (8.93) reduces to (Lorenzen and Mitchell, 1973; 1975):

$$\bar{\mu}_{zt} \cong \frac{2.718f\mu_{\max}}{(a + bX)H_e} \quad (8.94)$$

The algal growth rate declines as its population increases (which is called *self-shading*), and at some point, it just equals the decay rate. No further population increase is possible, so this becomes the maximum algal concentration. Substituting Eq. (8.94) into (8.90) and solving for the steady state algal concentration yields:

$$X_{\max} H_e = \frac{2.718f\mu_{\max}}{bk_d} - \frac{a}{b} H_e \quad (8.95)$$

Diehl (2002) and Diehl et al. (2002) have recently published the theory and supporting field data for a more detailed model that adds algal sinking and nutrient concentration gradients to the Lorenzen-Mitchell analysis.

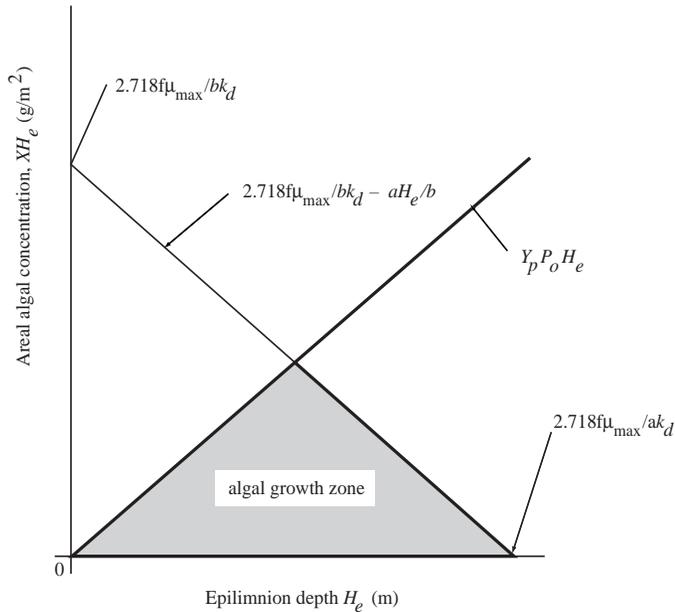


FIGURE 8.5 Algal epilimnetic growth zone.

Equation (8.95) predicts that the maximum areal algal concentration under light-limited conditions will be a linear function of epilimnion depth with a negative slope. Under nutrient-limited conditions, the maximum areal concentration is also linear, but it has a positive slope. These two lines form the boundaries to a triangular region, shown in Fig. 8.5, that contains all possible areal algal concentrations. Any given lake will have some specific epilimnetic depth. The maximum algal concentration that can occur in that lake can be determined by projecting a vertical line from the depth axis to the first line intersected. That line will give the concentration. If the epilimnetic depth is so deep that it lies outside the triangular region, no algae will be found in the lake. In fact, it is the reduction in mixed depth in the spring due to stratification that permits algal blooms to occur.

Algal Death in the Hypolimnion

As a first-order approximation, it may be assumed that the quantity of algae that settles into the hypolimnion is equal to the amount of algae that can be formed from the limiting nutrient in the epilimnion. This will be true even in light-limited lakes, because the algal concentration given by Eq. (8.89) is a dynamic equilibrium, and algae will be continuously formed and removed at that concentration until the limiting nutrient is exhausted. Algal decay results in oxygen consumption, so the oxygen balance in the hypolimnion is:

$$\beta_X X_{\max} V_e = (C_o - C) V_h \quad (8.96)$$

where C = the final oxygen concentration in the hypolimnion (kg/m^3)
 C_o = the initial oxygen concentration in the hypolimnion (kg/m^3)
 V_h = the volume of the hypolimnion (m^3)
 β_X = the algal oxygen demand ($\text{kg O}_2/\text{kg algae}$)

When algae decay, their nitrogen content is first released as ammonia, and the ammonia is subsequently converted to nitrate. This means that the algal “carbonaceous” oxygen demand of $1.55 \text{ g O}_2/\text{g algae}$ is satisfied first, and it proceeds as long as C is greater than zero. The ammonia released during this process is oxidized only if oxygen is left over from the first step.

