NAZIH K. SHAMMAS • LAWRENCE K. WANG

WATER ENGINEERING

Hydraulics, Distribution and Treatment

CourseSmart

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Water Engineering

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Hydraulics, Distribution and Treatment

First Edition

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Preface

History of this Book Series: Water and Wastewater Engineering

This book, *Water Engineering*, is the second textbook in the *Water and Wastewater Engineering* series, which is a revision of the classic text, originally authored by Professors Gordon M. Fair (Harvard University), John C. Geyer (John Hopkins University), and Daniel A. Okun (University of North Carolina). Professors Daniel A. Okun and Marvin L. Granstorm (Rutger University) were the driving forces of this new global edition that includes both US and SI design equations and examples. The current authors, Professors Nazih K. Shammas and Lawrence K. Wang were the students of Professor Okun, and Professor Granstorm, respectively. Just before beginning the preparation of this new edition of *Water and Wastewater Engineering,* the last surviving member of the original authors, Professor Daniel A. Okun, died on December 10, 2007. In the normal course of events Professor Okun would have been with the current authors in preparing this book series. This new book series is dedicated to the memory of Professors Fair, Geyer, Okun, and Granstrom.

Goals of this Book: Water Engineering

Today, effective design and efficient operation of water engineering works ask, above all, for a fuller understanding and application of scientific principles. Thus, the results of scientific research are being incorporated with remarkable success in new designs and new operating procedures. Like other fields of engineering, water engineering has its science and its art. To reach the audience to which this book is addressed, the science of water engineering is given principal emphasis. However, the art of water engineering is not neglected. Enough elements of water engineering practice, experience, common sense, and rules of thumb are included to keep the reader aware of the water environment and constructions that place water at the service of cities and towns, and of villages and homesteads.

Further Study in Addition to Classroom Education

The study of scientific principles is best accomplished in the classroom; the application of these principles follows as a matter of practice. To further bridge the way from principle to practice, we suggest that the study of these textbooks be supplemented by (1) visits to water works, (2) examination of plans and specifications of existing water systems, (3) readings in the environmental science and engineering periodicals, (4) study of the data and handbook editions of trade journals, (5) examination of the catalogs and bulletins of equipment manufacturers, and (6) searching for the latest water engineering developments from the Internet.

Intended Audience

Like its forerunners, this new book, *Water Engineering*, is intended for students of civil and environmental engineering, no matter whether they belong to the student body of a university or are already established in their profession. Specifically, the target audience is engineering students who have had introductory calculus, chemistry and fluid mechanics, typically civil, environmental and water resources engineering majors. Several chapters of the book contain introductory material appropriate for juniors as well as more advanced material that might only be appropriate for upper-level undergraduate engineering students. Specifically, applied hydrology, hydraulics, and pertinent physical, chemical, and biological properties of water are reviewed. The inclusion of this material makes this book important also to physical and investment planners of urban and regional developments. In this sense, too, this book and other books in the new series should be of interest to chemical engineers, geologists, chemists, and biologists who are collaborators of the water environment.

Course Suggestions

The book is comprehensive and covers all aspects of water including its quality, sources, supply, drinking water standards, treatment, transmission, storage, and distribution. This comprehensive coverage gives faculty the flexibility of choosing the material as they find fit for their courses, and this wide coverage is helpful to water engineers in their everyday practice.

Courses where this book may be used include

1. Water engineering
2. Water supply, transmission and distribution system
3. Water treatment
4. Design of water treatment plants
5. Design of water distribution networks
6. Civil and sanitary engineering design
7. Environmental engineering design
8. Hydraulics
9. Water resources engineering

Key Features of This Book

Several items unique to this textbook include

1. **Solved problems.** A reliable problem-solving experience for students is carried out throughout the text and demonstrated in every example problem to reinforce best practices.
2. **Photos and illustrations.** Photos and illustrations are used throughout the text to clarify water engineering infrastructure systems and show examples of built and constructed water supply, transmission, treatment, storage and distribution facilities.
3. **Current water treatment and infrastructure issues.** Current infrastructure and global issues are addressed in the text. Examples of such issues include (a) established water treatment technologies; (b) conventional and new pathogenic microorganisms and impurities; (c) *Cryptosporidium*, volatile organic compounds, heavy metals, and disinfection by-products control; (d) flotation, membrane filtration, and UV; (e) groundwater under the direct influence of surface water; (f) dual water systems; (g) cross-connections control and backflow prevention; (h) design nomograms for fast water infrastructure analysis; (i) computer-aided water distribution system modeling and analysis; (j) water safety and emergency response.
4. **Engineering equations and example problems with both US and SI Units for training engineers to work globally.** The text has a multitude of examples and problems. Such problems incorporate both SI and the more customary US unit systems. We feel that most other texts fall short in both these areas by not providing students with examples that help explain difficult technical concepts and by only focusing on one system of units.
5. **Applied hydraulics.** Hydraulics concepts are critical for the civil, environmental and water resources engineering professionals, and thus the readers. Applied hydraulics topics such as pumps, weirs, pressurized pipe flow, gravity flow, head losses are reviewed in this book for practical design of water-handling facilities.
6. **Prevention through design, residuals management, and water system safety.** Chapter 23 is dedicated to prevention through design (PtD), as it is important for readers to learn about this new strategy. NIOSH is promoting the inclusion of PtD in undergraduate engineering education, has reviewed this chapter and provided the illustrative case studies described in Chapter 6, Water Distribution Systems: Components, Design, and Operation, and Chapter 22, Residues Management, Safety, and Emergency Response. Other water engineering texts do not address these important topics.

Instructor Resources

The following resources are available to instructors on the book website at:

1. **Solutions manual.** Complete solutions for every homework problem and answers to all discussion questions in the text will be available to instructors.
2. **Image gallery.** Images from the text in electronic format, for preparation of lecture PowerPoint slides.
3. **Access to student resources.** Instructors will also have access to all the student resources.

The instructor resources are password protected, and will be made available to instructors who have adopted the text for their course. Visit the instructor section of the website to register for a password.

Student Resources

The following resources are available to students on the book website at:

1. **Data sets.** Data sets for all examples and homework exercises in the text will be provided, so that students may perform what-if scenarios, and to avoid errors due to data entry during problem solving.
2. **Access to design software.** Included with the text, so students may download the software from an online source.

Software

Included with this text is access to the software Haestad Methods Water Solutions by Bentley. Software exhibited in the text, include **WaterGEMS** that is used to illustrate the application of various available software programs in helping civil and environmental engineers design and analyze water distribution systems. It is also used by water utility managers as a tool for the efficient operation of distribution systems. See Chapter 7, Water Distribution Systems: Modeling and Computer Applications.

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Books do not come off the press through the efforts of authors alone. Their preparation passes successively through editorial and production stages. The authors would like to thank our colleagues, mentors, Wiley editors/reviewers Michael Leventhal and Bob Esposito, and the resource providers and designers who have guided our efforts along these ways.

Sincere appreciations are extended to the families of Professors Fair, Geyer, Okun, and Granstrom, and the reviewers whose suggestions and comments have significantly improved the overall quality of this book series.

We also thank Dr. Richard Rinehart, Prevention through Design National Initiative at NIOSH, who reviewed PtD (Chapter 23), and Dr. Carolyn M. Jones (SFPUC Health and Safety Program Manager at NIOSH), who provided the case studies included in Chapters 6 and 22.

A book is not written in long evenings and on holidays without the consent, encouragement, and cooperation of the writers' families. This, too, should be a matter of record.

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Chapter 1

Introduction to Water Systems

The right to water is an implicit part of the right to an adequate standard of living and the right to the highest attainable standard of physical and mental health, both of which are protected by the United Nations' *International Covenant on Economic, Social and Cultural Rights,* which was established in 1976. However, some countries continue to deny the legitimacy of this right. In light of this fact and because of the widespread noncompliance of states with their obligations regarding the right to water, the United Nations' Committee on Economic, Social and Cultural Rights confirmed and further defined the right to water in its General Comment No. 15 in 2002. The comment clearly states that the right to water emanates from and is indispensable for an adequate standard of living as it is one of the most fundamental conditions for survival:

> *The human right to water entitles everyone to sufficient, safe, acceptable, physically accessible and affordable water for personal and domestic uses. An adequate amount of safe water is necessary to prevent death from dehydration, reduce the risk of water-related disease and provide for consumption, cooking, personal and domestic hygienic requirements.*

According to the World Health Organization (WHO), 1.1 billion people (17% of the global population) lack access to safe drinking water, meaning that they have to revert to unprotected wells or springs, canals, lakes, or rivers to fetch water; 2.6 billion people lack adequate sanitation; and 1.8 million people die every year from diarrheal diseases, including 90% of children under age 5. This situation is no longer bearable. To meet the WHO's *Water for Life Decade (2005–2015)*, an additional 260,000 people per day need to gain access to improved water sources.

In 2004 about 3.5 billion people worldwide (54% of the global population) had access to piped water supply through house connections. Another 1.3 billion (20%) had access to safe water through other means than house connections, including standpipes, "water kiosks," protected springs, and protected wells.

In the United States 95% of the population that is served by community water systems receives drinking water that meets all applicable health-based drinking water standards through effective treatment and source water protection. In 2007, approximately 156,000 US public drinking water systems served more than 306 million people. Each of these systems regularly supplied drinking water to at least 25 people or 15 service connections. Beyond their common purpose, the 156,000 systems vary widely. Table 1.1 groups water systems into categories that show their similarities and differences. For example, the table shows that most people in the United States (286 million) get their water from a community water system. Of the approximately 52,000 community water systems, just 8% of those systems (4048) serve 82% of the people.

Water is used in population centers for many purposes: (a) for drinking and culinary uses; (b) for washing, bathing, and laundering; (c) for cleaning windows, walls, and floors; (d) for heating and air conditioning; (e) for watering lawns and gardens; (f) for sprinkling and cleaning streets; (g) for filling swimming and wading pools; (h) for display in fountains and cascades; (i) for producing hydraulic and steam power; (j) for employment in numerous and varied industrial processes; (k) for protecting life and property against fire; and (l) for removing offensive and potentially dangerous wastes from households, commercial establishments, and industries. To provide for these varying uses, which total about 100 gallons per capita per day (gpcd) or 378 liters per capita per day (Lpcd) in average North American *residential* communities and 150 gpcd (568 Lpcd) or more in large *industrial* cities, the supply of water must be satisfactory in quality and adequate in quantity, readily available to the user, relatively cheap, and easily disposed of after it has served its many purposes. Necessary engineering works are waterworks, or water supply systems, and wastewater works, or wastewater management systems.

Waterworks withdraw water from natural sources of supply, purify it if necessary, and deliver it to the consumer. Wastewater works collect the spent water of the community—about 70% of the water supplied—together with varying amounts of entering ground and surface waters.

Water Engineering: Hydraulics, Distribution and Treatment, First Edition. Nazih K. Shammas and Lawrence K. Wang.

Table 1.1 US public water systems size by population served in 2007

Water system		Very small (500 or less)	Small (501–3,300)	Medium (3,301–10,000)	Large (10,001–100,000)	Very large (>100,000)	Total
Community water system[a]	No. of systems	29,282	13,906	4,822	3,702	398	52,110
	Population served	4,857,007	19,848,329	27,942,486	105,195,727	128,607,655	286,451,204
	Percentage of systems	56	27	9	7	1	100
	Percentage of population	2	7	10	37	45	100
Nontransient noncommunity water system[b]	No. of systems	16,034	2,662	120	22	1	18,839
	Population served	2,247,556	2,710,330	639,561	533,845	203,000	6,334,292
	Percentage of systems	85	14	1	0	0	100
	Percentage of population	35	43	10	8	3	100
Transient noncommunity water system[c]	No. of systems	81,873	2,751	102	15	3	84,744
	Population served	7,230,344	2,681,373	546,481	424,662	2,869,000	13,751,860
	Percentage of systems	97	3	0	0	0	100
	Percentage of population	53	19	4	3	21	100
Total no. of systems		127,189	19,319	5,044	3,739	402	155,693

Source: Courtesy US Environmental Protection Agency.

[a] Community water system: a public water system that supplies water to the same population year-round.

[b] Nontransient noncommunity water system: a public water system that regularly supplies water to at least 25 of the same people at least 6 months per year, but not year-round. Some examples are schools, factories, office buildings, and hospitals that have their own water systems.

[c] Transient noncommunity water system: a public water system that provides water in a place such as a gas station or campground where people do not remain for long periods of time.

The collected wastewaters are treated and reused or discharged, usually into a natural water body, more rarely onto land. Often the receiving body of water continues to serve also as a source of important water supplies for many purposes. It is this multiple use of natural waters that creates the most impelling reasons for sound water quality management.

1.1 COMPONENTS OF WATER SYSTEMS

Each section of this chapter offers, in a sense, a preview of matters discussed at length in later parts of this book. There they are dealt with as isolated topics to be mastered in detail. Here they appear in sequence as parts of the whole so that their general purpose and significance in the scheme of things may be understood and may give reason for closer study.

Municipal water systems generally comprise (a) *collection works*, (b) *purification works*, (c) *transmission works*, and (d) *distribution works*. The relative functions and positions of these components in a surface water supply are sketched in Fig. 1.1. Collection works either tap a source continuously adequate in volume for present and reasonable future demands or convert an intermittently insufficient source into a continuously adequate supply. To ensure adequacy, seasonal and, in large developments, even annual surpluses must be stored for use in times of insufficiency. When the quality of the water collected is not satisfactory, purification works are introduced to render it suitable for the purposes it must serve: contaminated water is disinfected; aesthetically displeasing water made attractive and palatable; water containing iron or manganese deferrized or demanganized; corrosive water deactivated; and hard water softened. Transmission works convey the collected and purified supply to the community, where distribution works dispense it to consumers in wanted volume at adequate pressure. Ordinarily, the water delivered is metered so that an equitable charge can be made for its use and, often, also for its disposal after use.

1.2 REQUIRED CAPACITY

Water supply systems are designed to meet population needs for a reasonable number of years in the future. The rate of consumption is normally expressed as the mean annual use in gpcd or Lpcd, and seasonal, monthly, daily, and hourly departures in rate are given in percentages of the mean. In North America the spread in consumption is large: from

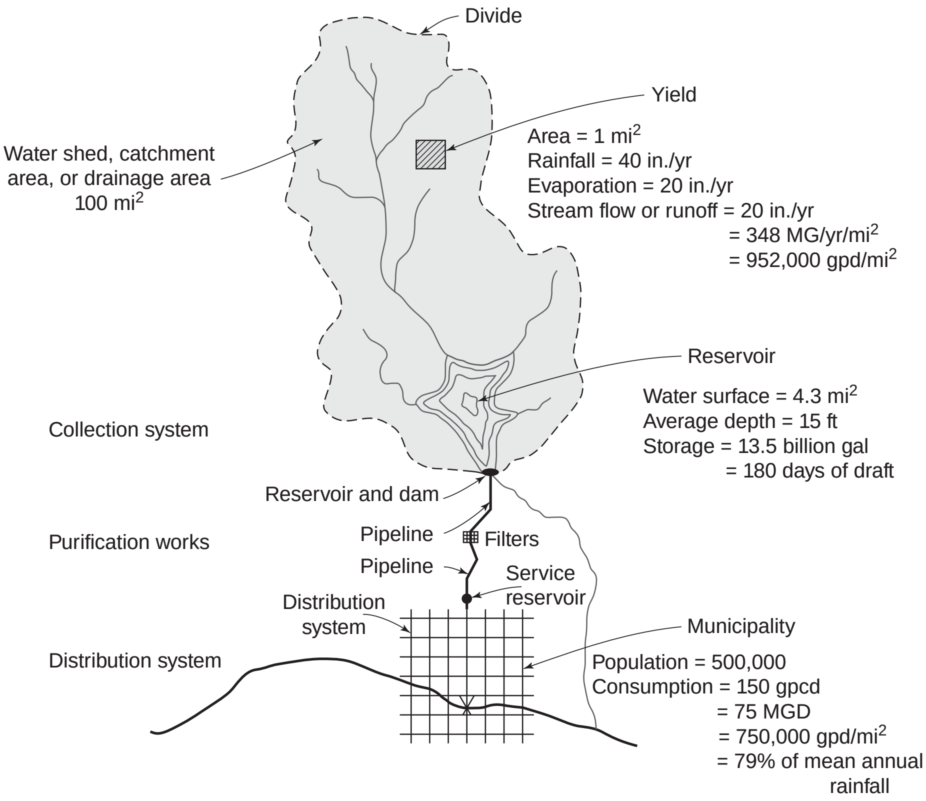


Figure 1.1 Rainfall, runoff, storage, and draft relations in the development of surface water (conversion factors: 1 mi^2 = 2.59 km^2; 1 in./yr = 25.4 mm/yr; 1 ft = 0.3048 m; 1 MG/yr/mi^2 = 1.46 ML/yr/km^2; 1 gpd/mi^2 = 1.461 L/d/km^2; 1 billion gal = 1 BG = 3.785 billion L = 3.785 BL; 1 gpcd = 3.785 Lpcd; 1 MGD = 3.785 MLD).

35 to 500 gpcd (132–1890 Lpcd), varying radically with industrial water demands. Average rates between 100 and 200 gpcd (378–757 Lpcd) are common, and a generalized average of 150 gpcd (568 Lpcd) is a useful guide to normal requirements.

The capacity of individual system components is set by what is expected of them. Distribution systems, for example, must be large enough to combat and control serious conflagrations without failing to supply maximum *coincident* domestic and industrial drafts. Fire demands vary with size and value of properties to be protected and are normally a function of the gross size of the community. The distribution system leading to the high-value district of an average American city of 100,000 people, for example, must have an excess of *fire standby* capacity equal in itself to the average rate of draft. For smaller or larger American communities, the standby capacity falls or rises, within certain limits, more or less in proportion to the square root of the population.

1.3 SOURCES OF WATER SUPPLY

The source of water commonly determines the nature of the collection, purification, transmission, and distribution works. Common sources of freshwater and their development are as follows:

1. *Rainwater*:
   (a) From roofs, stored in cisterns, for small individual supplies.
   (b) From larger, prepared watersheds, or catches, stored in reservoirs, for large communal supplies.
2. *Surface water*:
   (a) From streams, natural ponds, and lakes of sufficient size, by continuous draft.
   (b) From streams with adequate flood flows, by intermittent, seasonal, or selective draft of clean floodwaters, and their storage in reservoirs adjacent to the streams, or otherwise readily accessible from them.
   (c) From streams with low dry-weather flows but sufficient annual discharge, by continuous draft through storage of necessary flows in excess of daily use in one or more reservoirs impounded by dams thrown across the stream valleys.
   (d) From brackish and seawater by desalination. Desalination is an artificial process by which saline water is converted to freshwater. The most common desalination processes are distillation and reverse osmosis. Desalination is currently expensive compared to most alternative sources of water, and only a small fraction of total human use is satisfied by desalination. It is only economically practical for high-valued uses (such as household and industrial uses) in arid areas. The most extensive use is in the Persian (Arabian) Gulf. Mildly saline waters (brackish) are desalted most economically by reverse osmosis;

strongly saline waters by evaporation and condensation.

3. *Groundwater*:
    (a) From natural springs.
    (b) From wells.
    (c) From infiltration galleries, basins, or cribs.
    (d) From wells, galleries, and, possibly, springs, with flows augmented from some other source (i) spread on the surface of the gathering ground, (ii) carried into charging basins or ditches, or (iii) led into diffusion galleries or wells.
    (e) From wells or galleries with flows maintained by returning to the ground the water previously withdrawn from the same aquifer for cooling or similar purposes.

Several schemes have been proposed to make use of *icebergs* as a water source; to date, however, this has only been done for novelty purposes. One of the serious moves toward the practical use of icebergs is the formation of an Arabian–American investment group to search for the optimal way to transport and melt icebergs for use as a source of drinking water supply without the need for on-land storage. Glacier runoff is considered to be surface water.

An iceberg is a large piece of freshwater ice that has broken off from a snow-formed glacier or ice shelf and is floating in open water. Because the density of pure ice is about 920 kg/m^3 and that of sea water about 1025 kg/m^3, typically only one-tenth of the volume of an iceberg is above water. The shape of the rest of the iceberg under the water can be difficult to surmise from looking at what is visible above the surface. Icebergs generally range from 1 to 75 m (about 3–250 ft) above sea level and weigh 100,000–200,000 metric tonne (about 110,000–220,000 short ton). The tallest known iceberg in the North Atlantic was 168 m (about 551 ft) above sea level, making it the height of a 55 story building. Despite their size, icebergs move an average of 17 km (about 10 mi) a day. These icebergs originate from glaciers and may have an interior temperature of –15°C to −20°C (5°F to −4°F).

Municipal supplies may be derived from more than one source, the yields of available sources ordinarily being combined before distribution. *Dual public water supplies* (see Chapter 8) of unequal quality are unusual in North America. However, they do exist, for example, as a high-grade supply for general municipal uses and a low-grade supply for specific industrial purposes or firefighting. Unless the low-grade (nonpotable) supply is rigorously disinfected, its existence is frowned on by health authorities because it may be cross-connected, wittingly or unwittingly, with the high-grade (potable) supply. A *cross-connection* is defined as a junction between water supply systems through which water from doubtful or unsafe sources may enter an otherwise safe supply.

1.4 RAINWATER

Rain is rarely the immediate provenance of municipal water supplies. Instead, the capture of rainwater is confined to farms and rural settlements usually in semiarid regions devoid of satisfactory ground or surface waters. On homesteads, rainwater running off roofs is led through gutters and downspouts to rain barrels or cisterns situated on or in the ground. Storage transforms the intermittent rainfall into a continuous supply. For municipal service, sheds or catches on ground that is naturally impervious or made tight by grouting, cementing, paving, or similar means must usually be added.

The gross yield of rainwater is proportional to the receiving area and the amount of precipitation. However, some rain

EXAMPLE 1.1 CALCULATING THE VOLUME OF RAINFALL THAT CAN BE COLLECTED FROM A BUILDING ROOF

Make a rough estimate of the volume in gallons or liters of water that can be caught by 3,000 ft^2 (278.7 m^2) of horizontally projected roof area (the average area of American farm buildings) in a region where the mean annual rainfall is 15 in. (38.1 cm).

Solution 1 (US Customary System):

$$\text{Gross yield} = 3{,}000\ \text{ft}^2 \times (15/12\ \text{ft}) \times 7.48\ \text{gal/ft}^3 = 28{,}100\ \text{gal annually} = 28{,}100\ \text{gal}/365\ \text{days} = 77\ \text{gpd}.$$

Net yield approximates two-thirds gross yield = 18,800 gal annually = **51 gpd**.

About half the net annual yield, or 9,400 gal = 1,250 ft^3, must normally be stored to tide the supply over dry spells.

Solution 2 (SI System):

$$\text{Gross yield} = (278.7\ \text{m}^2)(38.1/100\ \text{m})(1{,}000\ \text{L/m}^3) = 106{,}178\ \text{L annually} = 291\ \text{L/day} = 291\ \text{L/d}.$$

Net yield approximates two-thirds gross yield = 291 L/d (2/3) = **194 L/d** = 70,790 L/year.

About half the net annual yield = 0.5 (70,790 L/year) = 35,395 L = 35.4 m^3 must be stored to tide the supply over dry spells.

is blown off the roof, evaporated, or lost in wetting the collecting surfaces and conduits and in filling depressions or improperly pitched gutters. Also, the first flush of water may have to be wasted because it contains dust, bird droppings, and other unwanted materials. The combined loss may be high. A cutoff, switch, or deflector in the downspout permits selective diversion of unwanted water from the system. Sand filters will cleanse the water as it enters the cistern and prevent its deterioration via the growth of undesirable organisms and consequent tastes, odors, and other changes in attractiveness and palatability.

The storage to be provided in *cisterns* depends on the distribution of rainfall. Storage varies with the length of dry spells and commonly approximates one-third to one-half the annual consumption. If rainfalls of high intensity are to be captured, standby capacity must exist in advance of filtration. Because their area is small, roofs seldom yield much water. A careful analysis of storm rainfalls and seasonal variations in precipitation is, therefore, required.

1.5 SURFACE WATER

In North America by far the largest volumes of municipal water are collected from surface sources. The quantities that can be gathered vary directly with the size of the catchment area, or watershed, and with the difference between the amounts of water falling on it and the amounts lost by evapotranspiration. The significance of these relationships to water supply is illustrated in Fig. 1.1. Where surface water and groundwater sheds do not coincide, some groundwater may enter from neighboring catchment areas or escape to them.

1.5.1 Continuous Draft

Communities on or near streams, ponds, or lakes may take their supplies from them by continuous draft if stream flow and pond or lake capacity are high enough at all seasons of the year to furnish requisite water volumes. Collecting works ordinarily include (a) an intake crib, gatehouse, or tower; (b) an intake conduit; and (c) in many places, a pumping station. On small streams serving communities of moderate size, an intake or diversion dam may create sufficient depth of water to submerge the intake pipe and protect it against ice. From intakes close to the community the water must generally be lifted to purification works and thence to the distribution system.

Most large streams are polluted by wastes from upstream communities and industries. Purification of their waters is then a necessity. Cities on large lakes usually guard their supplies against their own and their neighbor's wastewater and spent industrial-process waters by moving their intakes far away from shore and purifying both their water and wastewater. Diversion of wastewater from lakes will retard the lakes' eutrophication.

1.5.2 Selective Draft

Low stream flows are often left untouched. They may be wanted for other downstream purposes or they may be too highly polluted for reasonable use. Only clean floodwaters are then diverted into reservoirs constructed in meadow lands adjacent to the stream or otherwise conveniently available. The amount of water so stored must supply demands during seasons of unavailable stream flow. If draft is confined to a quarter year, for example, the reservoir must hold at least three-fourths of the annual supply. In spite of its selection and long storage, the water may have to be purified.

1.5.3 Impoundage

In their search for clean water and water that can be brought and distributed to the community by gravity, engineers have developed supplies from upland streams. Most of them are tapped near their source in high and sparsely settled regions. To be of use, their annual discharge must equal or exceed the demands of the community they serve for a reasonable number of years in the future. Because their dry season flows generally fall short of concurrent municipal requirements, their floodwaters must usually be stored in sufficient volume to ensure an adequate supply. Necessary reservoirs are impounded by throwing dams across the stream valley. In this way, amounts up to the mean annual flow can be utilized. The area draining to an impoundment is known as the catchment area or watershed. Its economical development depends on the value of water in the region, but it is a function, too, of runoff and its variation, accessibility of catchment areas, interference with existing water rights, and costs of construction. Allowances must be made for evaporation from new water surfaces generated by the impoundage (Fig. 1.2) and also often for release of agreed-on flows to the valley below the dam (compensating water). Increased ground storage in the flooded area and the gradual diminution of reservoir volumes by siltation must also be considered.

Intake structures are incorporated in impounding dams or kept separate. Other important components of impounding reservoirs are (a) spillways safely passing floods in excess of reservoir capacity and (b) diversion conduits safely carrying the stream past the construction site until the reservoir has been completed and its spillway can go into action. Analysis of flood records enters into the design of these ancillary structures.

Some impounded supplies are sufficiently safe, attractive, and palatable to be used without treatment other than protective disinfection. However, it may be necessary to remove high color imparted to the stored water by the decomposition of organic matter in swamps and on the flooded valley bottom; odors and tastes generated in the decomposition or growth of algae, especially during the first years after filling; and turbidity (finely divided clay or silt) carried into streams or reservoirs by surface wash, wave action, or bank

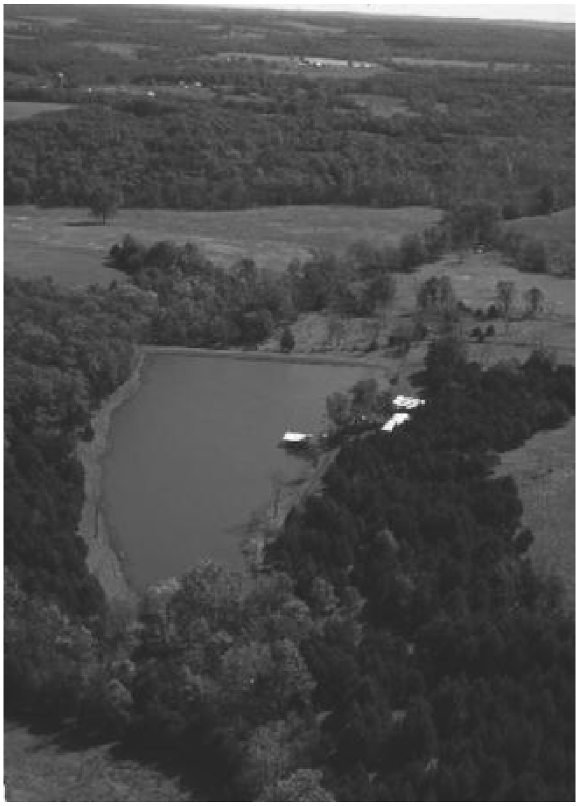


Figure 1.2 A watershed lake in Western Missouri provides water supply (Courtesy of the National Resources Conservation Service and USDA).

erosion. Recreational uses of watersheds and reservoirs may call for treatment of the flows withdrawn from storage.

Much of the water in streams, ponds, lakes, and reservoirs in times of drought, or when precipitation is frozen, is seepage from the soil. Nevertheless, it is classified as surface runoff rather than groundwater. Water seeps *from* the ground when surface streams are low and *to* the ground when surface streams are high. Release of water from ground storage or from accumulations of snow in high mountains is a determining factor in the yield of some catchment areas. Although surface waters are derived ultimately from precipitation, the relations between precipitation, runoff, infiltration, evaporation, and transpiration are so complex that engineers rightly prefer to base calculations of yield on available stream gaugings. For adequate information, gaugings must extend over a considerable number of years.

1.6 GROUNDWATER

Smaller in daily delivery, but many times more numerous than surface water supplies, are the municipal and private groundwater supplies of North America. Groundwater is drawn from many different geological formations: (a) from the pores of alluvial (water-borne), glacial, or aeolian (wind-blown) deposits of granular, unconsolidated materials such as sand and gravel, and from consolidated materials such as sandstone; (b) from the solution passages, caverns, and cleavage planes of sedimentary rocks such as limestone, slate, and shale; (c) from the fractures and fissures of igneous rocks; and (d) from combinations of these unconsolidated and consolidated geological formations. Groundwater sources, too, have an intake or catchment area, but the catch, or recharge, is by infiltration into soil openings rather than by runoff over its surface. The intake area may be nearby or a considerable distance away, especially when flow is confined within a water-bearing stratum or *aquifer* (from Latin *aqua*, “water,” and *ferre*, “to bear”) underlying an impervious stratum or *aquiclude* (from Latin *aqua*, “water,” and *cludere*, “to shut” or “to close out”).

EXAMPLE 1.2 ESTIMATES OF YIELDS FROM WATERSHEDS AND STORAGE REQUIREMENTS

Certain rough estimates of the yield of surface watersheds and storage requirements are shown in Fig. 1.1. Rainfall is used as the point of departure, merely to identify the dimensions of possible rainfall–runoff relationships. Determine

1. The yields from the watersheds,
2. The storage requirements,
3. The number of people who can be supported by a drainage area of 100 mi^2 (259 km^2) if there is adequate impoundage for water storage, and
4. The number of people who can be supported by a drainage area of 100 mi^2 (259 km^2) if there is no impoundage for water storage.

The following assumptions are made: (a) rainfall = 20 in./km^2 annually = 19.6 cm/km^2, (b) a stream flow of about 1 MGD/km^2 (million gallons per day per square mile) or (1.547 ft^3/s)/km^2 [or 1.46 MLD/km^2 (million liters per day per square kilometer)] is a good average for the well-watered sections of North America, (c) for 75% development (0.75 × 1 MGD/km^2 or 0.75 × 1.46 MLD/km^2), about half a year’s supply must generally be stored. In semiarid regions storage of three times the mean annual stream flow is not uncommon, that is, water is held over from wet years to supply demands during dry years, (d) average water consumption = 150 gpcd = 567.8 Lpcd, (e) for water supply by continuous draft, low water flows rather than average annual yields govern. In well-watered sections of North America, these approximate 0.1 ft^3/s or 64,600 gpd/km^2 (or 28.32 L/s, or 0.094316 MLD/km^2).

Solution 1 (US Customary System):

1. The following conversion factors and approximations are being employed:

   1 in. rainfall/km^2 = 17.378 MG

   Hence, 20 in./km^2 annually = 20 × 17.378 = 348 MG or 348/365 = **0.952 MGD**.

2. A stream flow of about 1 MGD/km^2 is a good average for the well-watered sections of North America. Not all of it can be adduced economically by storage.

   For 75% development (0.75 MGD/km^2, or 750,000 gpd/km^2), about half a year's supply must generally be stored. For a catchment area of 100 km^2, therefore

   Storage = (0.75 MGD/km^2)(100 km^2) × (0.5 × 365 days) = 13,688 MG = **13.5 BG** (billion gallons) approximately.

   In semiarid regions storage of three times the mean annual stream flow is not uncommon, that is, water is held over from wet years to supply demands during dry years.

3. For an average consumption of 150 gpcd, the drainage area of 100 km^2 and impoundage of 13.5 BG will supply a population of 100 × 750,000/150 = **500,000 persons**.

4. For water supply by continuous draft, low water flows rather than average annual yields govern.

   In well-watered sections of North America, these approximate 0.1 ft^3/s or 64,600 gpd/km^2.

   A catchment area of 100 km^2, therefore, can supply without storage

   100 × 64,600/150 = **43,000 people**.

   This is compared against 500,000 people in the presence of proper storage.

Solution 2 (SI System):

1. The following conversion factors and approximations are being employed:

   1 cm/km^2 = 67.12 ML (million liters)

   Hence, 19.6 cm/km^2 annually = 19.6 × 67.12 = 1315.6 ML annually = **3.6 MLD**.

2. A stream flow of about 1.46 MLD/km^2 is a good average for the well-watered sections of North America. Not all of it can be adduced economically by storage.

   For 75% development (0.75 × 1.46 MLD/km^2), about half a year's supply must generally be stored.

   For a catchment area of 259 km^2, therefore

   Storage = 0.75(1.46 MLD/km^2)(259 km^2)(0.5 × 365) = 51,758 ML = **51.758 BL** (billion liters).

   In semiarid regions storage of three times the mean annual stream flow is not uncommon, that is, water is held over from wet years to supply demands during dry years.

3. For an average consumption of 567.8 Lpcd, the drainage area of 259 km^2 and impoundage of 51.758 BL will supply a population of

   (0.75 × 1.46 MLD/km^2)(259 km^2)(1,000,000 L/ML)/(567.8 Lpcd) = **500,000 persons**.

4. For water supply by continuous draft, low water flows rather than average annual yields govern. In well-watered sections of North America these approximate 28.32 L/s or 0.094316 MLD/km^2.

   A catchment area of 259 km^2, therefore, can supply without storage

   (259 km^2)(0.094316 MLD/km^2)(1,000,000 L/ML)/(567.8 Lpcd) = **43,000 people**.

   This is compared against 500,000 people in the presence of proper storage.

The maximum yield of groundwater is directly proportional to the size of the intake area and to the difference between precipitation and the sum of evapotranspiration and storm runoff. Laterally, flow extends across the width of the aquifer; vertically, it is as deep as the zone of open pores and passages in Earth's crust and as shallow as the *groundwater table*. When the water surface rises and falls with seasonal changes in recharge, flow is unconfined or free, and the groundwater table slopes downward more or less parallel to the ground surface. Flow then moves at right angles to the water table contours. If a porous stratum dips beneath an impervious layer, flow is confined as in a pipe dropping below the hydraulic grade line. When this kind of aquifer is tapped, *artesian water* rises from it under pressure, in some geological situations, even in free-flowing fountains. In other geological formations, water is perched on a lens of impervious material above the true groundwater table.

Groundwater reaches daylight through springs when (a) the ground surface drops sharply below the normal groundwater table (depression springs); (b) a geological obstruction impounds soil water behind it and forces it to the surface (contact springs); and (c) a fault in an impervious stratum lets artesian water escape from confinement (also contact springs). A cutoff wall carried to bedrock will hold back subsurface as well as surface flows behind an impounding dam and so put the full capacity of the catchment area to use unless there is lateral leakage through the sides of the reservoir or around the abutments of the dam.

The rate of flow through the substantially vertical cross-section of ground at right angles to the direction of flow is not great. Because of the high resistance of the normally narrow pores of the soil, the water moves forward only slowly, traveling about as far in a year as stream flow does in an hour. Natural rates of flow are seldom more than a few feet per hour (or meters per hour); nor are they less than a few feet per day (or meters per day) in aquifers delivering useful water supplies. However, if a well is sunk into the ground and the level of water in it is lowered by pumping, water is discharged into the well not only from the direction of natural flow but from all directions. That is why wells can be spaced many times their own diameter apart and yet intercept most of the water escaping through the intervening space.

1.6.1 Springs

Springs are usually developed to capture the natural flow of an aquifer. In favorable circumstances their yield can be increased by driving collecting pipes or galleries, more or less horizontally, into the water-bearing formations that feed them. Pollution generally originates close to the point of capture. It is prevented by (a) excluding shallow seepage waters through encircling the spring with a watertight chamber penetrating a safe distance into the aquifer and (b) diverting surface runoff away from the immediate vicinity. Some springs yield less than 1 gpm (3.78 L/min); a few yield more than 50 MGD (189 MLD). Some are perennial; others are periodically or seasonally intermittent.

Figure 1.3 A well provides about 1.5 MGD (5.68 MLD) of water to Central Maui, HI (Courtesy of the Department of Water Supply, Maui County, HI).

1.6.2 Wells

Depending on the geological formations through which they pass and on their depth, wells are *dug*, *driven*, *bored*, or *drilled* into the ground. A well and its pumping equipment are shown in Fig. 1.3. Dug and driven wells are usually confined to soft ground, sand, and gravel at depths normally less than 100 ft (30 m). Hard ground and rock generally call for bored and drilled wells sunk to depths of hundreds and even thousands of feet. In well-watered regions successful wells of moderate depth and diameter yield 1–50 gpm (4–190 L/min) in hard rock and 50–500 gpm (190–1900 L/min) in coarse sand and gravel as well as coarse sandstone. Wells in deep aquifers may yield 100 gpm (400 L/min) or more.

Except in hard rock, particularly limestone, without sand or gravel cover, wells are generally not polluted by lateral seepage but by vertical entrance of pollution at or near the ground surface. Pollution is excluded by watertight casings or seals extending into the aquifer and at least 10 ft (3 m) below the ground surface, together with diversion of surface runoff from the well area and its protection against inundation by nearby streams.

1.6.3 Infiltration Galleries

Groundwater traveling toward streams or lakes from neighboring uplands can be intercepted by infiltration galleries laid more or less at right angles to the direction of flow and carrying entrant water to pumping stations. Water is drawn into more or less horizontal conduits from both sides, or the riverside is blanked off to exclude the often less satisfactory water seeping in from the river itself. Infiltration basins

EXAMPLE 1.3 DETERMINATION OF AQUIFER YIELD

Make a rough estimate of the yield of an aquifer 20 ft (6.10 m) deep through which water moves at a rate of 3 ft (0.91 m) a day (1) if all of the groundwater laterally within 500 ft (152 m) of the well comes fully within its influence and (2) if a gallery 1,000 ft (305 m) long collects water from both sides.

Solution 1 (US Customary System):

1. $20\ \text{ft} \times 500\ \text{ft} \times 2 \times 3\ \text{ft/d} \times (7.5\ \text{gal/ft}^3)/(1{,}440\ \text{min/d}) = \mathbf{310\ gpm}$.
2. $20\ \text{ft} \times 1{,}000\ \text{ft} \times 2 \times 3\ \text{ft/d} \times (7.5\ \text{gal/ft}^3)/(1{,}000{,}000) = \mathbf{0.90\ MGD}$.

Solution 2 (SI System):

1. $(6.10\ \text{m})(305\ \text{m})(0.91\ \text{m})(1{,}000\ \text{L/m}^3)(1/1{,}440\ \text{min}) = 1176\ \text{L/min}$.
2. $2(6.10\ \text{m})(305\ \text{m})(0.91\ \text{m})/\text{d} = 3{,}397\ \text{m}^3/\text{d} = 3{,}396{,}500\ \text{L/d} = \mathbf{3.4\ MLD}$.

and trenches are similar in conception. They are, in essence, large, or long, shallow, open wells. Filter cribs built into alluvial deposits of streams intercept the underflow. Groundwater can also be collected from the driftways and slopes of mines, galleries driven into mountainsides specifically for this purpose, or abandoned mines. Some infiltration galleries yield as much as 1 MGD/1,000 ft (12.4 MLD/1,000 m) of gallery. They are particularly useful in tapping aquifers of shallow depth or where deep saline waters are to be excluded.

1.6.4 Recharging Devices

As outlined earlier, the yield of groundwater works can be augmented or maintained at high level by water spreading or diffusion. The necessary structures are built close to the collecting works within the groundwater shed. Charging ditches or basins are filled with river or lake water by gravity or pumping. In the flooding method, water diverted from streams by check dams is led onto a suitable area of pervious soils. The applied waters soak into the ground and increase its natural flows. The incentive is either augmentation of a dwindling or inadequate supply or taking advantage of natural filtration as a means of water purification. Gathering a more uniformly cool water is also a consideration. Badly polluted surface water may be partially purified before it is introduced into the charging structure. Some diffusion galleries and wells return waters abstracted earlier from the ground for cooling and other purposes.

Groundwater collection works usually include pumps. To them water flows from all or much of the well field either by gravity through deep-lying conduits or under negative pressure through suction mains. Individual pumping units are often used instead, especially when the water table lies at considerable depths.

Most natural groundwaters are clean, palatable, and cool. However, passage through some soils may make them unpalatable, unattractive, corrosive, or hard (soap consuming). Their treatment must be varied according to needs.

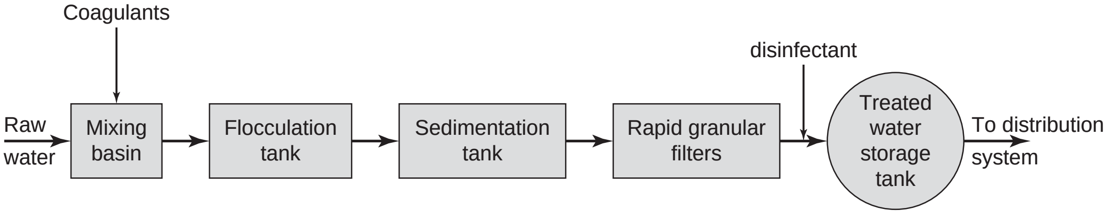
To determine the yield of groundwater areas, the engineer must know the geology as well as the hydrology of the region. He can learn much from existing supplies in nearby areas, but his ultimate judgment must generally rest on the behavior of test wells.

1.7 PURIFICATION WORKS

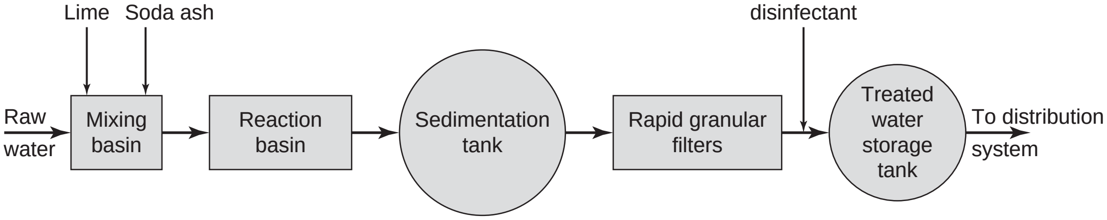
The quality of some waters from surface or ground sources is naturally satisfactory for all common uses. *Disinfection* may be the only required safeguard. Other waters contain objectionable substances that must be removed, reduced to tolerable limits, destroyed, or otherwise altered in character before the water is sent to the consumer. Impurities are acquired in the passage of water through the atmosphere, over the earth's surface, or through the pores of the earth. Their pollution is associated with man's activities, in particular, with his own use of water in household and industry and the return of spent water to natural water courses. Some of the *heavy metals* (lead, copper, zinc, and iron) come from the corrosion of metallic water pipes. Contamination of distribution systems through cross-connections with impure water supplies and through *backflow* in plumbing systems is another hazard. (Backflow permits water drawn into a fixture, tank, or similar device to flow back into the supply line by gravity or by siphonage.)

How to treat a given supply depends on its inherent traits and on accepted water quality standards. Municipal works must deliver water that is (a) hygienically safe, (b) aesthetically attractive and palatable, and (c) economically satisfactory for its intended uses. The most common classes of municipal water purification works and their principal functions are as follows:

1. *Filtration plants* remove objectionable color, turbidity, and bacteria as well as other potentially harmful organisms by filtration through sand or other granular substances after necessary preparation of the water by coagulation and sedimentation (Fig. 1.4a).



(*a*) Surface water filtration plant



(*b*) Groundwater softening plant

Figure 1.4 Common types of water treatment plants. (Note: A sedimentation tank may be replaced by a dissolved air flotation tank.)

2. *Iron and manganese treatment plants* remove excessive amounts of iron and manganese by oxidizing the dissolved metals and converting them into insoluble flocs removable by sedimentation and filtration.
3. *Softening plants* remove excessive amounts of scale-forming, soap-consuming ingredients, chiefly calcium and magnesium ions (a) by the addition of lime and soda ash, which precipitate calcium as a carbonate and magnesium as a hydrate (Fig. 1.4b) or (b) by passage of the water through cation exchange media that substitute sodium for calcium and magnesium ions and are themselves regenerated by brine.

Today most water supplies are either chlorinated or ozonated to ensure their disinfection. Lime or other chemicals are often added to reduce the corrosiveness of water to iron and other metals and so to preserve water quality during distribution and ensure a longer life for metallic pipes in particular. Odor- or taste-producing substances are adsorbed onto activated carbon or destroyed by high doses of chlorine, chlorine dioxide, or other oxidants. Numerous other treatment methods serve special needs. The perspective of a water treatment plant in northern Portugal is shown in Fig. 1.5.

Water purification plants must take into consideration these design functions:

1. *Process design:* An understanding of unit operations that bring about the removal or modification of objectionable substances.
2. *Hydraulic design:* A knowledge of how water flows through the structures composing water purification plants: channels, pipes including perforated pipes, gates, measuring devices, basins, beds of sand and other granular materials, and pumps.
3. *Structural design:* A comprehension of the behavior of needed structures under load.
4. *Economic design:* An appreciation of treatment costs and associated benefits.

The following normally applicable requirements provide the reader with a concept of the sizing of principal structures:

1. *Mixing basins* hold a few minutes of flow.
2. *Flocculating and reaction basins* hold about half an hour's flow.
3. *Sedimentation basins* hold an hour or more of flow and are rated at about 0.50 gpm/ft^2 (20 L/min/m^2) of water surface area.

Figure 1.5 Water treatment plant in Braganca, Portugal (*Source:* http://en.wikipedia.org/wiki/Image:Bragan%C3%A7a43.jpg).

EXAMPLE 1.4 DETERMINATION OF THE CAPACITY OF TREATMENT PLANT UNITS

Estimate the capacity of the components of a rapid sand filtration plant (Fig. 1.5) that is to deliver 10 MGD or 6,940 gpm (37.85 MLD or 26,268 L/min) of water to a city of 67,000 people.

Solution 1 (US Customary System):

1. Two mixing basins, $H = 10$ ft deep; number of mixing basins $N = 2$.
   - (a) Assumed detention period $t = 2$ min.
   - (b) Volume $V = Qt/N = 6{,}940 \times 2/2 = 6{,}940$ gal $= 928$ ft^3 each.
   - (c) Surface area $A = V/H = 928/10 = 92.8$ ft^2 $= 0.785\ D^2$.
   - (d) Diameter $D = \sqrt{\dfrac{A \times 4}{\pi}} = \sqrt{\dfrac{92.8 \times 4}{\pi}} = \mathbf{10.9\ ft}$.
2. Two flocculating basins, $H = 10$ ft deep.
   - (a) Assumed detention period $t = 30$ min; number of flocculating basins $N = 2$.
   - (b) Volume $V = Qt/N = 6{,}940 \times 30/2 = 104{,}000$ gal $= 13{,}900$ ft^3.
   - (c) Surface area $A = V/H = (13{,}900\ \text{ft}^3)/(10\ \text{ft}) = \mathbf{1{,}390\ ft^2}$ each (such as 20 ft by 70 ft).
3. Two settling basins, $H = 10$ ft deep, but allow for 2 ft of sludge; number of settling basins $N = 2$.
   - (a) Assumed detention period $t = 2$ h.
   - (b) Effective volume $V = Qt/N = 6{,}940 \times 2 \times 60/2 = 416{,}000$ gal $= 55{,}700$ ft^3.
   - (c) Surface area $A = V/H = 55{,}700/(10 - 2)$ ft $= \mathbf{6{,}960\ ft^2}$ (such as 35 ft by 200 ft).
   - (d) Surface rating $\text{SR} = Q/A = 6{,}940/6{,}960 = 1.0\ \text{gpm/ft}^2$.
4. Six rapid sand filters.
   - (a) Assumed surface rating $\text{SR} = Q/A = 3\ \text{gpm/ft}^2$; number of filters $N = 6$.
   - (b) Area $A = Q/(N \times \text{SR}) = 6{,}940/(6 \times 3) = \mathbf{385\ ft^2}$ (such as 15 ft by 26 ft).