Algal Control

Figure 8.5 and Eq. (8.96) provide the means of estimating the impacts of algae on lakes and reservoirs and clues to the control of excessive algal blooms:

- The initial nutrient concentration may be reduced by changes in land use and by restrictions on used water discharges.
- The algal decay rate could be increased by adding some sort of poison.
- The lake may be artificially mixed to increase the epilimnetic depth, thereby shifting the algal concentration rightwards and downwards along the light-limited line.

Restriction of nutrient input is the method of choice, but the underlying assumption is that the algae are nutrient-limited to begin with or that inputs can be reduced to the point that they become nutrient-limited. If the algae are or remain light-limited, control of nutrient inputs will not affect the concentration of algae in the epilimnion. The effect of nutrient restrictions is to reduce the slope of the nutrient line in Fig. 8.5, so if the algae are nutrient-limited, the reduction in algae is proportional to the reduction in nutrient input. Nutrient reductions will always reduce the amount of oxygen consumption in the hypolimnion and the amount of algal excretion product in the epilimnion. Nutrient input reductions are relatively easily attained when the inputs come from point sources like sewage treatment plants, but reductions are difficult to come by when the sources are diffuse like agriculture, forests, or lawns.

Poisons like copper sulfate [$\text{CuSO}_4 \cdot 5\text{H}_2\text{O}$], calcium hypochlorite [$\text{Ca}(\text{OCl})_2$], and various organic algicides act by increasing the algal death rate. This has the effect of reducing the vertical intercept of Eq. (8.95) and lowering the whole light-limited line. Normal dosages are a few tenths of a mg/L of either copper sulfate or calcium hypochlorite, but the dosages are species-specific and vary substantially depending on the kinds of algae present (Hale, 1957). For reasons of economy, the epilimnion should be small. The poison dosages needed to kill algae are often fatal to other wildlife, and this may prevent their use.

The reduction of epilimnetic algal concentrations by artificial mixing of lakes and reservoirs to control algae has been achieved many times (Cooley and Harris, 1954; Fast, 1971; Hogan et al., 1970; Hooper et al., 1952; Leach and Harlin, 1970; Lomax and Orsborn, 1970; Lorenzen and Fast, 1977; Lorenzen and Mitchell, 1975; Riddick, 1958; Symons, 1969; Torrest and Wen, 1976; Zieminski and Whittemore, 1970). Furthermore, cyanobacteria (formerly *blue-green algae*), which are especially objectionable, are more susceptible to control by mixing than are other kinds of algae. To be effective, the lake must be deep enough that the algae either are or will become light-limited. Referring to Fig. 8.5, it can be seen that increasing the depth of shallow, nutrient-rich systems will only increase the number of algae present, and this effect also has been observed (Hooper, Ball, and Tanner, 1952; Symons, 1969).

The usual method of mixing is air diffusion. This is preferred because many hypolimnetic waters are oxygen depleted, and mixing low-oxygen water into the epilimnion may kill or injure wildlife and may contribute to taste and odor problems. The amount of air required for satisfactory mixing is dependent on the total work required to mix the lake, the mixing work supplied by the wind, and the offsetting heat input from the sun. The resulting air requirement is strongly dependent on local conditions, but a rough and ready rule of thumb is 20 scfm per million square feet of lake surface (Lorenzen and Fast, 1977).

Littoral vs. Pelagic Zones

Lakes also stratify horizontally and may be divided into a "littoral" or near-shore zone and a "pelagic" or central zone. Contaminant levels are generally higher in the littoral than in the pelagic zone. The kinds of contaminants present depend largely on lake size and the intensity of currents and waves that develop in the lake as a consequence of its size.