Solution 2 (SI System):

1. Two mixing basins, $H = 3.05$ m deep; number of mixing basins $N = 2$.
   - (a) Assumed detention period $t = 2$ min.
   - (b) Volume $V = Qt/N = (26{,}268 \times 2)/2 = 26{,}268$ L $= 26.27$ m^3 each.
   - (c) Surface area $A = V/H = (26.27/3.048) = 8.62$ m^2.
   - (d) Diameter $D = \sqrt{\dfrac{A \times 4}{\pi}} = \sqrt{\dfrac{8.62 \times 4}{\pi}} = \mathbf{3.31\ m}$.
2. Two flocculating basins, $H = 3.05$ m deep.
   - (a) Assumed detention period $t = 30$ min; number of flocculating basins $N = 2$.
   - (b) Volume $V = Qt/N = (26{,}268\ \text{L/min} \times 30\ \text{min})/2 = 394{,}020$ L $= 394$ m^3.
   - (c) Surface area $A = V/H = (394\ \text{m}^3)/(3.05\ \text{m}) = \mathbf{129.27\ m^2}$ each (such as $\mathbf{6.1\ m \times 21.3\ m}$).
3. Two settling basins, $H = 3.05$ m deep, but allow for 0.61 m of sludge; number of settling basins $N = 2$.
   - (a) Assumed detention period $t = 2$ h.
   - (b) Effective volume $V = Qt/N = (26{,}268\ \text{L/min} \times 2 \times 60\ \text{min})/2 = 1{,}576{,}080$ L $= 1{,}576$ m^3.
   - (c) Surface area $A = V/H = (1{,}576\ \text{m}^3)/(3.05\ \text{m} - 0.61\ \text{m}) = \mathbf{646\ m^2}$ (such as 10.7 m by 61 m).
   - (d) Surface rating $\text{SR} = Q/A = (26{,}268\ \text{L/min})/(646\ \text{m}^2) = 40.7\ \text{L/min/m}^2$.
4. Six rapid sand filters.
   - (a) Assumed surface rating $\text{SR} = Q/A = 122.1\ \text{L/min/m}^2$; number of filters $N = 6$.
   - (b) Area $A = Q/(N \times \text{SR}) = (26{,}268\ \text{L/min})/(6 \times 122.1) = \mathbf{35.86\ m^2}$ (such as 4.6 m by 7.9 m).

4. *Slow sand filters* pass water at rates of about 3 MGD/acre (28 MLD/ha) in surface water filtration, stepping up to about 10 MGD/acre (94 MLD/ha) in groundwater treatment for iron and manganese removal or when they are preceded by roughing filters.
5. *Rapid filters* operate at rates of 125 MGD/acre or 2 gpm/ft^2 (1170 MLD/ha or 81 L/min/m^2), but rates run higher in modern works that include flocculating chambers.
6. *Coke tricklers* for aeration are rated at about 75 MGD/acre or 1.2 gpm/ft^2 (700 MLD/ ha or 50 L/min/m^2).

1.8 TRANSMISSION WORKS

Supply conduits, or aqueducts, transport water from the source of supply to the community and so form the connecting link between collection works and distribution systems. Source location determines whether conduits are short or long and whether transport is by gravity or pumping. Depending on topography and available materials, conduits are designed for open-channel or pressure flow. They may follow the hydraulic grade line as canals dug through the ground, flumes elevated above the ground, grade aqueducts laid in balanced cut and cover at the ground surface, and grade tunnels penetrating hills; or they may depart from the hydraulic grade line as pressure aqueducts laid in balanced cut and cover at the ground surface, pressure tunnels dipping beneath valleys or hills, and pipelines of fabricated materials following the ground surface, if necessary over hill and through dale, sometimes even rising above the hydraulic grade line. The 336 mi (541 km) long Central Arizona Project aqueduct shown in Fig. 1.6 is the largest and most expensive aqueduct system ever constructed in the United States. The Colorado River aqueduct of the Metropolitan Water District of Southern California is 242 mi (389 km) long and includes 92 mi (148 km) of grade tunnel, 63 mi (101 km) of canal, 54 mi (87 km) of grade aqueduct, 29 mi (47 km) of inverted siphons, and 4 mi (6.4 km) of force main. The Delaware aqueduct of New York City comprises 85 mi (137 km) of pressure tunnel in three sections. Pressure tunnels 25 mi (40 km) in length supply the metropolitan districts of Boston and San Francisco. The supply conduits of Springfield, MA, are made of steel pipe and reinforced concrete pipe and those of Albany, NY, of cast-iron pipe (CIP).

The size and shape of supply conduits are determined by hydraulic, structural, and economic considerations. Velocities of flow ordinarily lie between 3 and 5 ft/s (0.91 and 1.52 m/s). Requisite capacities depend on the inclusion and size of supporting *service* or *distributing reservoirs*. If these store enough water to (a) care for hourly variations in water consumption in excess of inflow, (b) deliver water needed to fight serious fires, and (c) permit incoming lines to be shut down for inspection and minor repairs, the supply conduits need operate only at the maximum daily rate, about 50% in excess of the average daily rate. Ordinarily, required storage approximates a day's consumption. Distribution reservoirs are open or covered basins in balanced cut and fill, standpipes, or elevated tanks. Selection depends on size and location in particular reference to available elevations above the area served (Fig. 1.7). More than one reservoir may be needed in large systems. Open reservoirs are troubled by soot and dust falls, by algal growths, and in seacoast cities by sea gulls. Today, covered reservoirs are preferred.

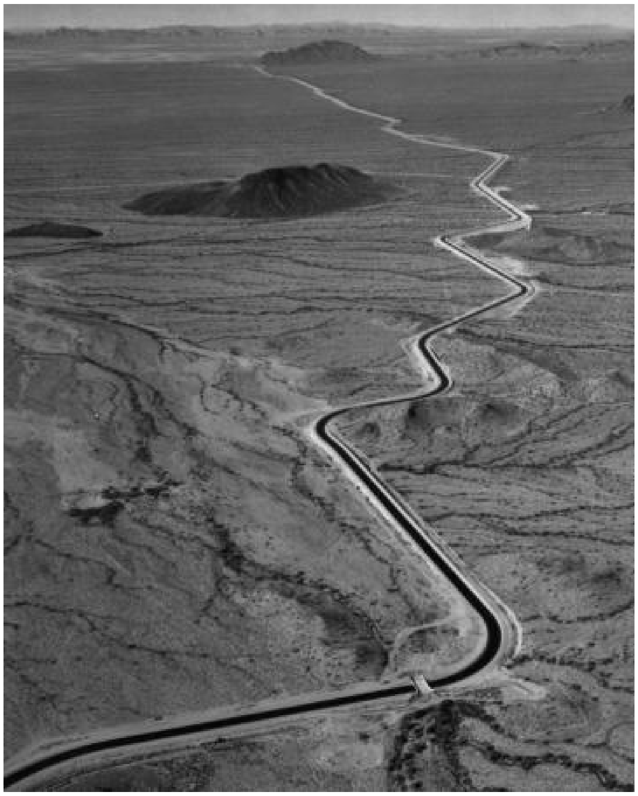


Figure 1.6 Central Arizona Project aqueduct (*Source:* http://en.wikipedia.org/wiki/Image:Arizona_cap_canal.jpg).

1.9 DISTRIBUTION WORKS

Supply conduits (Fig. 1.8) feed their waters into the distribution system that eventually serves each individual property—household, mercantile establishment, public building, or factory (Fig. 1.1). Street plan, topography, and location of supply works and service storage establish the type of distribution system and its character of flow. In accord with the street plan, two distribution patterns emerge: (a) a *branching pattern* on the outskirts of the community, in which ribbon development follows the primary arteries of roads and streets (Fig. 1.9a), and (b) a *gridiron pattern* within the built-up portions of the community where streets crisscross and water mains are interconnected (Fig. 1.9).

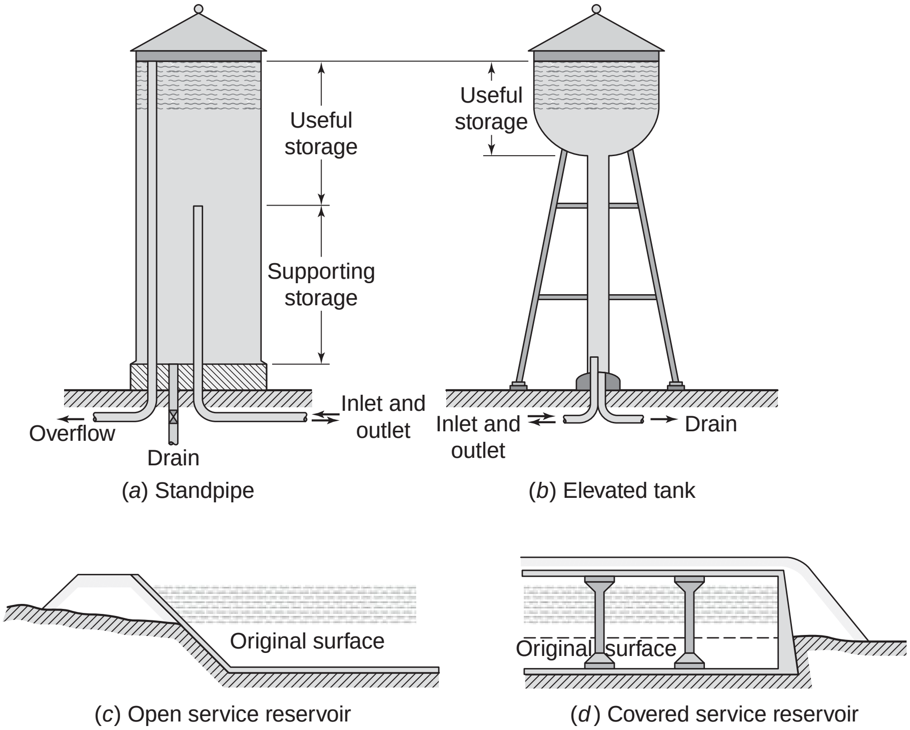


Figure 1.7 Four types of service, or distribution, reservoirs.

Hydraulically, the gridiron system has the advantage of delivering water to any spot from more than one direction and of avoiding dead ends. Gridiron systems are strengthened by substituting for a central feeder a loop or belt of feeders that supplies water to the *congested*, or *high-value*, district from at least two directions. This more or less doubles the delivery of the grid. In large systems feeders are constructed as pressure tunnels, pressure aqueducts, steel pipes, or reinforced concrete pipes. In smaller communities the entire distribution system may consist of CIPs. Cast iron is, indeed, the most common material for water mains, but asbestos-cement, in general, and plastics, in the case of small supplies, are also important.

Figure 1.8 A pipeline in the Goldfields Water Supply Scheme, Perth, Australia (*Source:* http://en.wikipedia.org/wiki/Image:Goldfields_Pipeline_SMC.JPG).

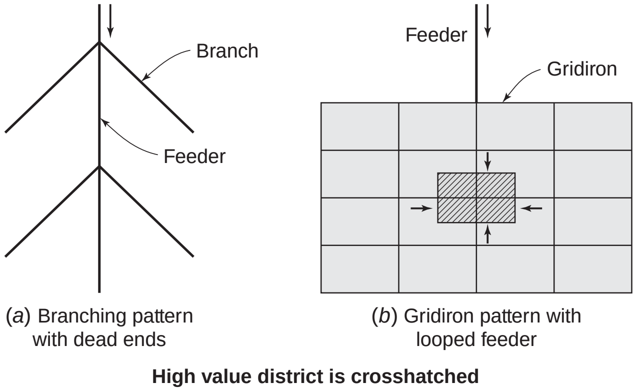


Figure 1.9 Patterns of water distribution systems.

EXAMPLE 1.5 ESTIMATION OF THE SIZE OF A WATER CONDUIT

Estimate roughly the size of a supply conduit leading to an adequate distributing reservoir serving (i) relatively small residential community of 10,000 people and (ii) a relatively large industrial community of 400,000 people.

The following are the design conditions specifically for a North American region:

a. The average daily water consumption for small communities with populations of 10,000 or less = 100 gpcd (378.5 Lpcd).
b. Average daily water consumption for communities with populations of greater than 10,000 = 150 gpcd (567.8 Lpcd).
c. Maximum daily water consumption is about 50% greater than average daily water consumption.
d. The design water velocity in the circular conduit when flowing full = 4 ft/s = 1.22 m/s.

Solution 1 (US Customary System):

1. Average daily water consumption at (a) 100 gpcd and (b) 150 gpcd for the 10,000 people community and the 400,000 people community, respectively:
   (i) $10{,}000 \times 100/1{,}000{,}000 =$ **1.0 MGD**.
   (ii) $400{,}000 \times 150/1{,}000{,}000 =$ **60 MGD**.
2. Maximum daily water consumption is 50% greater than the average:
   (i) $1.0 \times 1.5 = 1.5$ MGD $= 1.5 \times 1{,}000{,}000/(7.5 \times 24 \times 60 \times 60) = \mathbf{2.32\ ft^3/s}$.
   (ii) $60 \times 1.5 = 90$ MGD $= 90 \times 1{,}000{,}000/(7.5 \times 24 \times 60 \times 60) = \mathbf{139\ ft^3/s}$.
3. Diameter of circular conduit flowing at 4 ft/s:
   (i) Cross-sectional area $A = Q/v = 2.32/4 = \pi D^2/4 = 0.785\ D^2$.
   Diameter $D = 0.833$ ft = 10 in. for the small 10,000 people community.
   (ii) Cross-sectional area $A = Q/v = 139/4 = \pi D^2/4 = 0.785\ D^2$.
   Diameter $D = 6.667$ ft = **80 in**. for the large 400,000 people community.

Solution 2 (SI System):

1. Average daily water consumption = 378.5 Lpcd for the 10,000 people community and average daily water consumption = 567.8 Lpcd for the 400,000 people community.
   (i) $10{,}000 \times 378.5/1{,}000{,}000 =$ **3.785 MLD**.
   (ii) $400{,}000 \times 567.8/1{,}000{,}000 =$ **227.1 MLD**.
2. Maximum daily water consumption is 50% greater than the average:
   (i) $(3.785\ \text{MLD}) \times 1.5 = 5.6775\ \text{MLD} = 5677.5\ \text{m}^3/\text{d} = (5677.5\ \text{m}^3)/(1{,}440 \times 60)\ \text{s} = 0.066\ \text{m}^3/\text{s} = \mathbf{66\ L/s}$.
   (ii) $(227.1\ \text{MLD}) \times 1.5 = 340.65\ \text{MLD} = 340{,}650\ \text{m}^3/\text{d} = (340{,}650\ \text{m}^3)/(1{,}440 \times 60)\text{s} = \mathbf{3.94\ m^3/s}$.
3. Diameter of circular conduit flowing at 1.22 m/s:
   (i) Cross-sectional area $A = Q/v = (0.066\ \text{m}^3/\text{s})/(1.22\ \text{m/s}) = 0.054\ \text{m}^2 = 0.785\ D^2$.
   Diameter $D = 0.26$ m = 260 mm for the small 10,000 people community.
   (ii) Cross-sectional area $A = Q/v = (3.94\ \text{m}^3/\text{s})/(1.22\ \text{m/s}) = 3.23\ \text{m}^2 = 0.785\ D^2$.
   Diameter $D = 2.03$ m = **2,030 mm** for the large 400,000 people community.

1.9.1 High and Low Services

Sections of the community too high to be supplied directly from the principal, or *low-service*, works are generally incorporated into separate distribution systems with independent piping and service storage. The resulting *high services* are normally fed by pumps that take water from the main supply and boost its pressure as required. Areas varying widely in elevation may be formed into intermediate districts or zones. Gated connections between the different systems are opened by hand during emergencies or go into operation automatically by means of pressure-regulating valves.

1.9.2 Fire Supplies

Before the days of high-capacity, high-pressure, motorized fire engines, conflagrations in the congested central, or *high-value*, district of some large cities were fought through independent high-pressure systems of pipes and hydrants. Taking water from the public supply and boosting its pressure by pumps in power stations whenever an alarm was rung in, these systems performed well. For extreme emergencies, rigorously protected connections usually led to independent sources of water: rivers, lakes, or tidal estuaries. Large industrial establishments, with heavy investments in plant, equipment, raw materials, and finished products, concentrated in

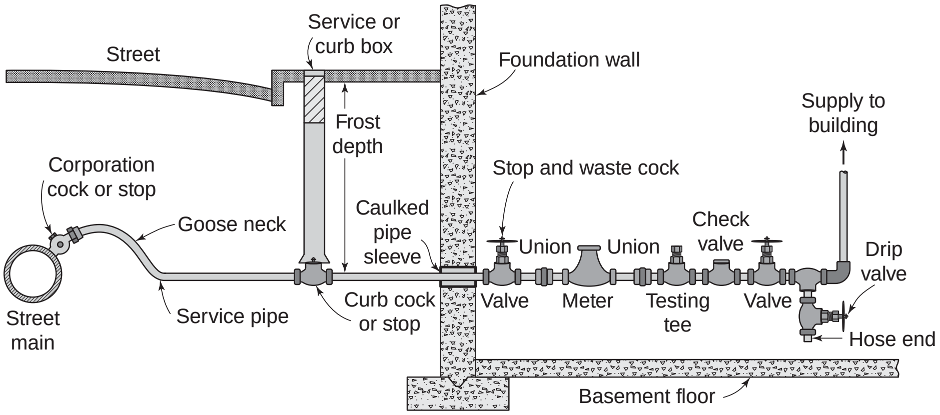


Figure 1.10 Service pipe, fittings, and accessories. There are many possible modifications, both inside and outside the building. In many instances, the meter is conveniently placed in a vault outside the building.

a small area, are generally equipped with high-pressure fire supplies and distribution networks of their own. Because such supplies may be drawn from sources of questionable quality, some regulatory agencies enforce rigid separation of private fire supplies and public systems. Others prescribe protected cross-connections that are regularly inspected for tightness. Ground-level storage and pumping are less advantageous.

1.9.3 Pressures

In normal municipal practice, pressures of 60–75 psig (416–520 kPa) are maintained in business blocks and 40 psig (278 kPa) in residential areas. Higher pressures, such as 100 psig (694 kPa) or more, delivering adequate amounts of water for firefighting through hoses attached directly to fire *hydrants* are no longer important. Instead, modern motor pumpers can discharge thousands of gallons per minute at even greater pressures. Moreover, low operating pressures make for low *leakage* from mains and reduce the amount of water that is *unaccounted* for. To supply their upper stories, tall buildings boost water to tanks at various elevations and on their roofs or in towers. In individual industrial complexes, the water pressure may be raised during fires by fixed installations of fire pumps.

1.9.4 Capacity

The capacity of distribution systems is dictated by domestic, industrial, and other normal water uses and by the *standby* or *ready-to-serve* requirements for firefighting. Pipes should be able to carry the maximum *coincident* draft at velocities that do not produce high pressure drops and water hammer. Velocities of 2–4 ft/s (0.60–1.2 m/s) and minimum pipe diameters of 6 in. (150 mm) are common in North American municipalities.

1.9.5 Service to Premises

Water reaches individual premises from the street main through one or more service pipes tapping the distribution system. The building supply between the public main and the take-offs to the various plumbing fixtures or other points of water use is illustrated in Fig. 1.10. Small services are made of cement-lined iron or steel, brass of varying copper content, admiralty metal, copper, and plastics such as polyethylene (PE), high-density polyethylene (HDPE), or polyvinyl chloride (PVC). Because lead and lead-lined pipes may corrode and release lead to the water, they are no longer installed afresh. For large services, coated or lined CIP is often employed. For dwellings and similar buildings, the minimum desirable size of service is $^3/_4$ in. (19 mm). *Pipe-tapping machines* connect services to the main without shutting off the water. They also make larger connections within water distribution systems.

1.10 WATER SYSTEMS MANAGEMENT

Construction of water supplies from the ground up, or their improvement and extension, progresses from preliminary investigations or planning through financing, design, and construction to operation, maintenance, and repair. Political and financial procedures are involved as well as engineering.

1.10.1 Municipal Supplies

The cost of public water supplies in the United States provides the reader with some concept of the magnitude of engineering activity and responsibility associated with their design and construction. Per capita investment in physical plant depends on many factors: nature, proximity, and abundance of suitable water sources; need for water treatment; availability and price of labor and materials; size and construction conditions of the system; habits of the people; and characteristics of the areas served. Wide differences in these factors make for much variation in initial costs. For communities in excess of 10,000 population, replacement costs in North America lie in the vicinity of $1,500 per capita (for 2008 price levels; for other years multiply by the ratio of an applicable *utilities price index*; see Appendix 16), with much of the investment in small communities chargeable to fire protection.

EXAMPLE 1.6 ESTIMATION OF WATERWORKS COST

Roughly, what is the replacement cost of a conventional filtration plant and other waterworks for a city of 100,000 people and what is the average plant flow?

The following conditions are assumed:

a. A per capita first cost of \$1,500 in 2008.

b. Thirty percent of the first cost is to be invested in the collection works, 10% in the purification works, and 60% in the distribution works.

c. A water consumption rate of 150 gpcd (568 Lpcd) for the city.

Solution 1 (US Customary System):

1. Assuming a per capita cost of \$1,500, the total first cost is 1,500 × 100,000 = **\$150,000,000**.
2. Assuming that 30% of this amount is invested in the collection works, 10% in the purification works, and 60% in the distribution works, the breakdown is as follows:
   - Collection works 0.3 × 150,000,000 = **\$45,000,000**.
   - Purification works 0.10 × 150,000,000 = **\$15,000,000**.
   - Distribution works 0.60 × 150,000,000 = **\$90,000,000**.
3. Assuming a water consumption rate of 150 gpcd, the total water consumption of the city is 150 × 100,000 gpd = **15 MGD**.

Solution 2 (SI System):

1. \$1,500 × 100,000 = **\$150,000,000**.
2. Same as Solution 1.
   - Collection works 0.3 × 150,000,000 = **\$45,000,000**.
   - Purification works 0.1 × 150,000,000 = **\$15,000,000**.
   - Distribution works 0.6 × 150,000,000 = **\$75,000,000**.
3. Assuming a water consumption of 568 Lpcd, the total water consumption of the city is 568 × 100,000 Lpd = **56.8 MLD**.

Of the various system components, collection and transportation works cost about a fourth, distribution works slightly less than a half, purification and pumping works about a tenth, and service lines and meters nearly a sixth of the total. The initial cost of conventional water filtration plants is about \$1,500,000 per MGD (\$396,000 per MLD) capacity, varying with plant size as the two-thirds power of the capacity. The cost of water treatment, excluding fixed charges, lies in the vicinity of \$420 per MG (\$111 per ML), varying with plant output capacity inversely as the two-fifths power of the daily production. Including interest and depreciation as well as charges against operation and maintenance, water costs \$300 to \$3,000 per million gallons (\$80 to \$800 per million liters) and is charged for accordingly. As one of our most prized commodities, water is nevertheless remarkably cheap—as low as 12 cents a ton delivered to the premises of large consumers and as little as 24 cents a ton to the taps of small consumers.

1.10.2 Individual Small Supplies

The term *individual* describes those situations in which the needs and amenities of water supply and wastewater disposal are normally satisfied by relatively small and compact systems individually owned, developed, operated, and kept within the property lines of the owner. Normally, this implies construction of wanted or required systems through individual rather than community effort. But there have been developments for villages and communities with scattered buildings in which local government has taken the initiative and assumed responsibility for construction and care of individualized systems. Property owners, as well as the community, then enjoy the benefits of adequate planning, design, construction, management, and supervision. Otherwise, unfortunately, necessary works are rarely designed by qualified engineers and often end up not satisfying their purposes, both in a sanitary and an economic sense.

Reasonably good results can be obtained if (a) engineering departments of central health authorities publish manuals of design, construction, and operation that fit local conditions and (b) they give needed advice and supervision as well as provide for regulation. Nevertheless, villages and fringe areas are best served, in the long run, by the extension of central water lines and sewers or by incorporation of *water* and *sewer districts* comprising more than a single unit of local government.

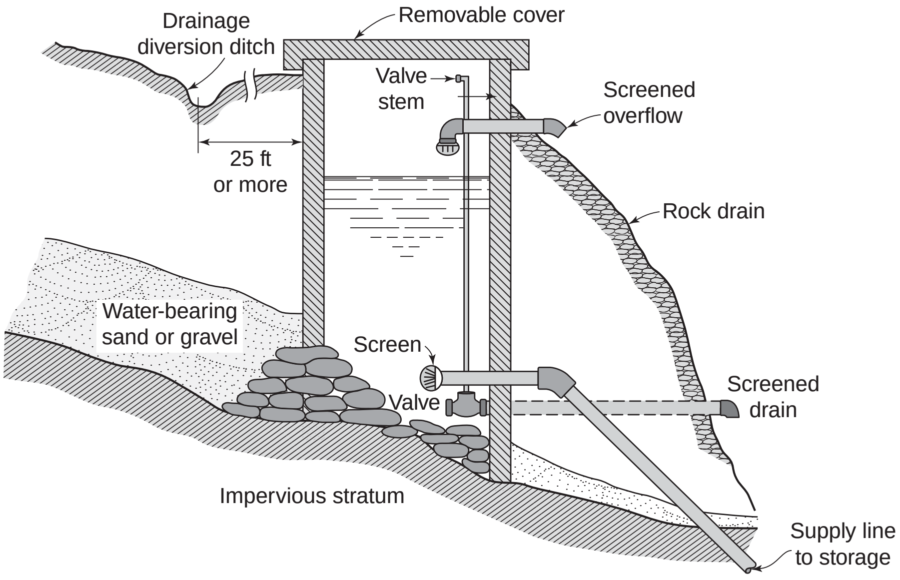


Figure 1.11 Water supply from spring (After US Public Health Service). Conversion factor: 1 ft = 0.3048 m.

1.11 INDIVIDUAL WATER SYSTEMS

Because of the natural purifying capacity and protection of the soil, individual and *rural* water supplies are generally drawn from springs, infiltration galleries, and wells. Where groundwater is highly mineralized or unavailable, rainwater is next best in general safety and quality. Only in uninhabited and well-protected upland areas should ponds and streams be tapped without purifying the waters drawn.

Some of the safeguards for groundwater works are illustrated in Figs. 1.11, 1.12, and 1.13. They share the following features in common:

1. Diversion of surface water from intake structures
2. Drainage of overflow or spillage waters away from intake structures
3. Water tightness of intake works for at least 10 ft (3 m) below the ground surface and, if necessary, until the aquifer is reached
4. Prevention of backflow into intakes; where there is no electric power, water is pumped by hand, wind, water, or gasoline engines

Individual and rural water supplies are not without their purification problems. Gravity and pressure filters are employed to improve waters of doubtful purity, and zeolite softeners and other ion-exchange units are used for the removal of unwanted hardness. Iron-bearing groundwaters that issue from their source sparklingly clear but become rusty on exposure to air (by oxidation and precipitation of iron) are best treated in manganese cation exchange units. Hexametaphosphates may keep iron from precipitating, but this requires skillful management. It may be advisable to seek an iron-free source instead.

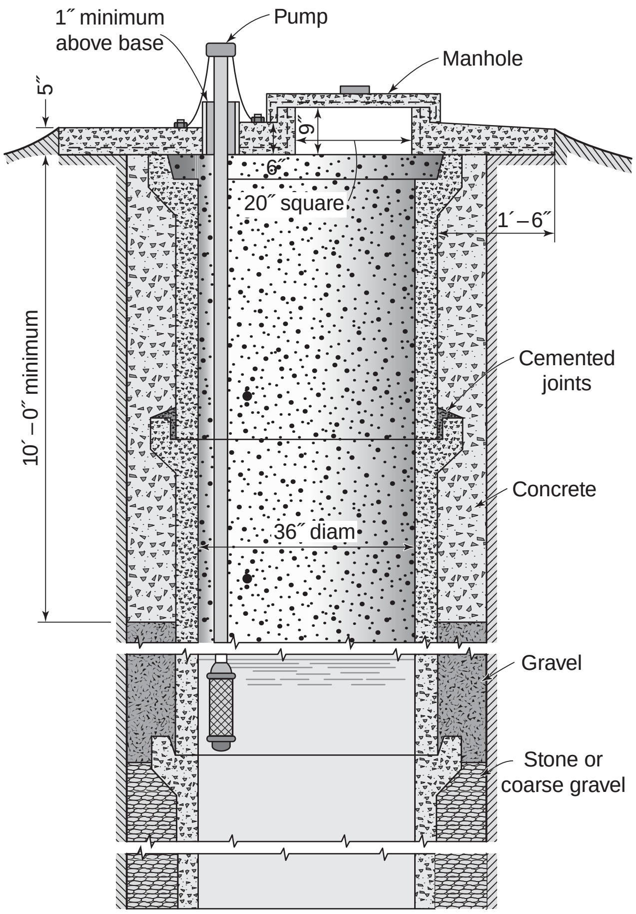


Figure 1.12 Water supply from dug well (After US Department of Agriculture). Conversion factors: 1′ = 1 ft = 0.3048 m; 1″ = 1 in. = 2.54 cm.

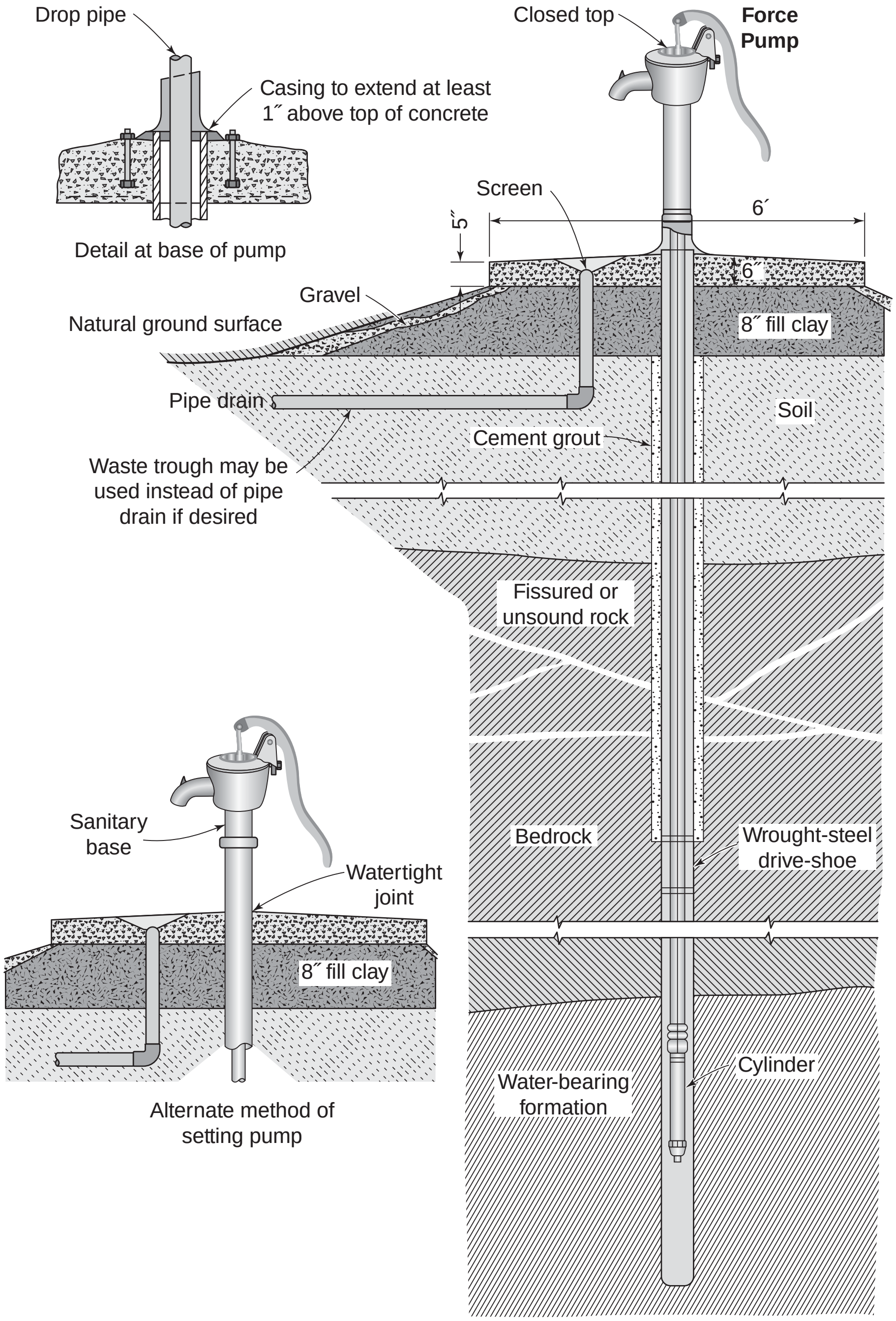


Figure 1.13 Water supply from driven well (After Virginia State Department of Health). Conversion factors: 1′ = 1 ft = 0.3048 m; 1″ = 1 in. = 2.54 cm.

Some soft groundwaters containing much carbon dioxide are highly corrosive. Passage through marble or limestone chips takes calcium into solution and reduces the carbon dioxide proportionately. Hardness is increased, but corrosiveness is decreased. For the chlorination of polluted rural supplies, there are solution-feed dosing devices that proportion the amount of added chlorine to flow. Instead, the householder may prefer to boil his drinking and culinary water. Investment in an inherently safe and satisfactory supply, however, is usually wisest in the long run.

PROBLEMS/QUESTIONS

1.1 What is the stream flow in MGD (MLD) for a catchment area of 80 mi^2 (207.2 km^2) where rainfall rate is 45 in./year (114.3 cm/year) and evaporation rate is 20 in./year (50.8 cm/year)?

1.2 A city is served by a raw water reservoir that has a water surface of 5.8 mi^2 (15.02 km^2) and an average effective depth of 18 ft (5.49 m). Determine the water storage volume.

1.3 The population of a city is 400,000, and the average daily per capita water demand is 150 gpcd (567.75 Lpcd). Determine the city's average daily water demand.

1.4 How many days of draft can a raw water reservoir support for a city of 400,000 people? The reservoir has a water surface of 5.8 mile^2 = 15.02 km^2 and an average effective depth of 18 ft = 5.49 m.

1.5 What percentage of mean annual runoff is to be consumed by a city of 400,000 people in an area with (a) rainfall rate = 45 in./year = 114.3 cm/year; (b) evaporation rate = 20 in./year = 50.8 cm/year; and (c) watershed catchment or drainage area = 80 mile^2 = 207.2 km^2?

1.6 Determine the net yield and water storage volume of a rainwater system assuming that (a) the net yield of a rain collection facility approximates two-thirds of its gross yield; (b) the mean annual rainfall = 25 in./year = 63.5 cm/year; (c) the mean annual evaporation rate is 8 in./year = 20.32 cm/year; (d) the rain collection roof area equals 3,200 ft^2 = 297.28 m^2; and (e) water storage volume equals 50% of annual net yield.

1.7 Determine the storage volume of a new raw water reservoir in accordance with the following given technical information: (a) city population = 400,000; (b) water consumption = 150 gpcd = 568 Lpcd; (c) watershed or catchment area = 80 mi^2 = 207.2 km^2; (d) rainfall rate = 45 in./year = 114.3 cm/year; (e) evaporation rate = 20 in./year = 50.8 cm/year; (f) minimum reservoir volume = 50% annual net yield or half of a year's water supply, whichever is greater; and (g) 75% water resources development.

1.8 Determine the number of people who can be sustainably supported by a watershed under the following conditions: (a) watershed area = 80 mi^2 = 207.2 km^2; (b) annual rainfall rate = 45 in./year = 114.3 cm/year; (c) annual evaporation rate = 20 in./year = 50.8 cm/year; (d) water resources development = 75%; (e) raw water reservoir volume to store 50% net annual yield or provide half of a year's water supply, whichever is higher = 13 BG = 49.205 BL; and (f) water consumption rate = 150 gpcd = 568 Lpcd.

1.9 Determine the number of people who can be adequately supported by a watershed under the following conditions: (a) watershed area = 80 mi^2 = 207.2 km^2; (b) water supply system with no reservoir for water storage; (c) low water flow = 0.1 ft^3/s = 64,600 gpd/mi^2 = 2.83 L/s = 0.00283 m^3/s; and (d) water consumption rate = 150 gpcd = 568 Lpcd.

1.10 Make a rough estimate of the groundwater movement velocity ($v = Q/A$) (ft/day or m/day) if (a) all of the groundwater laterally within 400 ft (122 m) of the well comes fully within its influence; and (b) the yield of the aquifer is 258 gpm (gallon per minute) = 976.53 L/min = 16.28 L/s; and the aquifer through which the groundwater moves is 25 ft (7.62 m) deep.

1.11 Estimate the surface area (ft^2, or m^2) of a slow sand filter that is to deliver water to a village of 1,000 people assuming that (a) the average daily water demand = 100 gpcd = 378.5 Lpcd; (b) the slow sand filter's filtration rate is 3 million gallons per acre per day (MGAD) = 3 MGD/acre = 28.08 MLD/ha = 2,808 MLD/km^2; and (c) two slow sand filters are required. Each filter is able to treat the full water flow and one of the two filters is a standby unit.

1.12 Estimate roughly the size of a water supply pipe leading to a water distributing reservoir serving a small village of 2000 people assuming that (a) the water consumption rate is 100 gpcd = 378.5 Lpcd and (b) water velocity in the pipe = 3 ft/s = 0.91 m/s.

1.13 Determine the diameter of a water main to serve a residential area, assuming (a) an average water demand of 150 gpcd = 568 Lpcd; (b) population = 30,000; (c) fire flow requirement = 500 gpm = 2,082 L/min = 32 L/s; and (d) recommended water velocity = 3.5 ft/s = 1.07 m/s.

1.14 Roughly, what is the replacement cost of the waterworks of a city of 10,000 people?

1.15 Define the technical terms of (a) water supply system; (b) community water system; (c) nontransient noncommunity water system; d) transient noncommunity water system; and (e) watershed.

1.16 Define the technical terms of (a) collection works or systems; (b) purification works; (c) transmission works or systems; (d) distribution works or systems; and (e) impounding dams.

1.17 Define the technical terms of (a) aquifer; (b) water table; (c) infiltration galleries; (d) well; and (e) artesian.

1.18 Define the technical terms of (a) mixing basin or tank; (b) flocculation or reaction basins; (c) sedimentation basins; (d) slow sand filtration; and (e) rapid sand filtration.

1.19 Define the technical terms of (a) evaporation; (b) evapotranspiration; (c) aeration; (d) desalination; and (e) reverse osmosis.

1.20 Define the technical terms of (a) runoff; (b) fresh water; (c) saline and brackish waters; (d) precipitation; and (e) service reservoir or distribution reservoir.

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Chapter 2

Water Sources: Surface Water

Figure 2.1 shows where water is and how it is distributed on Earth. The bar on the left shows where the water on Earth exists; about 97% of all water is in the oceans. The middle bar shows the distribution of the rest 3% of all Earth's water, which is freshwater. The majority, about 69%, is locked up in glaciers and ice caps, mainly in Greenland and Antarctica. You might be surprised that of the remaining freshwater, almost all of it is below our feet, as groundwater. No matter where on Earth one is standing, chances are that, at some depth, the ground below is saturated with water. Of all the freshwater on Earth, only about 0.3% is contained in rivers and lakes—yet rivers and lakes are not only the water we are most familiar with, but also where most of the water we use in our everyday lives exists.

For a detailed explanation of where Earth's water is, look at the data in Table 2.1. Notice how of the world's total water supply of about 326 million mi^3 (1360 million km^3), more than 97% is saline. Also, of the total freshwater, about 7 million mi^3 (29 million km^3) is locked up in ice and glaciers. Another 30% of freshwater is in the ground. Thus, surface-water sources only constitute about 30,300 mi^3 (126,300 km^3), which is about 0.009% of total water.

What interconnects groundwater and surface water is the *water cycle* (Fig. 2.2). The water cycle has no starting point, but we will begin in the oceans, because that is where most of Earth's water exists. The sun, which drives the water cycle, heats water in the oceans. Some of it evaporates as vapor into the air. Ice and snow can sublimate directly into water vapor. Rising air currents take the vapor up into the atmosphere, along with water from *evapotranspiration*, which is water transpired from plants and evaporated from the soil. The vapor rises into the air where cooler temperatures cause it to condense into clouds. Air currents move clouds around the globe; cloud particles collide, grow, and fall out of the sky as precipitation. Some precipitation falls as snow and can accumulate as ice caps and glaciers, which can store frozen water for thousands of years. Snowpacks in warmer climates often thaw and melt when spring arrives, and the melted water flows overland as snowmelt. Most precipitation falls back into the oceans or onto land, where, due to gravity, the precipitation flows over the ground as *surface runoff*. A portion of runoff enters rivers in valleys in the landscape, with stream flow moving water toward the oceans. Runoff and groundwater seepage accumulate and are stored as freshwater in lakes.

Not all runoff flows into rivers, though. Much of it soaks into the ground as *infiltration*. Some water infiltrates deep into the ground and replenishes *aquifers* (saturated subsurface formations), which store huge amounts of freshwater for long periods of time. Some infiltration stays close to the land surface and can seep back into surface-water bodies (and the ocean) as groundwater discharge, and some groundwater finds openings in the land surface and emerges as freshwater springs. Over time, though, all of this water keeps moving; some reenters the oceans, where the water cycle "ends"—or, where it "begins."

2.1 SOURCES OF SURFACE WATER

In the United States by far the largest volumes of municipal water are collected from surface sources. Possible yields vary directly with the size of the catchment area, or watershed, and with the difference between the amount of water falling on it and the amount lost by evapotranspiration. The significance of these relations to water supply is illustrated in Fig. 1.1. Where surface-water and groundwater sheds do not coincide, some groundwater may enter from neighboring catchment areas or escape to them.

Communities on or near streams, ponds, or lakes may withdraw their supplies by *continuous draft* if stream flow and pond or lake capacity are high enough at all seasons of the year to furnish requisite water volumes. Collecting works include ordinarily (a) an intake crib, gatehouse, or tower; (b) an intake conduit; and (c) in many places, a pumping station. On small streams serving communities of moderate size, intake or diversion dams can create a sufficient depth of water to submerge the intake pipe and protect it against ice. From intakes close to the community the water must generally be lifted to purification works and thence to the distribution system (Fig. 2.3).

Because most large streams are polluted by wastes from upstream communities and industries, their waters

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Water Engineering: Hydraulics, Distribution and Treatment, First Edition. Nazih K. Shammas and Lawrence K. Wang.

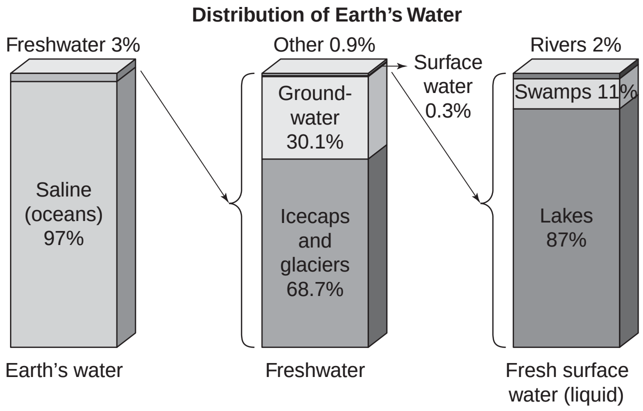

Figure 2.1 Distribution of Earth's water (Courtesy of USGS).

Table 2.1 Estimated water distribution on earth

	Surface area (mi^2)	Volume (mi^3)	% of total
Salt water			
The oceans	139,500,000	317,000,000	97.2
Inland seas and saline lakes	270,000	25,000	0.008
Freshwater			
Freshwater lakes	330,000	30,000	0.009
All rivers (average level)	—	300	0.0001
Antarctic ice cap	6,000,000	6,300,000	1.9
Arctic ice cap and glaciers	900,000	680,000	0.21
Water in the atmosphere	197,000,000	3,100	0.001
Groundwater within half a mile from surface	—	1,000,000	0.31
Deep-lying groundwater	—	1,000,000	0.31
Total (rounded)	—	326,000,000	100.00

Source: Courtesy of US Geological Survey.

Conversion factors: 1 mi = 1.609 km; 1 mi^2 = 640 acre = 2.59 km^2 = 2.59×10^6 m^2; 1 mi^3 = 4.1673 km^3.

must be purified before use. Cities on large lakes must usually guard their supplies against their own and their neighbors' wastewaters and spent industrial process waters by moving their intakes far away from shore and purifying both their water and their wastewater. Diversion of wastewaters and other plant nutrients from lakes will retard lake eutrophication.

Low stream flows are left untouched when they are wanted for other valley purposes or are too highly polluted for reasonable use. Only clean floodwaters are then diverted into reservoirs constructed in meadowlands adjacent to the stream or otherwise conveniently available. The amount of water so stored must supply demands during seasons of unavailable stream flow. If draft is confined to a quarter year, for example, the reservoir must hold at least three-fourths of a community's annual supply. In spite of its selection and long storage, the water may still have to be purified.

In search of clean water and water that can be brought and distributed to the community by gravity, engineers have developed supplies from upland streams. Most of them are tapped near their source in high and sparsely settled regions. To be of use, their annual discharge must equal or exceed the demands of the community they serve for a reasonable number of years in the future. Because their dry-season flows generally fall short of concurrent municipal requirements, their floodwaters must usually be stored in sufficient volume to ensure an adequate supply. Necessary reservoirs are impounded by throwing dams across the stream valley (Fig. 2.4). In this way, up to about 70% or 80% of the mean annual flow can be utilized. The area draining to impounded

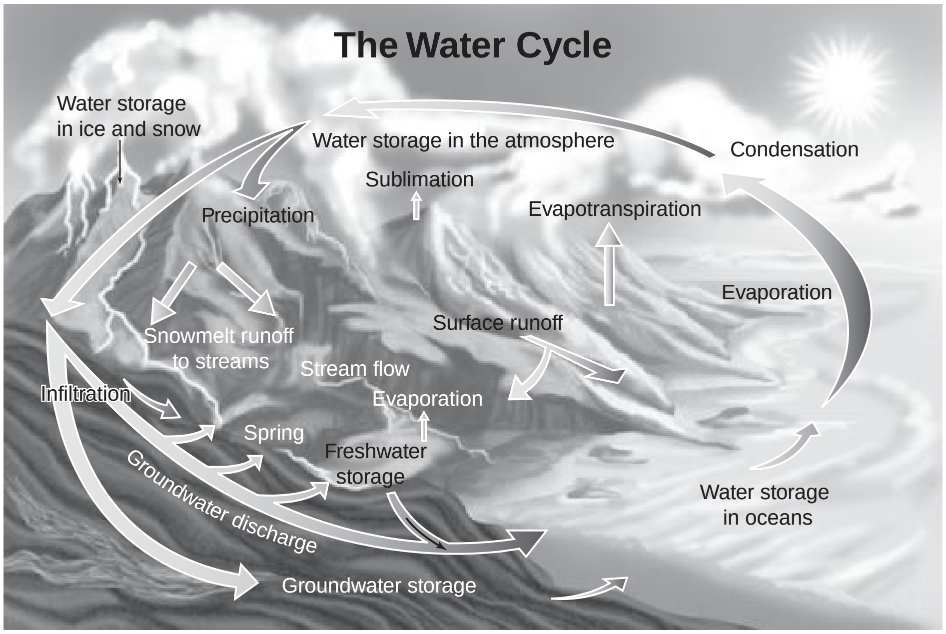

Figure 2.2 The water cycle (Courtesy of USGS).

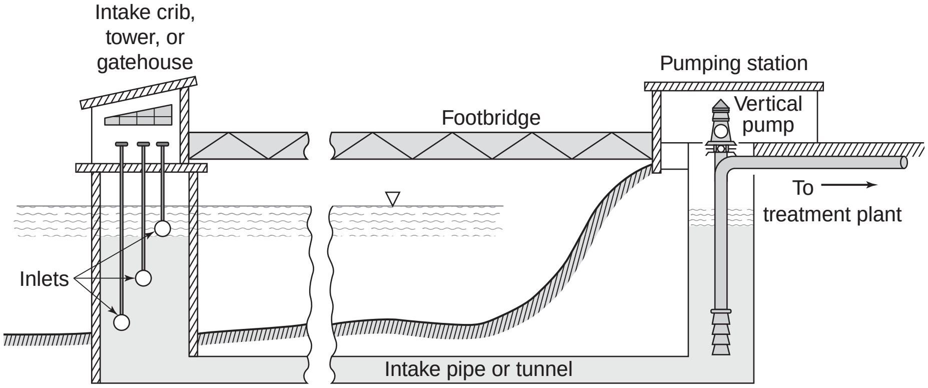

Figure 2.3 Continuous draft of water from large lakes and streams.

reservoirs is known as the *catchment area* or *watershed*. Its economical development depends on the value of water in the region, but it is a function, too, of runoff and its variation, accessibility of catchment areas, interference with existing water rights, and costs of construction. Allowances must be made for evaporation from new water surfaces generated by the impoundage and often, too, for release of agreed-on flows to the valley below the dam (compensating water). Increased ground storage in the flooded area and the gradual diminution of reservoir volumes by siltation must also be considered.