Small lakes have weak currents and little or no waves, except during storms. The littoral zone in these lakes is usually defined to be the zone of rooted aquatic plants, and it is subdivided into zones of (a) emergent plants, (b) floating plants, and (c) submerged plants (Wetzel, 1975). The emergent plants are generally limited to water depths less than about 1.5 m, the floating plants can exist in depths between 0.5 and 3.0 m, and the submerged plants can maintain themselves at virtually any depth that has sufficient sunlight.

Conditions in the small lake littoral zone are good for plant growth. Land drainage, with its generally high nutrient load, must first pass through the littoral zone, and the weak currents there insure that much of the nutrient load is retained. The littoral zone is shallow and well illuminated, and the rooted plants provide a large surface area for the attachment and growth of microscopic algae. The net result is that plant productivity in the littoral zone is very high and may, in some cases, comprise the bulk of the net photosynthesis in the whole lake (Wetzel, 1975). The net result of all of this activity is a water that is high in turbidity, color, taste, and odor.

The littoral zone of large lakes is defined to be the region between the shoreline and the breakers (Coastal Engineering Research Center, 1984). Within this zone, there are few aquatic plants, the bottom is coarse grained, and the water has a high turbidity because of suspended silt and sand. There is almost always a *littoral drift*, i.e., a current parallel to the shore and tending in the same direction as the mean wind. Somewhat farther offshore, there may be a *coastal jet*, i.e., a fairly strong current parallel to the coast (Csanady, 1975).

Water quality in littoral zones is generally not suitable for water supplies, and better quality water may be had farther offshore. The likelihood of contamination from spills on land is also greatly reduced, because they are intercepted by the drift and coastal jet. Placement of the intake in the open water will also provide the opportunity to choose between epilimnetic and hypolimnetic waters, although there are more hazards due to boating and shipping.

Wells

Aquifer Choice

Groundwaters occur in a variety of geological strata, including unconsolidated materials, porous rocks, fractured rocks, and limestones with solution channels. Any of these formations can be developed into an acceptable source, as long as:

- The water can be brought up to acceptable quality standards by treatment.
- The permeability of the formation permits adequate flows into the wells.
- The volume and recharge rate of the aquifer permit useful long-term withdrawal rates.

Except where igneous or other impermeable rocks lie close to the surface, usable groundwater supplies are found everywhere that has a net runoff of precipitation. They are even found in deserts, but these supplies are usually fossil deposits from an earlier climatological epoch (and nonrenewable) or fed from distance precipitation and subject to low renewal rates.

It is commonly believed that well waters are pure, because they have been filtered through great thicknesses of fine-pored material. This, of course, is false. First, some common contaminants are derived from the weathering of the formations that produce the well waters. Second, there are a variety of ways by which wells become contaminated. And third, it is not true that all wells draw on naturally filtered water: in heavily weathered limestone regions, the wells may tap actual underground pools or streams.

The composition of groundwater supplies frequently exhibits strong vertical stratification. In part, this is because the waters in different strata may be separated by impermeable materials, which prevent vertical mixing of the supplies. The separated aquifers may be in formations with different mineralogies. In this case, the weathering products would be different, both in amount and kind.

Shallow aquifers near the surface are replenished by local precipitation and are vulnerable to contamination by spills on the surface and drainage from local agriculture and lawns, transportation modes, and even land treatment of used water. Most contaminants enter the aquifer with water percolating from the surface, and they tend to form a layer on top of the supply. Vertical mixing deeper into the supply well proceeds slowly unless the groundwater table undergoes seasonal variations in elevation. These variations move the surface contaminant layer up and down through the soil and so distribute the contaminant in depth.

Deep aquifers may be fed from afar and are subject to similar contamination, depending on their origin.

Common contaminants due to surface spills and land use practices are gasoline and other fuels (from leaking storage tanks), tetraethyl lead (in the gasoline), road salts (NaCl and CaCl₂), ammonia, and nitrate. The occurrence of these contaminants is predictable, and well fields can be sited to avoid them. Where this is not possible, suitable regulations can sometimes mitigate the contamination.