Intake structures are incorporated in impounding dams or kept separate. Other important components of impounding reservoirs are (a) spillways (Fig. 2.5) safely passing floods in excess of reservoir capacity and (b) diversion conduits safely carrying the stream past the construction site until the reservoir has been completed and its spillway can go into action. Analysis of flood records enters into the design of these ancillary structures. Some impounded supplies are sufficiently safe, attractive, and palatable to be used without treatment other than protective disinfection. However, it may be necessary to remove (a) high *color* imparted to the stored water by the decomposition of organic matter in swamps and on the flooded valley floor; (b) *odors and tastes* generated in the decomposition or growth of algae, especially during the first years after filling; and (c) *turbidity* (finely divided clay or silt) carried into streams or reservoirs by surface wash, wave action, or bank erosion. Recreational uses of watersheds and reservoirs may endanger the water's safety and call for treatment of the flows withdrawn from storage.

Much of the water entering streams, ponds, lakes, and reservoirs in times of drought, or when precipitation is frozen, is seepage from the soil. Nevertheless, it is classified as surface runoff rather than groundwater. Water seeps *from* the ground when surface streams are low and *to* the ground when surface streams are high. Release of water from ground storage or from accumulations of snow in high mountains is a determining factor in the yield of some catchment areas. Although surface waters are derived ultimately from precipitation, the relations between precipitation, runoff, infiltration, evaporation, and transpiration are so complex that engineers rightly prefer to base calculations of *yield* on available *stream gaugings*. For adequate information, gaugings must extend over a considerable number of years.

In the absence of adequate natural storage, engineers construct impounding reservoirs (Fig. 2.6). More rarely they excavate storage basins in lowlands adjacent to streams. Natural storage, too, can be regulated. Control works (gates and weirs or sills) at the outlets to lakes and ponds are examples. Some storage works are designed to serve a single purpose

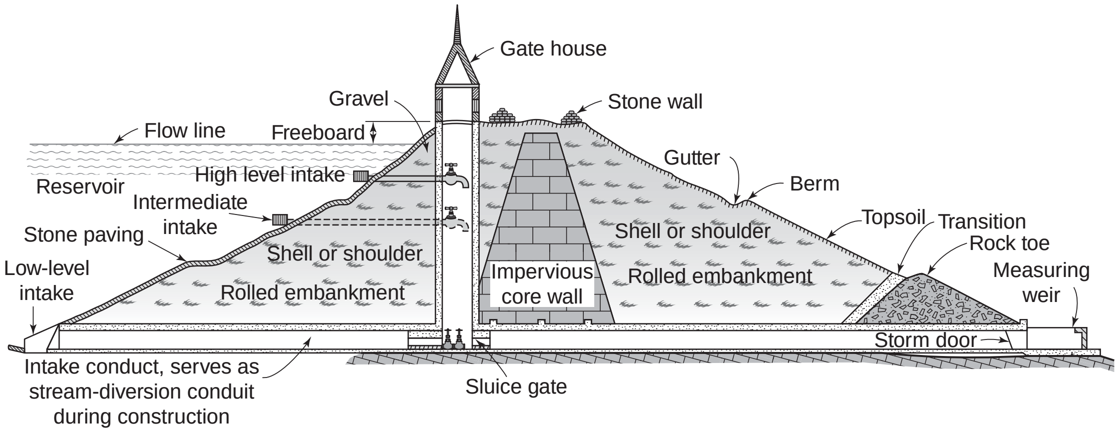

Figure 2.4 Dam and intake towers for an impounded surface-water supply.

Figure 2.5 Concrete spillway in dam (Courtesy of the Department of Water Supply, Maui County, HI).

only; others are planned to perform a number of different functions and to preserve the broader economy of natural resources. Common purposes include

1. Water supply for household, farm, community, and industry
2. Dilution and natural purification of municipal and industrial wastewaters
3. Irrigation of arable land
4. Harnessing water power
5. Low-water regulation for navigation
6. Preservation and cultivation of useful aquatic life
7. Recreation, for example, fishing, boating, and bathing
8. Control of destructive floods

The greatest net benefit may accrue from a judicious combination of reservoir functions in *multipurpose* developments. The choice of single-purpose storage systems should indeed be justified fully.

Figure 2.6 Hoover dam, Clark County, NV (Courtesy of the National Resources Conservation Service and USDA).

Storage is provided when stream flow is inadequate or rendered unsatisfactory by heavy pollution. Release of stored waters then swells flows and dilutes pollution. Storage itself also affects the quality of the waters impounded. Both desirable and undesirable changes may take place. Their identification is the responsibility of *limnology*, the science of lakes or, more broadly, of inland waters.

If they must receive wastewaters, stream flows should be adjusted to the pollution load imposed on them. Low-water regulation, as such, is made possible by headwater or upstream storage, but lowland reservoirs, too, may aid dilution and play an active part in the natural purification of river systems. Whether overall results are helpful depends on the volume and nature of wastewater flows and the chosen regimen of the stream.

2.2 SAFE YIELD OF STREAMS

In the absence of storage, the safe yield of a river system is its lowest *dry-weather flow*; with full development of storage, the safe yield approaches the *mean annual flow*. The economical yield generally lies somewhere in between. The attainable yield is modified by (a) evaporation, (b) bank storage, (c) seepage out of the catchment area, and (d) silting. Storage–yield relations are illustrated in this chapter by calculations of storage to be provided in impounding reservoirs for water supply. However, the principles demonstrated are also applicable to other purposes and uses of storage.

2.3 STORAGE AS A FUNCTION OF DRAFT AND RUNOFF

A dam thrown across a river valley impounds the waters of the valley. Once the reservoir has filled, the water drawn from storage is eventually replenished by the stream, provided runoff, storage, and draft are kept in proper balance. The balance is struck graphically or analytically on the basis of historical records or replications generated by suitable statistical procedures of operational hydrology.

Assuming that the reservoir is full at the beginning of a dry period, the maximum amount of water S (MG/mi^2 or ML/km^2) that must be withdrawn from storage to maintain a given average draft D (MG/mi^2 or ML/km^2) equals the maximum cumulative difference between the draft D and the runoff Q (MG/mi^2 or ML/km^2) in a given dry period, or

$$S = \text{maximum value of } \sum (D - Q) \tag{2.1}$$

To find S, $\sum(D - Q)$ is summed arithmetically or graphically. The *mass diagram method* illustrated in Fig. 2.7 is a useful demonstration of finding $\sum(D - Q) = \sum D - \sum Q$. The shorter the interval of time for which runoff is recorded, the more exact the result. As the maximum value is approached, therefore, it may be worthwhile to shift to short

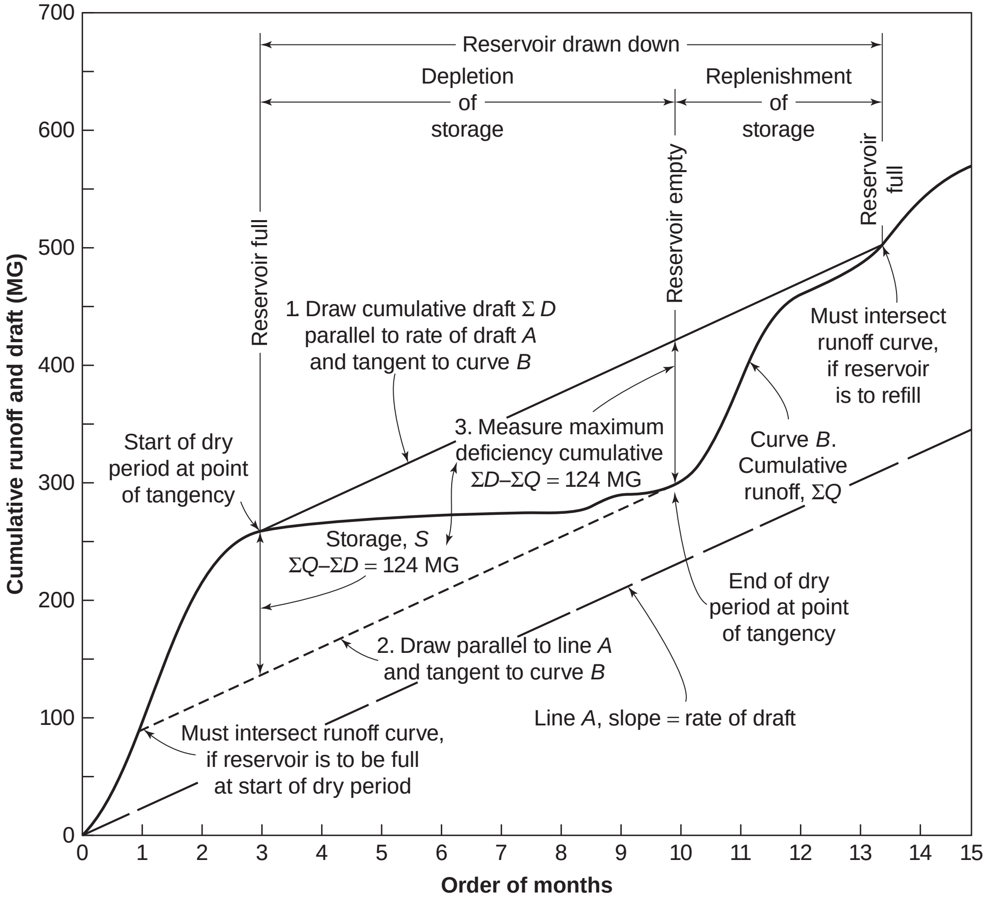

Figure 2.7 Mass diagram method for the determination of storage required in impounding reservoirs. (A constant draft of 750,000 gpd/mi^2 = 23 MG/mi^2 for a month of 30.4 days is assumed.) Conversion factors: 1 MG = 1,000,000 gal = 3.785 ML = 3,785,000 L; 1 MG/mi^2 = 1.461 ML/km^2; 1 gpd/mi^2 = 1.461 L/d/km^2.

intervals of time—from monthly to daily values, for example. The additional storage identified by such a shift may be as much as 10 days of draft.

Assuming that inflow and drafts are repeated cyclically, in successive sets of T years, a procedure called *sequent peak* was developed for determining minimum storage for no shortage in draft based on two needed cycles. Example 2.1 illustrates the procedure.

For variable drafts and inclusion of varying allowances for evaporation from the water surface created by the impoundage, the analytical method possesses distinct advantages over the graphical method. The principal value of the mass diagram method, indeed, is not for the estimation of storage requirements, but for determining the yield of catchment areas on which storage reservoirs are already established.

2.4 DESIGN STORAGE

Except for occasional series of dry years and very high density housing and industrial developments, *seasonal storage* generally suffices in the well-watered regions of the United States. Water is plentiful, stream flows do not vary greatly from year to year, reservoirs generally refill within the annual hydrologic cycle, and it does not pay to go in for advanced or complete development of catchment areas. In semiarid regions, on the other hand, water is scarce, stream flows fluctuate widely from year to year, runoff of wet years must be conserved for use during dry years, and it pays to store a large proportion of the mean annual flow. In these circumstances, operational records of adequate length become important, along with computational aids.

Given a series of storage values for the flows observed or generated statistically, the engineer must decide which value he will use. Will it be the highest on record, or the second, third, or fourth highest? Obviously, the choice depends on the degree of protection to be afforded against water shortage. This must also be considered in terms of drought experience, which is a function of the length of record examined. To arrive at a reasonable answer and an economically justifiable storage design, the engineer may resort to (a) a *statistical analysis* of the arrayed storage values and (b) estimates of

EXAMPLE 2.1 CALCULATION OF REQUIRED STORAGE FROM RUNOFF RECORDS

From the recorded monthly mean runoff values shown in column 2 of Table 2.2, find the required storage for the estimated rates of draft listed in column 3 of Table 2.2.

Table 2.2 Calculation of required storage (Example 2.1)[a]

Order of months (1)	Recorded runoff, Q (2)	Estimated draft, D (3)	$Q - D$ (4)	$\sum(Q - D)$ (5)	Storage $S = P - T^b$ (6)	Waste, W (7)	Reservoir state[c] (8)
1	94	27	+67	67	67	0	*R*
2	122	27	+95	162	162	0	*R*
3	45	30	+15	177P_1	176	1	*L*
4	5	30	−25	152	151	0	*F*
5	5	33	−28	124	123	0	*F*
6	2	30	−28	96	95	0	*F*
7	0	27	−27	69	68	0	*F*
8	2	27	−25	44	43	0	*F*
9	16	30	−14	30	29	0	*F*
10	7	36	−29	1T_1	0	0	*E*
11	72	33	+39	40	39	0	*R*
12	92	30	+62	102	101	0	*R*
1	94	27	+67	169	168	0	*R*
2	122	27	+95	264	176	87	*L*
3	45	30	+15	279 P_2	176	15	*L*
4	5	30	−25	254	151	0	*F*
5	5	33	−28	226	123	0	*F*
6	2	30	−28	198	95	0	*F*
7	0	27	−27	171	68	0	*F*
8	2	27	−25	146	43	0	*F*
9	16	30	−14	132	29	0	*F*
10	7	36	−29	103T_2	0	0	*E*
11	72	33	+39	142	39	0	*R*
12	92	30	+62	204P_3	101	0	*R*

Conversion factor: 1 MG/mi^2 = 1.461 ML/km^2.

[a] Runoff Q, draft D, and storage S are expressed in MG/mi^2.

[b] P, peak; T, trough.

[c] R, rising; F, falling; L, spilling; E, empty.

Solution 1 (US Customary System):

Conversion factors: 1 MG/mi^2 = 1.461 ML/km^2; 1 MGD/mi^2 = 1.461 MLD/km^2.

Column 2: These are observed flows for the Westfield Little River, near Springfield, MA.

Column 3: The values 27, 30, 33, and 36 MG/mi^2 = 0.89, 1.1, 1.09, and 1.18 MGD/mi^2, respectively, for 30.4 days/month. For a total flow of 462 MG/mi^2 in 12 months, the average flow is $462/365 = 1.27$ MGD/mi^2, and for a total draft of 360 MG/mi^2 the development is $100 \times 360/462 = 78\%$.

Column 4: Positive values are surpluses; negative values, deficiencies.

Column 5: P_1 is the first peak and T_1 is the first trough in the range P_1P_2, where P_2 is the second higher peak; similarly T_2 is the second trough in the range P_2P_3.

Column 6: The required maximum storage $S_\mathrm{m} = \max(P_j - T_j) = P_\mathrm{m} - T_\mathrm{m} = \boldsymbol{P_1 - T_1} = 177 - 1 =$ **176 MG/mi^2** in this case. **The fact that $\boldsymbol{P_2 - T_2}$ = 279 − 103 = 176 MG/mi^2 also implies that there is seasonal rather than over-year storage**. Storage at the end of month i is $S_i = \min\{S_\mathrm{m}, [S_{i-1} + (Q_i - D_j)]\}$; for example, in line 2, $S_\mathrm{m} - 176$ and $[S_{i-1} + (Q_i - D_j)] = 67 + 95 = 162$, or $S_i = 162$; in line 3, however, $S_\mathrm{m} = 176$ and $[S_{i-1} + (Q_i - D_j)] = 162 + 95 = 257$ or $S_i = S_\mathrm{m} = 176$.

Column 7: The flow wasted $W_i = \max\{0, [(Q_i - D_i) - (S_\mathrm{m} - S_{i-1})]\}$; for example, line 3, $(Q_i - D_i) - (S_\mathrm{m} - S_{i-1}) = 15 - (176 - 162) = 1$ or $W_i = 1$; in line 3 of the second series, however, $(Q_i - D_i) - (S_\mathrm{m} - S_{i-1}) = 15 - (176 - 176) = 15$. There is no negative waste.

Solution 2 (SI System):

An SI or metric system solution can be obtained using these conversation factors: 1 MG = 3785 m^3 = 3.785 ML; 1 ML = 1000 m^3; 1 MGD = 3785 m^3/day = 3.785 MLD = 0.0438 m^3/s = 43.8 L/s; 1 mi^2 = 2.59 km^2; 1 MG/mi^2 = 1.461 ML/km^2; 1 MGD/mi^2 = 1.461 MLD/km^2.

the difficulties and costs associated with shortage in supply. Storage values equaled or exceeded but once in 20, 50, or 100 years, that is, 5%, 2%, and 1% of the years, are often considered. For water supply, Hazen (1956) suggested employing the 5% value in ordinary circumstances. In other words, design storage should be adequate to compensate for a drought of a severity not expected to occur more often than once in 20 years. In still drier years, it may be necessary to *curtail the use of water* by limiting or prohibiting, for example, lawn sprinkling and car washing.

Restricting water use is irksome to the public and a poor way to run a public utility. As a practical matter, moreover, use must be cut down well in advance of anticipated exhaustion of the supply. It would seem logical to consider not only the frequency of curtailment but also the depletion point at which conservation should begin. In practice, the iron ration generally lies between 20% and 50% of the total water stored. Requiring a 25% reserve for the drought that occurs about once in 20 years is reasonable. An alternative is a storage allowance for the drought to be expected once in 100 years. This is slightly less in magnitude than the combination of a 25% reserve with a once-in-20-years risk.

In undeveloped areas, few records are even as long as 20 years. Thus, estimation of the 5%, 2%, and 1% frequencies, or of recurrence intervals of 20, 50, and 100 years, requires extrapolation from available data. *Probability plots* lend themselves well to this purpose. However, they must be used with discretion. Where severe droughts in the record extend over several years and require annual rather than seasonal storage values to be used, the resulting series of storage values becomes nonhomogeneous and is no longer strictly subject to ordinary statistical interpretations. They can be made reasonably homogeneous by including, besides all truly seasonal storage values, not only all true annual storage values, but also any seasonal storage values that would have been identified within the periods of annual storage if the drought of the preceding year or years had not been measured. Plots of recurrence intervals should include minor storage capacities as well as major ones. The results of these statistical analyses are then conveniently reduced to a set of *draft–storage–frequency* curves.

2.5 LOSS BY EVAPORATION, SEEPAGE, AND SILTING

When an impounding reservoir is filled, the hydrology of the inundated area and its immediate surroundings is changed in a number of respects: (a) the reservoir loses water by evaporation to the atmosphere and gains water by direct reception of rainfall; (b) rising and falling water levels alter the pattern of groundwater storage and movement into and out of the surrounding reservoir banks; (c) at high stages, water may seep from the reservoir through permeable soils into neighboring catchment areas and so be lost to the area of origin; and (d) quiescence encourages subsidence of settleable suspended solids and silting of the reservoir.

2.5.1 Water-Surface Response

The response of the new water surface is to establish new hydrologic equilibria (a) through loss of the runoff once coming from precipitation on the land area flooded by the reservoir Qa (closely), where Q is the areal rate of runoff of the original watershed and a is the water surface area of the reservoir; and through evaporation from the water surface Ea, where E is the areal rate of evaporation; and (b) through gain of rainfall on the water surface Ra, where R is the areal rate of rainfall. The net rate of loss or gain is $[R-(Q+E)]a$; a negative value records a net loss and a positive value a net gain.

Individual factors vary within the annual hydrologic cycle and from year to year. They can be measured. Exact calculations, however, are commonly handicapped by inadequate data on evaporation. Required hydrological information should come from local or nearby observation stations, areas of water surface being determined from contour maps of the reservoir site. The mean annual water surface as a fraction of the reservoir area at the spillway, f, is normally about 0.90 or 90%.

For convenience, the water-surface response is expressed in one of the following ways:

1. Revised runoff $Q_r = Q-(Q+E-R)(fa/A)$ **(2.2)**
2. Equivalent draft $D_e = (Q+E-R)(fa/A)$ **(2.3)**
3. Effective catchment area $A_e = A-fa[1-(R-E)/Q]$ **(2.4)**
4. Adjusted flow line $F = Q+E-R$ **(2.5)**
5. Effective draft $D_{ed} = D_{md}+D_e(A)$ **(2.6)**

where Q_r = revised runoff, in./year or cm/year; Q = mean annual runoff, in./year or cm/year; R = mean annual rainfall, in./year or cm/year; E = mean annual evaporation, in./year or cm/year; a = reservoir area, mi^2 or km^2; A = catchment area, mi^2 or km^2; f = 90% = 0.9 = effective

EXAMPLE 2.2 DESIGN OF STORAGE REQUIREMENT FOR VARIOUS FREQUENCIES

Examination of the 25-year record of runoff from an eastern stream shows that the storage amounts listed in Table 2.3 are needed in successive years to maintain a draft of 750,000 gpd/mi^2 (1,096,000 L/d/km^2). Estimate the design storage requirement that is probably reached or exceeded but once in 20, 50, and 100 years.

Table 2.3 Storage requirements (Example 2.2)

Order of year	1	2	3	4	5	6	7	8	9	10	11	12	13
Calculated storage (MG)	47	39	104	110	115	35	74	81	124	29	37	82	78
Order of year	14	15	16	17	18	19	20	21	22	23	24	25	
Calculated storage (MG)	72	10	117	51	61	8	102	65	73	20	53	88	

Conversion factors: 1 MG = 1,000,000 gal = 3.785 ML = 3,785,000 L.

Solution:

1. The 25 calculated storage values arrayed in order of magnitude are plotted on arithmetic-probability paper in Fig. 2.8 at 100 $k_i/(n+1)$ where $k_i = 1, 2, 3, \ldots 25$ and $n = 25$; $100 \times 1/26 = 3.8\%$, $100 \times 2/26 = 7.7\%$, $100 \times 3/26 = 11.5\%$, and so forth. A straight line of best fit is identified in this instance, but not necessarily others, the arithmetic mean storage being $\mu = \Sigma X_i/n = (47 + 39 + 104 + \cdots + 88)/25 =$ **67 MG** (254 ML) and the standard deviation $\sigma = [\Sigma(X_i - \mu)^2/(n-1)]^{0.5}$ where $X_i = 47, 39, \ldots 88$, $\mu = 67$ MG and $n = 25$. Then $\sigma = 33$ MG (125 ML). The straight line of best fit is plotted in Fig. 2.8 using the data points (3.8% and 8 MG), (7.7% and 10 MG), (11.5% and 20 MG), and so forth. The storage data are chosen from Table 2.3 starting from the lowest 8 MG in the 19th year.

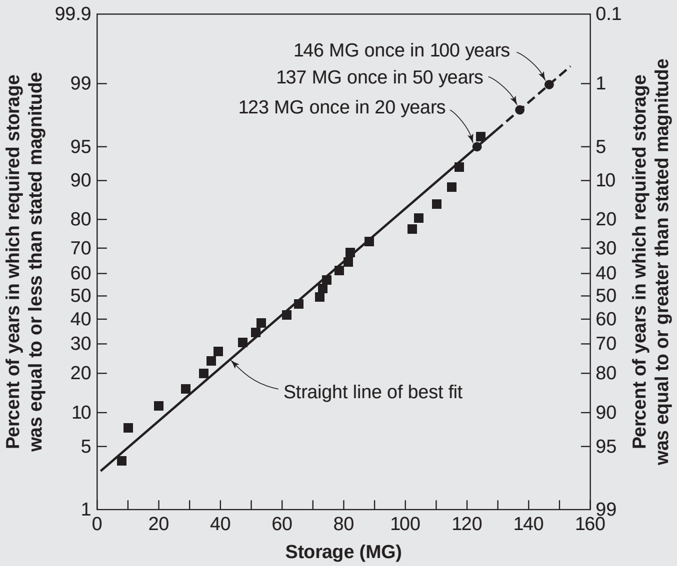

Figure 2.8 Frequency distribution of required storage plotted on arithmetic-probability paper. Conversion factors: 1 MG = 1,000,000 gal = 3.785 ML = 3,785,000 L.

2. The storage requirements reached or exceeded once in 20, 50, and 100 years, or 5%, 2%, and 1% of the time, are read as **123**, **137**, and **146 MG**, respectively (466, 519, and 553 ML, respectively). Probability paper is used because it offers a rational basis for projecting the information beyond the period of experience. The once-in-20-years requirement with 25% reserve suggests a design storage of **123/0.75 = 164 MG/mi^2 (230 ML/km^2) of drainage area**.

factor for reservoir area; D_e = equivalent draft, in./year or cm/year; (MGD/mi^2 or MLD/km^2); F = adjusted flow line at spillway level, in. or cm; A_e = effective catchment area, mi^2 or km^2; D_{ed} = effective draft, MGD or MLD; and D_{md} = mean draft, MGD or MLD.