Common contaminants due to formation weathering are total dissolved solids, calcium, magnesium, sulfate, sodium, chloride, ferrous iron, manganous manganese, and sulfide. Occasionally, one finds arsenic, copper, lead, or mercury, which are derived from local ore deposits. Reliable treatment processes exist for the removal of all of these contaminants. However, they can frequently be avoided by switching to a different aquifer, and this is often a matter of simply changing the well depth.

Groundwaters can also be contaminated by wells. The most common problem is the well casing. The placement of the casing frequently leaves small channels between the casing wall and the surrounding formation, and these channels serve as routes by which surface drainage may enter the aquifer. These channels may be closed by grouting operations and by paving the ground near the well to deflect surface drainage. The inside of the well casing is an even better route of contamination, and the tops of the casings must be sealed to prevent the entry of surface drainage, soil, and dust.

The same problems exist with other kinds of wells that may penetrate the aquifer, even if they do not draw from it. These other wells include gas and oil wells and deep well injection facilities. In these cases, contamination from below the aquifer is also possible, because these systems are usually pressurized. The kinds of contaminants associated with gas and oil wells include various hydrocarbons, hydrogen sulfide, and brines associated with the deposits. Virtually anything can be found in deep well injection systems, and because such systems are often used to dispose of especially hazardous materials, the contamination from them may pose severe public health problems.

Well Construction and Operation

The construction and operation of wells also can contribute to water supply contamination. This occurs three ways:

- Failure to properly develop the well
- Corrosion of the well materials
- Precipitate formation during pumping

When wells are constructed, the formation next to the casing normally becomes plugged with fines (Johnson Division, 1975). This occurs in two general ways. First, if the formation consists of unconsolidated materials, the well-drilling operation will compact the materials near the casing, reducing their porosity and permeability. Second, if the well is drilled in rock, the drilling muds used to lubricate the bits and remove the cuttings will deposit clays in the pores of the rock. The compactions and deposits are eliminated by *developing* the well. This generally consists of rapidly changing the direction of flow through the well screen or flushing the screens with high-velocity water jets. Well development will also remove fine material from the formation itself, at least near the casing, and this will increase well yields. Failure to develop wells generally results in reduced water flow rates and continuing problems with sands, silts, and clays in the water supply.

Well casings and screens are subject to corrosion at varying rates and ultimately must be replaced. However, corrosion rates can be greatly accelerated if unsuitable materials are used in certain waters. Groundwater conditions leading to excessive rates of corrosion include (Johnson Division, 1975):

- pH less than 7
- Dissolved oxygen concentration greater than 2.0 mg/L
- Almost any hydrogen sulfide concentration, even below 1 mg/L
- Total dissolved solids in excess of 1000 mg/L
- Carbon dioxide concentration above 50 mg/L
- Chloride concentration greater than 500 mg/L

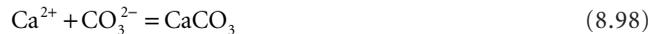
The well screen is more vulnerable to corrosion than the casing, because its function is to screen out the sands and gravels surrounding the well and prevent their entry into the pump. Any small failure of the screen will allow large amounts of material into the casing.

The corrosion products are also of interest. Most of the product will be ferrous or ferric iron. However, many older wells also have a lead packer that closes the opening between the casing and the well screen. This packer is also subject to corrosion, and this will introduce lead into the supply. Many submersible pumps have brass or bronze fittings, which leach lead.

Finally, there is precipitate formation. This occurs in the screen openings and the pores of the formation near the well. It is a problem associated with hard, alkaline waters. The flow through the formation pores and screen can be rapid, and this results in a low-pressure zone. Any dissolved gases in the formation are at equilibrium with the static formation pressure, so they tend to come out of solution in the low-pressure zone. In the case of carbon dioxide, its loss from solution causes bicarbonate to decompose, forming new carbon dioxide to replace that lost and carbonate:



The increased carbonate will precipitate calcium and ferrous iron as carbonates:



Both of these materials can enter the water distribution system, causing nuisances, and in large quantities, both can clog the screen and the surrounding formation. They are generally removed with acids, but acids can corrode the screen and casing and should be used cautiously.

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