The values of A (total catchment area) and a (reservoir surface area) obtained are used as follows in recalculating storage requirements: Q_r replaces Q; $D + D_e$ replaces D; and A_e replaces A. A fourth allowance calls for raising the flow line of the reservoir by $Q + E - R$, expressed in units of length yearly. In rough approximation, the spillway level is raised by a foot or two (0.3048 m or 0.6096 m) in the eastern United States.

2.5.2 Seepage

If the valley enclosing a reservoir is underlain by porous strata, water may be lost by seepage. Subsurface exploration alone can foretell how much. Seepage is not necessarily confined to the dam site. It may occur wherever the sides and bottom of the reservoir are sufficiently permeable to permit water to escape through the surrounding hills.

2.5.3 Silting

Soil erosion on the watershed causes reservoir silting. Both are undesirable. Erosion destroys arable lands. Silting destroys useful storage (see Fig. 2.9). How bad conditions are in a given catchment area depends principally on soil and rock types, ground-surface slopes, vegetal cover, methods of cultivation, and storm-rainfall intensities.

Silt accumulations cannot be removed economically from reservoirs by any means so far devised. Dredging is expensive, and attempts to flush out deposited silt by opening scour valves in dams are fruitless. Scour only produces gullies in the silt. In favorable circumstances, however, much of the heaviest load of suspended silt can be steered through the reservoir by opening large sluices installed for this purpose. Flood flows are thereby selected for storage in accordance with their quality as well as their volume.

Reduction of soil erosion is generally a long-range undertaking. Involved are proper farming methods, such as contour plowing; terracing of hillsides; reforestation; cultivation of permanent pastures; prevention of gully formation through construction of check dams or debris barriers; and revetment of stream banks.

Figure 2.9 A watershed dam in northwest Iowa is completely silted in. The dam no longer functions to store water, since its capacity has been lost (Courtesy of the National Resources Conservation Service and USDA).

In the design of impounding reservoirs for silt-bearing streams, suitable allowance must be made for loss of capacity by silting. Rates of deposition are especially high in impoundments on flashy streams draining easily eroded catchment areas. The proportion of sediment retained is called its *trap efficiency*. A simple calculation will show that 2000 mg/L of suspended solids equals 8.3 tons/MG (1.989 tonnes/ML) and that an acre-ft of silt weighs almost 1,500 tons (1360.8 tonnes) if its unit weight is 70 lb/ft^3 (1211.4 kg/m^3). In some parts of the United States the volume of silt V_s in acre-ft (1 acre-ft = 0.32585 MG = 43,560 ft^3 = 1,233.5 m^3) deposited annually can be approximated by the equation

$$V_s = cA^n \text{ (US customary units)} \qquad \textbf{(2.7)}$$

where A is the size of the drainage area in mi^2 (note: 1 mi^2 = 2.59 km^2), and c and n are coefficients with a value of $n = 0.77$ for the US southwestern streams and values of c varying from 0.43 through 1.7 to 4.8 for low, average, and high deposition, respectively, the corresponding values for southeastern streams being $c = 0.44$ only and $n = 1.0$. Understandably, the magnitudes of c and n, here reported, apply only to the regions for which they were developed.

The volume of silt deposited annually can also be approximated by the following equation using metric units:

$$V_s = 1{,}233.5c[A/(2.59 \times 10^6)]^n \text{ (SI units)} \qquad \textbf{(2.7a)}$$

where

V_s = volume of silt deposited annually, m^3

A = the size of the drainage area, m^2

c = a coefficient with a value varying from 0.43 to 1.7 to 4.8 for low, average, and high deposition, respectively

n = a coefficient to be determined specifically for a target drainage area 0.77 for southwestern steams in the United States

A plot of trap efficiency against the proportion of the mean annual flow stored in a reservoir traces curves quite similar to curves for the expected performance of settling basins of varying effectiveness. Close to 100% of the sediment transported by influent streams may be retained in reservoirs storing a full year's tributary flow. Trap efficiency drops to a point between 65% and 85% when the storage ratio is reduced to 0.5 (half a year's inflow) and to 30–60% when the storage ratio is lowered to 0.1 (5 weeks' inflow). Silting is often fast when reservoirs are first placed in service and may be expected to drop off and reach a steady state as delta building goes on and shores become stabilized. An annual silting rate of 1.0 acre-ft/mi^2 (note: 1 acre-ft/mi^2 = 476.25 m^3/km^2 = 0.47625 ML/km^2) of watershed corresponds roughly to a yearly reduction in storage of 0.32585 MG/mi^2 (0.476

EXAMPLE 2.3 CALCULATIONS OF MEAN ANNUAL RUNOFF AND DRAFT

A mean draft of 30.0 MGD (113.6 MLD) is to be developed from a catchment area of 40.0 mi^2 (103.6 km^2). First calculations ask for a reservoir area of 1,500 acres (6.07 km^2) at flow line. The mean annual rainfall is 47.0 in./year (119.4 cm/year), the mean annual runoff is 27.0 in./year (68.6 cm/year), and the mean annual evaporation is 40.0 in./year (101.6 cm/year). Find the following:

1. The revised mean annual runoff
2. The equivalent mean draft
3. The equivalent land area
4. The adjusted flow line

Solution 1 (US Customary System):

1. By Eq. (2.2), the revised annual runoff is

   $Q_r = Q - (Q + E - R)(fa/A)$

   $Q_r = 27.0 - (27.0 + 40.0 - 47.0)[(0.9 \times 1{,}500/640)/(40.0)] = 27.0 - 1.1 =$ **25.9 in./year**.

   Here, 1 mi^2 = 640 acres.

2. By Eq. (2.3), the equivalent mean draft is

   $D_e = (Q + E - R)(fa/A)$

   $D_e = (27.0 + 40.0 - 47.0)[(0.9 \times 1{,}500/640)/(40.0)] = 1.1$ in./year.

   $D_e = 1.1$ in./year = 52,360 gpd/mi^2 = 0.052 MGD/mi^2

   and the effective draft D_{ed} is

   $D_{ed} = D_{md} + D_e(A) = 30.0$ MGD + (0.052 MGD/mi^2)(40.0 mi^2) = **32.1 MGD**.

3. By Eq. (2.4), the equivalent land area is

   $A_e = A - fa[1 - (R - E)/Q]$

   $A_e = 40.0 - (0.9 \times 1{,}500/640)[1 - (47.0 - 40.0)/27.0] = 40.0 - 1.6 =$ **38.4 mi^2**.

4. By Eq. (2.5), the adjusted flow line is

   $F = Q + E - R$

   $F = 27.0 + 40.0 - 47.0 = 20$ in., equaling $20 \times 0.9 =$ **18 in. at spillway level**.

Solution 2 (SI System):

1. By Eq. (2.2), the revised annual runoff is

   $Q_r = Q - (Q + E - R)(fa/A)$

   $Q_r = 68.58 - (68.58 + 101.60 - 119.38)[(0.9 \times 6.07)/(103.6)] = 68.58 - 2.68 =$ **65.9 cm/year**.

2. By Eq. (2.3), the equivalent mean draft is

   $D_e = (Q + E - R)(fa/A)$

   $D_e = (68.58 + 101.60 - 119.38)[(0.9 \times 6.07)/(103.6)] = 2.68$ cm/year.

   $D_e = 2.68$ cm/year = 0.07337 MLD/km^2 and the effective draft D_{ed} is

   $D_{ed} = D_{md} + D_e(A) = 113.55$ MLD + (0.07337 MLD/km^2)(103.6 km^2) = 121.2 MLD.

   Here, 1 cm/year = 0.0273793 MLD/km^2.

3. By Eq. (2.4), the equivalent land area is

   $A_e = A - fa[1 - (R - E)/Q]$

   $A_e = 103.6 - (0.9 \times 6.07)[1 - (119.38 - 101.60)/68.58] = 103.6 - 4.47 =$ **99.13 km^2**.

4. By Eq. (2.5), the adjusted flow line is

   $F = Q + E - R$

   $F = 68.58 + 101.60 - 119.38 = 50.8$ cm, equaling (50.8 cm) $\times$ 0.9 = **45.7 cm at spillway level**.

EXAMPLE 2.4 VOLUME OF SILTING

Determine the volume of silt accumulations for a drainage area having the following characteristics:

Area = $100\,\text{mi}^2 = 259\,\text{km}^2 = 2.59 \times 10^8\,\text{m}^2 = 259{,}000{,}000\,\text{m}^2$
Average deposition of silt: $c = 1.7$
Area located in the southwestern United States: $n = 0.77$

Solution 1 (US Customary System):

$$V_s = cA^n \tag{2.7}$$

where

V_s = volume of silt deposited annually, acre-ft
A = the size of the drainage area, mi^2

$$\begin{aligned} V_s &= 1.7 \times (100)^{0.77} \\ &= 1.7 \times 34.7 \\ &= \mathbf{59\ acre\text{-}ft}. \end{aligned}$$

Solution 2 (SI System):

$$V_s = 1{,}233.5c[A/(2.59 \times 10^6)]^n \tag{2.7a}$$

where

V_s = volume of silt deposited annually, m^3
A = the size of the drainage area in m^2

$$\begin{aligned} V_s &= 1{,}233.5c[A/(2.59 \times 10^6)]^n \\ V_s &= 1{,}233.5(1.7)[(2.59 \times 10^8)/(2.59 \times 10^6)]^{0.77} \\ &= 1{,}233.5(1.7)(34.6737) \\ &= \mathbf{72{,}708.98\ m^3}. \end{aligned}$$

ML/km^2) because an acre 3 ft deep (4,047 m^2 area by 0.9144 m deep) is about 1 MG (3.785 ML).

2.6 AREA AND VOLUME OF RESERVOIRS

The surface areas and volumes of water at given horizons are found from a contour map of the reservoir site. Areas enclosed by each contour line are planimetered, and volumes between contour lines are calculated. The *average-end-area method* is generally good enough for the attainable precision of measurements.

For uniform contour intervals h (ft or m) and successive contour areas $a_0, a_1, \ldots, a_n$ (acre or m^2), the volume V of water (acre-ft or m^3) stored up to the nth contour is

$$V = 1/2\,\text{h}[(a_0 + a_1) + (a_1 + a_2) + \cdots + (a_{n-1} + a_n)]$$

$$V = 1/2\,h\left(a_0 + a_n + 2\sum_{1}^{n-1} a\right) \tag{2.8}$$

For general use, surface areas and volumes are commonly plotted against contour elevations as in Fig. 2.10. Note that volumes must be determined from the surface area

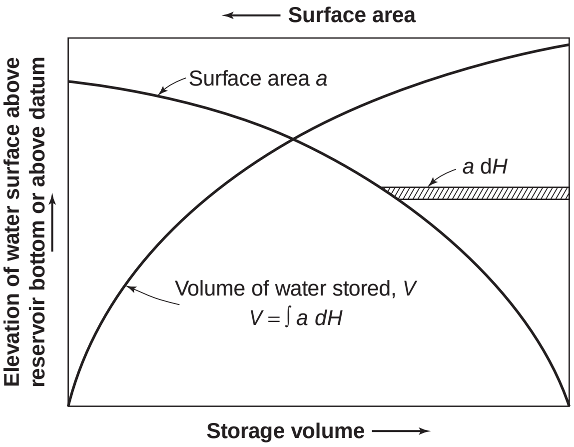

Figure 2.10 Surface area of a reservoir and volume of water stored.

curve by planimetering the area enclosed between the curve and its ordinate.

In reservoir operation, a small amount of water lies below the invert of the reservoir outlet. Constituting the dregs of the impoundage, this water is of poor quality. The associated reduction in *useful storage* is offset, in general, by bank storage released from the soil as the reservoir is drawn down. Moreover, the water below the outlet sill does form a conservation pool for fish and wildlife.

Surface areas and volumes enter not only into the solution of hydrologic problems but also into the management of water quality, such as the control of algae by copper sulfate and destratification by pumping or aeration.

2.7 MANAGEMENT OF CATCHMENT AREAS

The comparative advantage of developing surface rather than underground waters is offset, in large measure, by the unsteadiness of surface runoff, in both quantity and quality, and the recurrence of flow extremes. Those hydrologic factors that enter strongly into the development of surface-water supplies must, therefore, be kept clearly in mind in their design and operation, with special reference to

1. The principles of selecting, preparing, and controlling catchment areas.
2. The choice and treatment of reservoir areas and the management of natural ponds and lakes as well as impounding reservoirs.
3. The siting, dimensioning, construction, and maintenance of necessary engineering works, including dams and dikes, intake structures, spillways, and diversion works. Also keep in mind that river systems may have to be developed for multiple purposes, not just for municipal uses.

The gathering grounds for public water supplies vary *in size* from a few hundred acres to thousands of square miles, and in *character* from sparsely inhabited uplands to densely populated river valleys. The less developed they are, the better, relatively, they lend themselves to exploitation for steady yields and the production of water of high quality.

2.7.1 Upland Areas

Occasionally, a water utility can, with economic justification, acquire the entire watershed of its source and manage solely for water supply purposes, *excluding habitations and factories* to keep the water safe and attractive; *letting arable lands lie fallow* to prevent wasteful runoff and high turbidities; *draining swamps* to reduce evaporation and eliminate odors, tastes, and color; and *cultivating woodlots* to hold back winter snows and storm runoff and help preserve the even tenor of stream flow. As competition for water and land increases, land holdings of water utilities are understandably confined to the marginal lands of water courses, especially those closest to water intakes themselves. Yet water quality management need not be neglected. Scattered habitations can be equipped with acceptable sanitary facilities; wastewaters can be adequately treated or, possibly, diverted into neighboring drainage areas not used for water supply; swamps can be drained; and soil erosion can be controlled. *Intelligent land management* of this kind can normally be exercised most economically when water is drawn from *upland sources* where small streams traverse land of little value and small area. However, some upland watersheds are big enough to satisfy the demands of great cities. The water supplies of Boston, New York, and San Francisco are examples.

2.7.2 Lowland Areas

When water is drawn from large lakes and wide rivers that, without additional storage, yield an abundance of water, management of their catchments ordinarily becomes the concern of more than one community (examples are the Ohio and Mississippi Rivers) and sometimes of more than a single state (the Delaware River is a notable example) and even of a single country (e.g., the Great Lakes are shared with Canada and the Colorado River is shared with Mexico). Regional, interstate, and international authorities must be set up to manage and protect land and water resources of this kind.

2.7.3 Quality Control

To safeguard their sources, water utilities can fence and post their lands, patrol watersheds, and obtain legislative authority for enforcing reasonable rules and regulations for the environmental management of the catchment area. When the cost of policing the area outweighs the cost of purifying its waters in suitable treatment works, purification is often preferred. It is likewise preferred when lakes, reservoirs, and streams become important recreational assets and their enjoyment can be encouraged without endangering their quality. It goes without saying that recreation must be properly supervised and recreational areas suitably located and adequately equipped with sanitary facilities.

2.7.4 Swamp Drainage

Three types of swamps may occur on catchment areas:

1. Rainwater swamps where precipitation accumulates on flat lands or where rivers overflow their banks in times of flood
2. Backwater swamps or reaches of shallow flowage in sluggish, often meandering streams where bends or other obstructions can hamper flow

3. Seepage-outcrop swamps where hillside meets the plain or where sand and gravel overlie clay or other impervious formations

Rainwater swamps can be drained by ditches cut into the floodplain; backwater swamps by channel regulation; and seepage-outcrop swamps by marginal interception of seepage waters along hillsides sometimes supplemented by the construction of central surface and subsurface drains.

2.8 RESERVOIR SITING

In the absence of natural ponds and lakes, intensive development of upland waters requires the construction of impounding reservoirs. Suitable siting is governed by interrelated considerations of adequacy, economy, safety, and palatability of the supply. Desirable factors include

1. *Surface topography* that generates a low ratio of dam volume to volume of water stored; for example, a narrow gorge for the dam, opening into a broad and branching upstream valley for the reservoir. In addition, a favorable site for a stream diversion conduit and a spillway, and a suitable route for an aqueduct or pipeline to the city are desirable.
2. *Subsurface geology* that ensures (a) safe foundations for the dam and other structures; (b) tightness against seepage through abutments and beneath the dam; and (c) materials, such as sand, gravel, and clay, for construction of the dam and appurtenant structures.
3. A *reservoir valley* that is sparsely inhabited, neither marshy nor heavily wooded, and not traversed by important roads or railroads; the valley being *so shaped* that waters pouring into the reservoir are not short-circuited to the outlet, and *so sloped* that there is little shallow flowage around the margins. Natural purification by storage can be an important asset. Narrow reservoirs stretching in the direction of prevailing winds are easily short-circuited and may be plagued by high waves. Areas of shallow flowage often support heavy growths of water plants while they are submerged and of land plants while they are uncovered. Shoreline vegetation encourages mosquito breeding; decaying vegetation imparts odors, tastes, and color to the water.
4. *Reservoir flowage* that interferes as little as possible with established property rights, proximity to the intake to the community served, and location at such an elevation that supply can be by gravity.

Large reservoirs may inundate villages, including their dwellings, stores, and public buildings; mills and manufacturing establishments; farms and farmlands, stables, barns, and other outhouses; and gardens, playgrounds, and graveyards. Although such properties can be seized by the *right of eminent domain*, a wise water authority will proceed with patience and understanding. To be humane and foster goodwill, the authority will transport dwellings and other wanted and salvable buildings to favorable new sites, establish new cemeteries or remove remains and headstones to grounds chosen by surviving relatives, and assist in reconstituting civil administration and the regional economy.

When reservoir sites are flooded, land plants die and organic residues of all kinds begin to decompose below the rising waters; nutrients are released; algae and other microorganisms flourish in the eutrophying environment; and odors, tastes, and color are intensified. Ten to 15 years normally elapse before the biodegradable substances are minimized and the reservoir is more or less stabilized.

In modern practice, reservoir sites are cleared only in limited measure as follows:

1. *Within the entire reservoir area:* (a) dwellings and other structures are removed or razed; (b) barnyards, cesspools, and privies are cleaned, and manure piles are carted away; (c) trees and brush are cut close to the ground, usable timber is salvaged, and slash, weeds, and grass are burned; (d) swamp muck is dug out to reasonable depths, and residual muck is covered with clean gravel, the gravel, in turn, being covered with clean sand; and (e) channels are cut to pockets that would not drain when the water level of the reservoir is lowered.
2. *Within a marginal strip between the high-water mark reached by waves and a contour line about 20 ft (6.1 m) below reservoir level:* (a) stumps, roots, and topsoil are removed; (b) marginal swamps are drained or filled; and (c) banks are steepened to produce depths near the shore that are close to 8 ft (2.44 m) during much of the growing season of aquatic plants—to do this, upper reservoir reaches may have to be improved by excavation or fill or by building auxiliary dams across shallow arms of the impoundage.

Soil stripping, namely, the removal of all topsoil containing more than 1% or 2% organic matter from the entire reservoir area, is no longer economical.

In malarious regions, impounding reservoirs should be so constructed and managed that they will not breed dangerous numbers of anopheline mosquitoes. To this purpose, banks should be clean and reasonably steep. To keep them so, they may have to be protected by riprap.

2.9 RESERVOIR MANAGEMENT

The introduction of impounding reservoirs into a river system or the existence of natural lakes and ponds within it raises questions of quality control. Limnological factors are important not only in the management of ponds, lakes, and reservoirs but also in reservoir design.

2.9.1 Quality Control

Of concern in the quality management of reservoirs is the control of water weeds and algal blooms; the bleaching of color; the settling of turbidity; destratification by mixing or aeration; and, in the absence of destratification, the selection of water of optimal quality and temperature by shifting intake depths in order to suit withdrawals to water uses or to downstream quality requirements.

2.9.2 Evaporation Control

The thought that oil spread on water will suppress evaporation is not new. It is well known that

1. Certain chemicals spread spontaneously on water as layers no more than a molecule thick.
2. These substances include alcohol (hydroxyl) or fatty acid (carboxyl) groups attached to a saturated paraffin chain of carbon atoms.
3. The resulting *monolayers* consist of molecules oriented in the same direction and thereby offering more resistance to the passage of water molecules than do thick layers of oil composed of multilayers of haphazardly oriented molecules.
4. The hydrophilic radicals (OH or COOH) at one end of the paraffin chain move down into the water phase, while the hydrophobic paraffin chains themselves stretch up into the gaseous phase. Examples of suitable chemicals are alcohols and corresponding fatty acids.

The cost and difficulty of maintaining adequate coverage of the water surface have operated against the widespread use of such substances. Small and light plastic balls have also been used to retard evaporation from water surfaces of reservoirs.

2.10 DAMS AND DIKES

Generally speaking, the great dams and barrages of the world are the most massive structures built by man. To block river channels carved through mountains in geologic time periods, many of them are wedged between high valley walls and impound days and months of flow in deep reservoirs. Occasionally, water reservoirs reach such levels that their waters would spill over low saddles of the divide into neighboring watersheds if saddle dams or dikes were not built to complement the main structure. In other ways, too, surface topography and subsurface geology are of controlling influence. Hydraulically, they determine the siting of dams; volumes of storage, including subsurface storage in glacial and alluvial deposits; and spillway and diversion arrangements. Structurally, they identify the nature and usefulness of foundations and the location and economic availability of suitable construction materials. Soils and rock of many kinds can go into the building of dams and dikes. Timber and steel have found more limited application. Like most other civil engineering constructions, therefore, dams and their reservoirs are derived largely from their own environment.

Structurally, dams resist the pressure of waters against their upstream face by gravity, arch action, or both. Hydraulically, they stem the tides of water by their tightness as a whole and the relative imperviousness of their foundations and abutments. They combine these hydraulic and structural properties to keep seepage within tolerable limits and channeled such that the working structures remain safe. Various materials and methods of construction are used to create dams of many types. The following are the most common types: (a) embankment dams of earth, rock, or both and (b) masonry dams (today largely concrete dams) built as gravity, arched, or buttressed structures.

2.10.1 Embankment Dams

Rock, sand, clay, and silt are the principal materials of construction for rock and earth embankments. Permeables provide weight, impermeables watertightness. Optimal excavation, handling, placement, distribution, and compaction with special reference to selective placement of available materials challenge the ingenuity of the designer and constructor. Permeables form the shells or shoulders, impermeables the core or blanket of the finished embankment. Depending in some measure on the abundance or scarcity of clays, relatively thick cores are centered in a substantially vertical position, or relatively thin cores are displaced toward the upstream face in an inclined position. Common features of an earth dam with a central clay core wall are illustrated earlier in Fig. 2.4. Concrete walls can take the place of clay cores, but they do not adjust well to the movements of newly placed, consolidating embankments and foundations; by contrast, clay is plastic enough to do so. If materials are properly dispatched from borrow pits, earth shells can be ideally graded from fine at the watertight core to coarse and well draining at the upstream and downstream faces. In rock fills, too, there must be effective transition from core to shell, the required change in particle size ranging from a fraction of a millimeter for fine sand through coarse sand (about 1 mm) and gravel (about 10 mm) to rock of large dimensions.

Within the range of destructive wave action, stone placed either as paving or as *riprap* wards off erosion of the upstream face. Concrete aprons are not as satisfactory, sharing as they do most of the disadvantages of concrete core walls. A wide *berm* at the foot of the protected slope helps to keep riprap in place. To prevent the downstream face from washing away, it is commonly seeded with grass or covering vines and provided with a system of surface and subsurface drains. Berms break up the face into manageable drainage areas and give access to slopes for mowing and maintenance. Although they

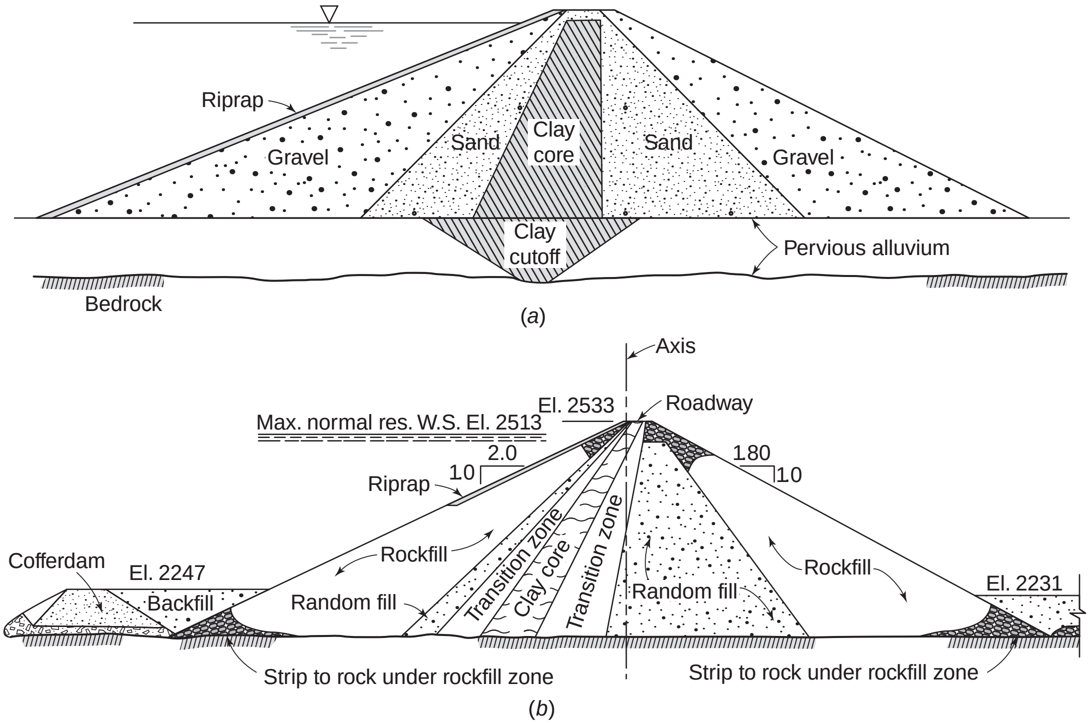

Figure 2.11 Zoned earth-fill and rock-fill dams: (a) earth-fill dam on pervious alluvium; (b) rock-fill dam on bedrock.

are more or less horizontal, berms do slope inward to gutters; moreover, they are pitched lengthwise for the gutters to conduct runoff to surface or subsurface main drains and through them safely down the face or abutment of the dam, eventually into the stream channel.

Earth embankments are constructed either as *rolled fills* or *hydraulic fills*; rock embankments are built as *uncompacted* (dumped) or *compacted* fills. In rolled earth fills, successive layers of earth 4–12 in. (100–300 mm) thick are spread, rolled, and consolidated. Sheep's foot rollers do the compacting, but they are helped in their work by heavy earthmoving vehicles bringing fill to the dam or bulldozing it into place. Portions of embankment that cannot be rolled in this way are compacted by hand or power tampers. Strips adjacent to concrete core walls, the walls of outlet structures, and the wingwalls of spillway sections are examples.

In *hydraulic fills* water-carried soil is deposited differentially to form an embankment graded from coarse at the two faces of the dam to fine in the central core.

Methods as well as materials of construction determine the strength, tightness, and stability of embankment dams. Whether their axis should be straight or curved depends largely on topographic conditions. Whether upstream curves are in fact useful is open to question. The intention is to provide axial compression in the core and prevent cracks as the dam settles. Spillways are incorporated into some embankment dams and divorced from others in separate constructions.

Where rock outcrops on canyon walls can be blasted into the streambed or where spillways or stream diversion tunnels are constructed in rock, rock embankment becomes particularly economical. In modern construction, rock fills are given internal clay cores or membranes in somewhat the same fashion as earth fills (Fig. 2.11). Concrete slabs or timber sheathing once much used on the upstream face can be dangerously stressed and fail as the fill itself, or its foundation, settles. They are no longer in favor.

2.10.2 Masonry Dams

In the construction of gravity dams, *cyclopean* masonry and mass concrete embedding great boulders have, in the course of time, given way to poured concrete; in the case of arched dams rubble has also ceded the field to concrete. Gravity dams are designed to be in compression under all conditions of loading. They will fit into almost any site with a suitable foundation. Some arched dams are designed to resist water pressures and other forces by acting as vertical cantilevers and horizontal arches simultaneously; for others, arch action alone is assumed, thrust being transmitted laterally to both sides of the valley, which must be strong enough to serve as abutments. In constant-radius dams, the upstream face is vertical or, at most, slanted steeply near the bottom; the downstream face is projected as a series of concentric, circular contours in plan. Dams of this kind fit well into U-shaped valleys, where cantilever action is expected to respond favorably to the high-intensity bottom loads. In constant-angle dams, the upstream face bulges upvalley; the downstream face curves inward like the small of a man's back. Dams of this kind fit well into V-shaped valleys, where arch action becomes their main source of strength at all horizons.

Concrete buttresses are designed to support flat slabs or multiple arches in buttress dams. Here and there, wood and steel structures have taken the place of reinforced concrete. Their upstream face is normally sloped one on one and may terminate in a vertical cutoff wall.

All masonry dams must rest on solid rock. Foundation pressures are high in gravity dams; abutment pressures are intense in arched dams. Buttress dams are light on their foundations. Making foundations tight by sealing contained pockets or cavities and seams or faults with cement or cement-and-sand grout under pressure is an important responsibility. Low-pressure grouting (up to 40 psig or 278 kPa) may be followed by high-pressure grouting (200 psig or 1390 kPa) from permanent galleries in the dam itself, and a curtain of grout may be forced into the foundation at the heel of gravity dams to obstruct seepage. Vertical drainage holes just downstream from the grout curtain help reduce uplift.

2.11 SPILLWAYS

Spillways have been built into the immediate structure of both embankment and masonry dams, in each instance as masonry sections (see Fig. 2.5). Masonry dams may indeed serve as spillways over their entire length. In general, however, spillways are placed at a distance from the dam itself to divert flow and direct possible destructive forces—generated, for example, by ice and debris, wave action, and the onward rush of waters—away from the structure rather than toward it. Saddle dams or dikes may be built to a lower elevation than the main impounding dam in order to serve as emergency floodways.

The head on the spillway crest at the time of maximum discharge is the principal component of the *freeboard*, namely, the vertical distance between maximum reservoir level and elevation of dam crest. Other factors are wave height (trough to crest), wave runup on sloping upstream faces, wind setup or tilting of the reservoir surface by the drag exerted in the direction of persistent winds in common with differences in barometric pressure, and (for earth embankments only) depth of frost.

Overflow sections of masonry and embankment dams are designed as masonry structures and separate spillways as *saddle*, *side channel*, and *drop inlet* or *shaft* structures. Spillways constructed through a *saddle* normally discharge into a natural floodway leading back to the stream below the dam. Usually they take the form of open channels and may include a relatively low overflow weir in the approach to the floodway proper. Overflow sections and overflow weirs must be calibrated if weir heads are to record flood discharges accurately, but their performance can be approximated from known calculations of similar structures. If their profile conforms to the ventilated lower nappe of a sharp-crested weir of the same relative height d/h (Fig. 2.12), under the design head, h, the rate of discharge, Q, becomes

$$Q = 2/3c\ \sqrt{2g}l\,h^{3/2} = \mathrm{C}l\,h^{3/2} \qquad \textbf{(2.9)}$$

where $C = 2/3c$ is the coefficient of discharge, g the gravity constant, and l the unobstructed crest length of the weir. For a crest height d above the channel bottom, the magnitude of C is approximately

$$C = 4.15 + 0.65\,h/d \text{ for } h/d < 4 \text{ or } C = 4.15 - 6.75 \qquad \textbf{(2.10)}$$

Under heads h' other than the design head h, C approximates to

$$C = 4.15(h'/h)^{7/5} \text{ up to a ratio of } h'/h = 3.0 \qquad \textbf{(2.11)}$$

If the entrance to the floodway is streamlined, little if any energy is lost—certainly no more than $0.05v^2/2g$. As suggested in Fig. 2.12, substantial quiescence within the reservoir must be translated into full channel velocity. Discharge is greatest when flow becomes critical. The velocity head h

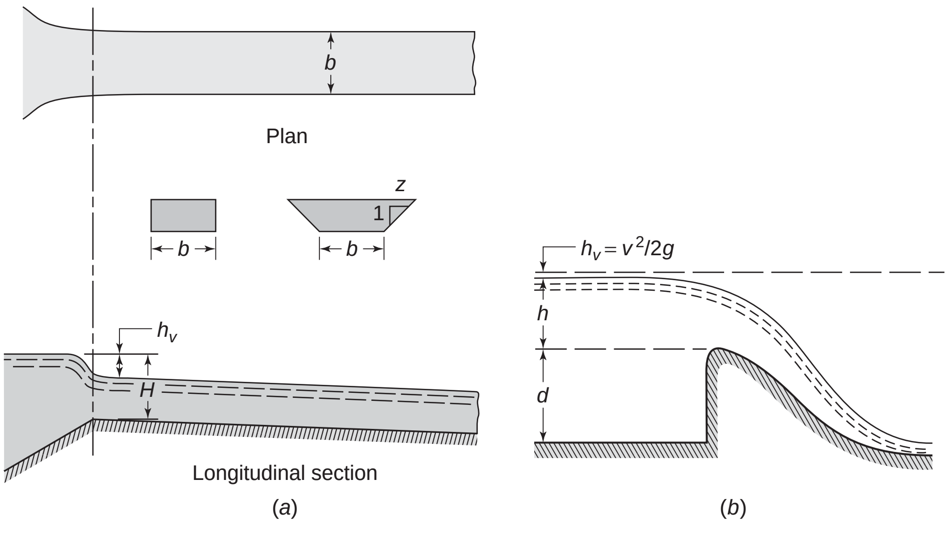

Figure 2.12 Spillways: (a) channel spillway and (b) ogee spillway (Kindness of Arthur Casagrande).

then equals one-third the height H of the reservoir surface above the entrance sill to a rectangular channel, and the rate of discharge Q becomes

$$Q = 2/3CbH\sqrt{2gH/3} = 3.087CbH^{3/2} \qquad \textbf{(2.12)}$$

where b is the width of the channel and C is an entrance coefficient varying from 1.0 for a smooth entrance to 0.8 for an abrupt one. A trapezoidal channel with side slopes of 1:2 discharges

$$Q = 8.03Ch_v^{1/2}(H - h_v)[b + z(H - h_v)] \qquad \textbf{(2.13)}$$

where

$$h_v = \frac{3(2zH + b) - (16z^2H^2 + 16zbH + 9b^2)^{1/2}}{10z} \qquad \textbf{(2.14)}$$

Best hydraulic but not necessarily best economic efficiency is obtained when a semicircle can be inscribed in the cross-section.

Flow is uniform below the entrance when friction and channel slope are in balance. Otherwise, flow becomes nonuniform and channel cross-section must be adjusted accordingly. A weir within the channel produces a backwater curve.

Side-channel spillways occupy relatively little space in the cross-section of a valley. The crest more or less parallels one abutting hillside and can be made as long as wanted. The channel, into which the spillway pours its waters, skirts the end of the dam and delivers its flows safely past the toe. If it is blasted out of tight rock, the channel can be left unlined. Crest length and channel size are determined in much the same way as for washwater gutters in rapid sand filters.

As shown in Fig. 2.13, *shaft* or *drop-inlet spillways* consist of an overflow lip supported on a shaft rising from an outlet conduit, often the original stream-diversion tunnel. The lip can be of any desired configuration. A circular lip and trumpet-like transition to the shaft form a *morning-glory spillway* that must lie far enough from the shore to be fully effective. By contrast, a three-sided semicircular lip can be placed in direct contact with the shore; accessibility is its advantage. The capacity of shaft spillways is governed by their constituent parts and by flow conditions including air entrainment and hydraulic submersion. Hydraulic efficiency and capacity are greatest when the conduit flows full. Model studies are useful in arriving at suitable dimensions.

Flashboards or stop logs and gates of many kinds are added to spillways to take advantage of storage above crest level. They must be so designed and operated that the dam itself is not endangered in times of flood.

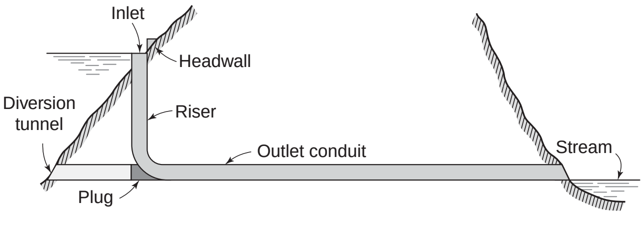

Figure 2.13 Shaft spillway and diversion tunnel.

2.12 INTAKES

Depending on the size and nature of the installation, water is drawn from rivers, lakes, and reservoirs through relatively simple submerged intake pipes, or through fairly elaborate tower-like structures that rise above the water surface and may house intake gates; openings controlled by stop logs; racks and screens, including mechanical screens, pumps, and compressors; chlorinators and other chemical feeders; venturi meters and other measuring devices; even living quarters and shops for operating personnel (Fig. 2.3). Important in the design and operation of intakes is that the water they draw be as clean, palatable, and safe as the source of supply can provide.

2.12.1 River Intakes

Understandably, river intakes are constructed well upstream from points of discharge of wastewater and industrial wastes. An optimal location will take advantage of deep water, a stable bottom, and favorable water quality (e.g., if pollution hugs one shore of the stream), all with proper reference to protection against floods, debris, ice, and river traffic (Fig. 2.14). Small streams may have to be dammed up by *diversion* or *intake dams* to keep intake pipes submerged and preclude hydraulically wasteful air entrainment. The resulting intake pool will also work well as a settling basin for coarse silt and allow a protective sheet of ice to form in winter.

2.12.2 Lake and Reservoir Intakes

Lake intakes are sited with due reference to sources of pollution, prevailing winds, surface and subsurface currents, and

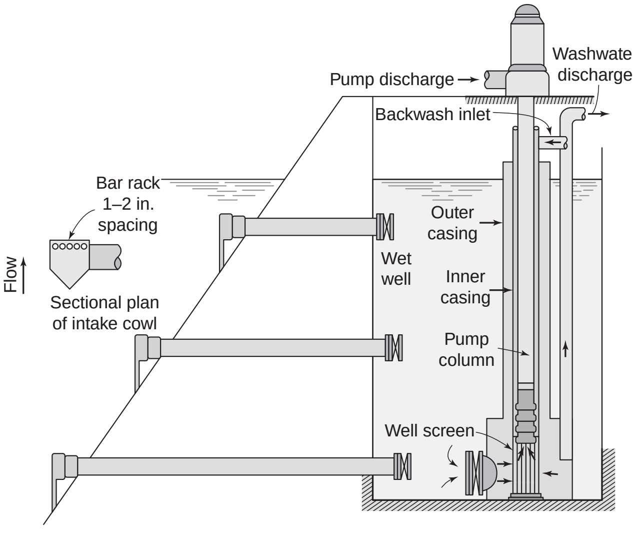

Figure 2.14 River or lake intake with vertical pump and backwashed well-type screen.

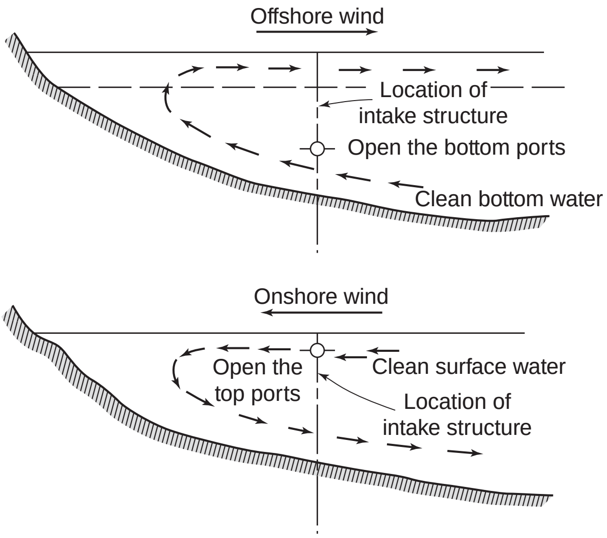

Figure 2.15 Effect of onshore and offshore winds on water quality at water intake.

shipping lanes. As shown in Fig. 2.15, shifting the depth of draft makes it possible to collect clean bottom water when the wind is offshore and, conversely, clean surface water when the wind is onshore. If the surrounding water is deep enough, bottom sediments will not be stirred up by wave action, and ice troubles will be few.

Reservoir intakes resemble lake intakes but generally lie closer to shore in the deepest part of the reservoir (see Fig. 2.6). They are often incorporated into the impounding structure itself (see Fig. 2.4). Where a reservoir serves many purposes, the intake structure is equipped with gates, conduits, and machinery not only for water supply but also for regulation of low-water flows (including compensating water); generation of hydroelectric power; release of irrigation waters; and control of floods. Navigation locks and fish ladders or elevators complete the list of possible control works.

2.12.3 Submerged and Exposed Intakes

Submerged intakes are constructed as *cribs* or *screened bellmouths*. Cribs are built of heavy timber weighted down with rocks to protect the intake conduit against damage by waves and ice and to support a grating that will keep large objects out of the central intake pipe.

Exposed intake gatehouses, often still misnamed cribs, are tower-like structures built (a) into dams, (b) on banks of streams and lakes, (c) sufficiently near the shore to be connected to it by a bridge or causeway, and (d) at such distance from shore that they can be reached only by boat (see Figs. 2.3 and 2.4). In *dry intakes*, ports in the outer wall admit water to gated pipes that bridge a circumferential dry well and open into a central wet well comprising the entrance to the intake conduit. In *wet intakes*, water fills both wells. Open ports lead to the outer well, whence needed flows are drawn through gated openings into the inner well.

2.12.4 Intake Velocities and Depths

In cold climates, ice troubles are reduced in frequency and intensity if intake ports lie as much as 25 ft (7.5 m) below the water surface and entrance velocities are kept down to 0.30 ft/s (0.9 m/s). At such low velocity, ice spicules, leaves, and debris are not entrained in the flowing water and fish are well able to escape from the intake current.

Bottom sediments are kept out of intakes by raising entrance ports 4–6 ft (1.2–1.8 m) above the lake or reservoir floor. Ports controlled at numerous horizons permit water quality selection and optimization. A vertical interval of 15 ft (4.5 m) is common. Submerged gratings are given openings of 2–3 in. (5–7.5 cm). Specifications for screens commonly call for two to eight meshes to the inch and face (approach) velocity of 0.30 ft/s (0.9 m/s). Wet wells should contain blow-off gates for cleaning and repairs.

2.12.5 Intake Conduits and Pumping Stations

Intakes are connected to the shores of lakes and reservoirs (a) by pipelines (often laid with flexible joints) or (b) by tunnels blasted through rock beneath the lake or reservoir floor. Pipelines are generally laid in a trench on the floor and covered after completion. This protects them against disturbance by waves and ice. Except in rock, conduits passing through the foundations of dams are subjected to heavy loads and to stresses caused by consolidation of the foundation.

Intake conduits are designed to operate at self-cleansing velocities of 3–4 ft/s (0.9–1.2 m/s). Flow may be by gravity or suction. Pump wells are generally located on shore. Suction lift, including friction, should not exceed 15–20 ft (4.5–6 m). Accordingly, pump wells or rooms are often quite deep. The determining factor is the elevation of the river, lake, or reservoir in times of drought. Placing pumping units in dry wells introduces problems of hydrostatic uplift and seepage in times of flood. Wet wells and deep-well pumps may be used instead.

2.13 DIVERSION WORKS

Depending on the geology and topography of the dam site and its immediate surroundings, streams are diverted from the construction area in two principal ways:

1. The entire flow is carried around the site in a diversion conduit or tunnel. An upstream cofferdam and, if necessary, a downstream cofferdam lay the site dry. After fulfilling its duty of bypassing the stream and protecting the valley during construction, the diversion conduit is usually incorporated in the intake or regulatory system of the reservoir (see Figs. 2.4 and 2.13).

2. The stream is diverted to one side of its valley, the other side being laid dry by a more or less semicircular cofferdam. After construction has progressed far enough in the protected zone, stream flow is rediverted through a sluiceway in the completed section of the dam, and a new cofferdam is built to pump out the remaining portion of the construction site.

Diversion conduits are built as grade aqueducts and tunnels, or as pressure conduits and tunnels. As a matter of safety, however, it should be impossible for any conduit passing through an earth embankment dam to be put under pressure; a leak might bring disaster. Accordingly, gates should be installed only at the inlet portal, never at the outlet portal. If a pipe must work under pressure, it should be laid within a larger access conduit. To discourage seepage along their outer walls, conduits passing through earth dams or earth foundations are often given projecting fins or collars that increase the length of path of seepage (by, say, 20% or more) and force flow in the direction of minimum as well as maximum permeability. At their terminus near the toe of the dam, moreover, emerging conduits should be surrounded by rock, through which residual seepage waters can escape safely.

The capacity of diversion conduits is determined by flood-flow requirements. Variations in the head and volume of floodwater impounded behind the rising dam are important factors in this connection. Rising heads normally increase the capacity of diversion conduits, and increasing storage reduces the intensity of floods. At the same time, however, dangers to the construction site and the valley below mount higher.

2.14 COLLECTION OF RAINWATER

Rain is rarely the immediate source of municipal water supplies—a notable example is the water supply of the communities in the islands of Bermuda, on which streams are lacking and groundwater is brackish. The use of rainwater is generally confined (a) to farms and towns in semiarid regions devoid of satisfactory groundwater or surface-water supplies, and (b) to some hard-water communities in which, because of its softness, roof drainage is employed principally for household laundry work and general washing purposes, while the public supply satisfies all other requirements. In most hard-water communities, the installation and operation of municipal water-softening plants can ordinarily be justified economically. Their introduction is desirable and does away with the need for supplementary rainwater supplies and the associated objection of their possible cross-connection with the public supply.

For individual homesteads, rainwater running off the roof is led through gutters and downspouts to a rain barrel or cistern situated on the ground or below it (Fig. 2.16). *Barrel* or *cistern* storage converts the intermittent rainfall into a continuous supply. For municipal service, roof water may be combined with water collected from sheds or catches on the surface of ground that is naturally impervious or rendered so by grouting, cementing, paving, or similar means.

The *gross yield* of rainwater supplies is proportional to the receiving area and the amount of precipitation. Some rain, however, is blown off the roof by wind, evaporated, or lost in wetting the collecting area and conduits and in filling depressions or improperly pitched gutters. Also, the first flush of

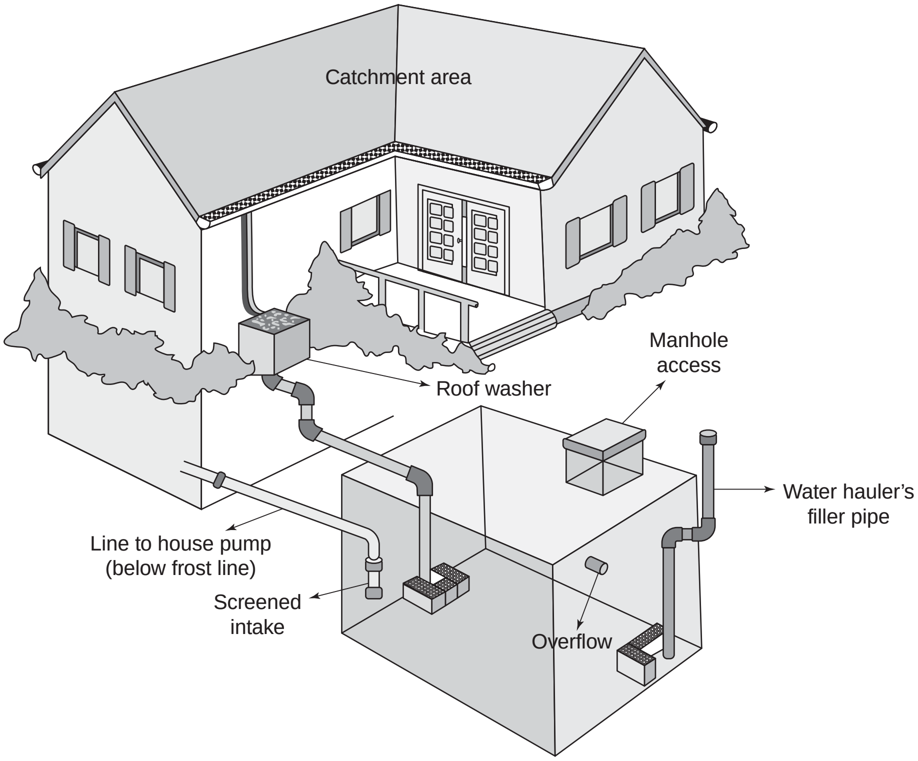

Figure 2.16 Rainwater collection.

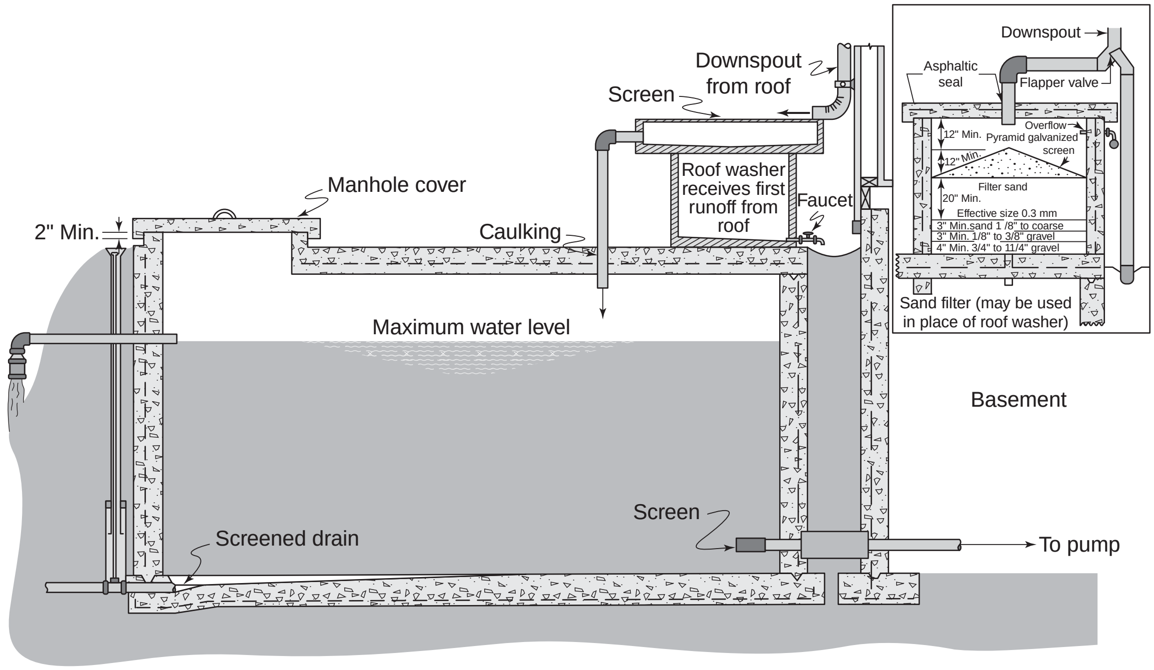

Figure 2.17 Cistern equipped with sand filtration for collection of surface water in rural area for drinking. Conversion factor: 1″ = 1 in. = 2.54 cm.

water contains most of the dust and other undesirable washings from the catchment surfaces and may have to be wasted. The combined loss is particularly great during the dry season of the year. A cutoff, switch, or deflector in the downspout permits selection of the quality of water to be stored. *Sand filters* (Fig. 2.17) are successfully employed to cleanse the water and prevent its deterioration (a) by growth of undesirable organisms and (b) by the bacterial decomposition of organic materials, both of which may give rise to tastes, odors, and other changes in the attractiveness and palatability of the water.

Storage to be provided in cisterns depends on seasonal rainfall characteristics and commonly approximates one-third to one-half of the annual needs in accordance with the length of dry spells. If the water is to be filtered before storage, standby capacity in advance of filtration must be provided if rainfalls of high intensity are not to escape. Because of the relatively small catchment area available, roof drainage cannot be expected to yield an abundant supply of water, and a close analysis of storm rainfalls and seasonal variations in precipitation must be made if catchment areas, standby tanks, filters, and cisterns are to be proportioned and developed properly.

A properly located and constructed controlled catchment and cistern, augmented by satisfactory treatment facilities, will provide safe water. A *controlled catchment* is a defined surface area from which rainfall runoff is collected. For these controlled catchments, simple guidelines to determine water yield from rainfall totals can be established. When the controlled catchment area has a smooth surface or is paved and the runoff is collected in a cistern, water loss due to evaporation, replacement of soil moisture deficit, and infiltration is small. As a general rule, losses from smooth concrete or asphalt-covered ground catchments average less than 10%. For shingled roofs or tar and gravel surfaces, losses should not exceed 15%; and for sheet metal roofs the loss is negligible. A conservative design can be based on the assumption that the amount of water that can be recovered for use is three-fourths of the total annual rainfall. The location of the cistern should be governed by both convenience and quality protection. A cistern should be as close to the point of ultimate use as practical. A cistern should not be placed closer than 50 ft (15 m) to any part of a sewage-disposal installation and should be on higher ground.

Cisterns collecting water from roof surfaces should be located adjacent to the building, but not in basements subject to flooding. They may be placed below the surface of the ground for protection against freezing in cold climates and to keep water temperatures low in warm climates, but should be situated on the highest ground practicable, with the surrounding area graded to provide good drainage.

The size of cistern needed will depend on the size of the family and the length of time between periods of heavy rainfall. The size of the catchment area will depend on the amount of rainfall and the character of the surface. The safety factor allowed should be for lower than normal rainfall levels. Designing for two-thirds of the mean annual rainfall will result usually in a catchment area of adequate capacity (Fig. 2.18).

EXAMPLE 2.5 SIZING OF CISTERN AND REQUIRED CATCHMENT AREA

A farmhouse for a family of four people has minimum drinking and culinary requirements of 120 gpd (454 L/d). The mean annual rainfall is 45 in. (114 cm), and the effective period between rainy periods is 100 days. Determine

1. The size of the required rainfall collection cistern
2. The size of the required catchment area

Solution 1 (US Customary System):

1. *The size of the required rainfall collection cistern:*

   The minimum volume of the cistern required will be

   120 gpd × 100 days = **12,000 gal** = 12,000/7.48 = 1,604 ft^{3}.

   Say, a **10-ft-deep, 13-ft-square cistern**.

2. *The size of the required catchment area:*

   Because the mean annual rainfall is 45 in., the total design rainfall is 45 × 2/3 = 30 in.

   The total year's requirement = 365 days × 120 gpd = 43,800 gal.

   Referring to Fig. 2.18, the catchment area required to produce 43,800 gal from a design storm of 30 in. is **3,300 ft^2**.

   Say, a **100-ft × 33-ft catchment area**.

Solution 2 (SI System):

1. *The size of the required rainfall collection cistern:*

   The minimum volume of the cistern required will be

   (454.2 L/d) × 100 days = **45,420 L** = 45.42 m^3.

   Say, a **3-m-deep, 4-m-square cistern** (**3 m × 4 m × 4 m**).

2. *The size of the required catchment area:*

   Because the mean annual rainfall is 114 cm, then the total design rainfall is (114 cm) × 2/3 = 76.2 cm.

   The total year's requirement = (365 days)(454 L/d) = 165,800 L.

   Referring to Fig. 2.18, the catchment area required to produce 165,800 L (43,800 gal) from a design storm of 76 cm (30 in.) is 307 m^2 (**3,300 ft^2**).

   Say, a **31-m × 10-m catchment area**.

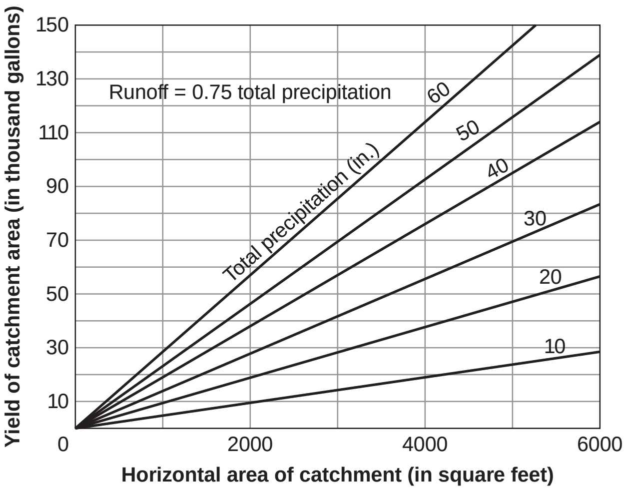

Figure 2.18 Yield of impervious catchment area. Conversion factors: 1 in. = 25.4 mm; 1 gal = 3.785 L; 1 ft^2 = 0.0929 m^2.

PROBLEMS/QUESTIONS

2.1 The response of a new water surface will establish new hydrologic equilibria. Determine the equivalent mean draft under the following conditions: (a) areal rate of runoff of the original watershed Q = 27.5 in./year (69.85 cm/year); (b) areal rate of evaporation E = 40.5 in./year (102.87 cm/year); (c) areal rate of rainfall R = 47.5 in./year (120.65 cm/year); (d) reservoir area a = 1,500 acres (6.07 km^2); (e) catchment area A = 25,600 acres (103.60 km^2); and (f) mean annual water surface = 90% of reservoir area at spillway level.

2.2 Determine the revised mean annual runoff, Q_r, assuming (a) the original mean annual runoff Q = 27.5 in./year (69.85 cm/year); and (b) the equivalent mean draft D_e = 1.08 in./year (2.74 cm/year).

2.3 Determine the effective catchment area assuming (a) original catchment area A = 25,600 acres (103.60 km^2); (b) reservoir area a = 1,500 acres (6.07 km^2); (c) mean annual water surface = 90% of reservoir area at spillway level; (d) original watershed's runoff rate Q = 27.5 in./year (69.85 cm/year); (e) evaporation rate E = 40.5 in./year (102.87 cm/year); and (f) rainfall rate R = 47.5 in./year (120.65 cm/year).

2.4 In some parts of the United States, the volume of silt V_s in acre-ft deposited annually can be approximated by the following equation:

$$V_s = c(A)^n$$

where

A = drainage area, mi^2

c = 0.43 for low deposition; 1.7 for average deposition; 4.8 for high deposition

n = 0.77 for southwestern streams

Determine the volume of silt accumulations for drainage area, where A, c, and n are 40 mi^2 (103.60 km^2), 1.7, and 0.77, respectively.

2.5 The volume of silt deposited annually can be approximated by the following equation:

$$V_s = 1{,}233.5c[A/(2.59 \times 10^6)]^n$$

where

V_s = volume of silt deposited annually, m^3

A = the size of the drainage area, m^2

c = a coefficient with a value varying from 0.43 through 1.7 to 4.8 for low, average, and high deposition, respectively

n = a coefficient to be determined specifically for a target drainage area

Determine the volume of silts deposited annually V_s in m^3, assuming it is known that $c = 1.7$, $n = 0.77$, and $A = 648 \times 10^6$ m^2 (160,119 acres).

2.6 Using the same metric equation from Problem 2.5 of $V_s = 1{,}233.5c[A/(2.59 \times 10^6)]^n$ and assuming $c = 1.7$, $n = 0.77$, and drainage area = 984×10^6 m^2 (243,143 acres), determine the volume of silt deposited annually.

2.7 A research team has investigated two drainage areas in the southwestern region of the United States, and has measured the volumes of their annually deposited silts. The following are the field data collected by the researchers using the metric units:

For drainage area 1:

$$A_1 = 984 \times 10^6 \text{m}^2 = 243{,}143 \text{ acres} = 379.8 \text{ mi}^2$$

$$V_{s_1} = 203{,}000 \text{ m}^3 = 164.6 \text{ acre-ft}$$

For drainage area 2:

$$A_2 = 648 \times 10^6 \text{ m}^2 = 160{,}119 \text{ acres} = 250 \text{ mi}^2$$

$$V_{s2} = 147{,}000 \text{ m}^3 = 119.17 \text{ acre-ft}$$

Determine the coefficients of c and n for the southwestern region of the United States.

2.8 The volume of silt deposited annually can be approximated by the following equation using English units:

$$V_s = c(A)^n$$

where

V_s = volume of silt deposited annually, acre-ft

A = the size of the drainage area, mi^2

c = a coefficient with a value varying from 0.43 through 1.7 to 4.8 for low, average, and high deposition, respectively

n = a coefficient to be determined specifically for a target drainage area

A research team has investigated two drainage areas in the southwestern region of the United States, and has measured the volumes of their annually deposited silts. The following are the field data collected by the researchers using English units:

For drainage area 1:

$$A_1 = 380 \text{ mi}^2 = 984.2 \times 10^6 \text{ m}^2$$

$$V_{s1} = 165 \text{ acre-ft} = 203{,}528 \text{ m}^3$$

For drainage area 2:

$$A_2 = 250 \text{ mi}^2 = 647.5 \times 10^6 \text{ m}^2$$

$$V_{s1} = 120 \text{ acre-ft} = 148{,}020 \text{ m}^3$$

Determine the coefficients of c and n for the southwestern region of the United States.

2.9 A spillway weir has an unobstructed crest length of 20 ft (6.1 m) and a coefficient of discharge equal to 5.50. If the design water head is 0.75 ft (0.23 m), what is the rate of discharge?

2.10 A house for a family of four people has minimum drinking and culinary requirements of 25 gpcd (95 Lpcd). The mean annual rainfall is 50 in./year (127 cm/year) and the effective period between rainy periods is 150 days. Determine (a) the size of the required rainfall collection cistern and (b) the size of the required catchment area.

2.11 Define the following technical terms: (a) draft; (b) eutrophication; (c) sublimation; (d) condensation; and (e) infiltration.

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Chapter 3

Water Sources: Groundwater

Groundwater from wells and springs has served as a source of domestic water supply since antiquity. Table 3.1 shows that in the United States more water systems have groundwater than surface water as a source—but more people drink from a surface-water system. Thirty-five percent (107 million) of the total population of 306 million served by public water systems depend on groundwater. Groundwater works are 10 times more numerous than surface-water installations (about 141,000 to 14,000); the average capacity of groundwater facilities is, however, much smaller. Contributions from groundwater also play a major role in the supplies depending on surface sources. It is the discharge of groundwater that sustains the dry-season flow of most streams.

Groundwater is more widely distributed than surface water. Its nearly universal, albeit uneven, occurrence and other desirable characteristics make it an attractive source of water supply. Groundwater offers a naturally purer, cheaper, and more satisfactory supply than does surface water. It is generally available at the point of use and obviates the need to incur substantial transmission costs. It occurs as an underground reservoir, thus eliminating the necessity of impoundment works. It is economical even when produced in small quantities.

To an increasing degree, engineers are being called on to investigate the possibility of developing groundwater as a usable resource. The following factors need to be considered:

1. The effective water content, that is, the maximum volume of water that can be withdrawn from a body of groundwater through engineering works. The effective *porosity* and *storage coefficient* of the water-bearing material control the useful storage.
2. The ability of the aquifer to transmit water in requisite quantities to wells or other engineering installations. *Permeability* and *transmissivity* are the indicators of this capability.
3. The suitability of the quality of water for the intended use, after treatment if necessary.
4. The reliability and permanence of the available supply with respect to both the quantity and the quality of water.

As a source of a permanent and reliable water supply, only that portion of the subsurface water that is in the zone of saturation need be considered. In this zone almost all the interstices are completely filled with water under hydrostatic pressure (atmospheric pressure or greater). That water is free to move in accordance with the laws of saturated flow from places where it enters the zone of saturation (recharge areas) to places where it is discharged. The main features of the groundwater phase of the hydrologic cycle are depicted in Fig. 3.1.

3.1 POROSITY AND EFFECTIVE POROSITY

The amount of groundwater stored in saturated materials depends on the material's *porosity*, the ratio of the aggregate volume of interstices in a rock or soil to its total volume. It is usually expressed as a percentage. The concept of porosity involves all types of interstices, both primary (original) and secondary. Primary interstices were created at the time of the rock's origin. In granular unconsolidated sediments, they coincide with intergranular spaces. In volcanic rocks, they include tubular and vesicular openings. Secondary interstices result from the action of geologic, mechanical, and chemical forces on the original rock. They include joints, faults, fissures, solution channels, and bedding planes in hard rocks. The extent of fracturing and intensity of weathering exert a profound influence on the distribution of larger interstices. The importance of secondary porosity in determining the amount of water that can be obtained from a formation is often great in those hard rocks that lack intergrain porosity. This type of porosity is dependent on local conditions and gives water-bearing formations a heterogeneous character. The distribution of secondary porosity varies markedly with depth.

Porosity is a static quality of rocks and soils. It is not itself a measure of *perviousness* or *permeability*, which are dynamic quantities controlling the flow. Not all the water stored in a saturated material is available for movement; only the interconnected interstices can participate in flow. Water in isolated openings is held immobile. Furthermore, water in a part of the interconnected pore space is held in

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Water Engineering: Hydraulics, Distribution and Treatment, First Edition. Nazih K. Shammas and Lawrence K. Wang.

Table 3.1 US water sources in 2007

Water system		Groundwater	Surface water	Totals
Community water system[a]	No. of systems	40,646	11,449	52,095
	Population served	90,549,995	195,887,109	286,437,104
	Percentage of systems	78	22	100
	Percentage of population	32	68	100
Nontransient noncommunity water system[b]	No. of systems	18,151	679	18,830
	Population served	5,503,282	787,555	6,290,837
	Percentage of systems	96	4	100
	Percentage of population	87	13	100
Transient noncommunity water system[c]	No. of systems	82,851	1,878	84,729
	Population served	11,077,369	2,668,985	13,746,354
	Percentage of systems	98	2	100
	Percentage of population	81	19	100
Total no. of systems		141,648	14,006	155,654

Source: Courtesy US Environmental Protection Agency.

[a] Community water system: a public water system that supplies water to the same population year-round.

[b] Nontransient noncommunity water system: a public water system that regularly supplies water to at least 25 of the same people at least 6 months per year, but not year-round. Some examples are schools, factories, office buildings, and hospitals that have their own water systems.

[c] Transient noncommunity water system: a public water system that provides water in a place such as a gas station or campground where people do not remain for long periods of time.

place by molecular and surface tension forces. This is the dead storage and is called *specific retention*. Thus not all the water stored in a geologic formation can be withdrawn by normal engineering operations. Accordingly, there is a difference between total storage and useful storage. That portion of the pore space in which flow takes place is called *effective porosity*, or *specific yield* of the material, defined as the proportion of water in the pores that is free to drain away or be withdrawn under the influence of gravity. Specific yields vary from 0 for plastic clays to 30% or more for uniform sands and gravels. Most aquifers have yields of 10–20%.

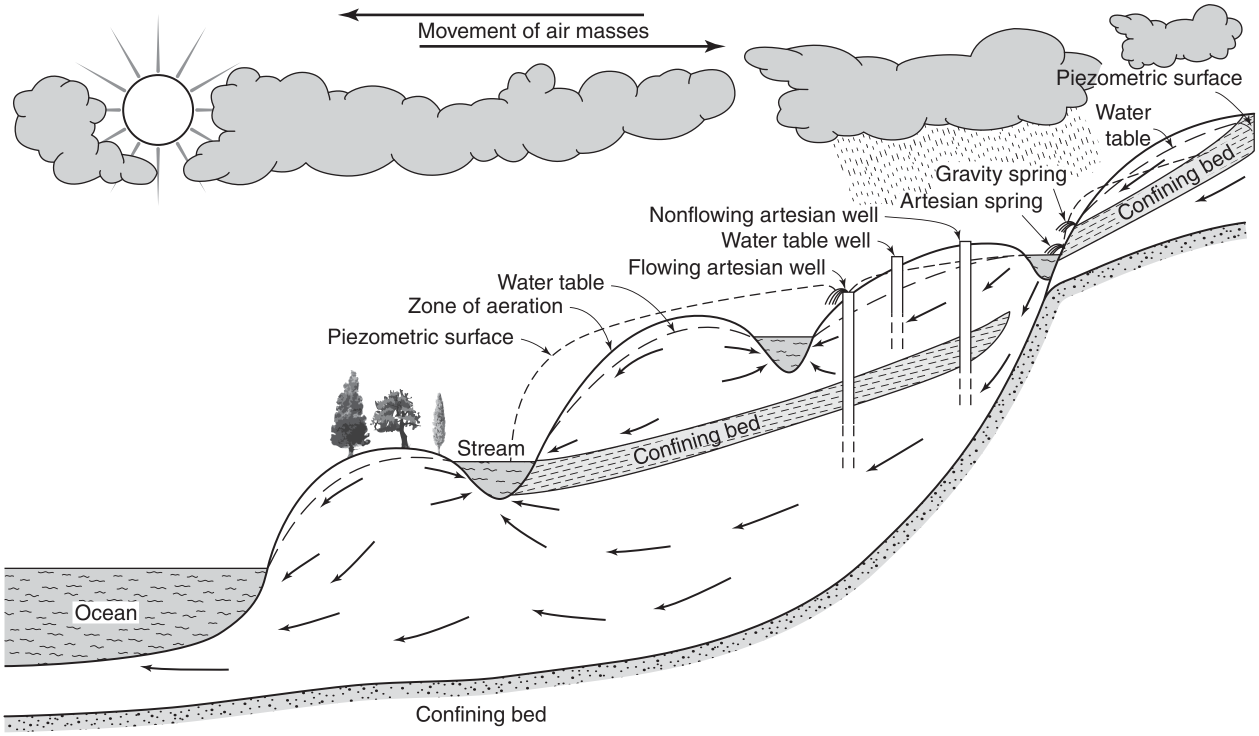

Figure 3.1 Groundwater features of the hydrologic cycle (After McGuinness of USGS).

3.2 PERMEABILITY

The *permeability* or *perviousness* of a rock is its capacity for transmitting a fluid under the influence of a hydraulic gradient. An important factor affecting the permeability is the geometry of the pore spaces and of the rock particles. The nature of the system of pores, rather than their relative volume, determines the resistance to flow at given velocities. There is no simple and direct relationship between permeability and porosity. Clays with porosities of 50% or more have extremely low permeability; sandstones with porosities of 15% or less may be quite pervious.

A standard unit of intrinsic permeability, dependent only on the properties of the medium, is *darcy*, D. It is expressed as flow, in cubic centimeters per second, of a fluid of one centipoise viscosity, through a cross-sectional area of 1 cm^2 of the porous medium under a pressure gradient of 1 atm/cm. It is equivalent to a water flow of 18.2 gpd/ft^2 (0.743 $\text{m}^3/\text{d/m}^2$) under a hydraulic gradient of 1 ft/ft (1 m/m) at a temperature of 60.0°F (15.53°C).

The *homogeneity* and *isotropy* of a medium refer to the spatial distribution of permeability. A porous medium is isotropic if its permeability is the same in all directions. It is called *anisotropic* if the permeability varies with the direction. Anisotropy is common in sedimentary deposits where the permeability across the bedding plane may be only a fraction of that parallel to the bedding plane. The medium is *homogeneous* if the permeability is constant from point to point over the medium. It is *nonhomogeneous* if the permeability varies from point to point in the medium. Aquifers with secondary porosity are nonhomogeneous. Isotropy and homogeneity are often assumed in the analysis of groundwater problems. The effects of nonhomogeneity and anisotropy can, however, be incorporated into an analysis under certain conditions.

3.3 GROUNDWATER GEOLOGY

The geologic framework of an area provides the most valuable guide to the occurrence and availability of groundwater. Rocks, the solid matter forming Earth's crust, are an assemblage of minerals. In the geologic sense, the term *rock* includes both the hard, consolidated formations and loose, unconsolidated materials. With respect to their origin, they fall into three broad categories: *igneous*, *metamorphic*, and *sedimentary*.

The two classes of igneous rocks, intrusive and extrusive, differ appreciably in their hydrologic properties. Fresh intrusive rocks are compact and, in general, not water bearing. They have very low porosities (less than 1%) and are almost impermeable. When fractured and jointed they may develop appreciable porosity and permeability within a few hundred feet of the surface. Permeability produced by fracturing of unweathered rocks generally ranges from 0.001 to 10.0 D, where D is darcy, which is the unit of permeability, named after Henry Darcy.[1] Extrusive or volcanic rocks can be good aquifers.

Metamorphic rocks are generally compact and highly crystalline. They are impervious and make poor aquifers.

Rocks may be grouped into hydrologic units on the basis of their ability to store and transmit water. An *aquifer* is a body of rock that acts as a hydrologic unit and is capable of transmitting significant quantities of water. An *aquiclude* is a rock formation that contains water but is not capable of transmitting it in significant amounts. Aquicludes usually form the boundaries of aquifers, although they are seldom absolute barriers to groundwater movement. They often contain considerable water in storage, and there is frequently some interchange between the free groundwater above an aquiclude and the confined aquifer below. Materials that have permeabilities intermediate between those of aquifers and aquicludes have been termed *aquitards*.

The boundaries of a geologic rock unit and the dimensions of an aquifer often do not correspond precisely. The latter are arrived at from the considerations of the degree of hydraulic continuity and from the position and character of hydrologic boundaries. An aquifer can thus be a geologic formation, a group of formations, or part of a formation.

Sedimentary formations include both consolidated, hard rocks (shale, sandstone, and limestone) and loose, unconsolidated materials (clay, gravel, and sand). Some sandstones may be almost impermeable, and others highly pervious. The degree of cementation plays a crucial role. Partially cemented or fractured sandstones have very high yields. Porosity of sandstones ranges from less than 5% to a maximum of about 30%. Permeability of medium-range sandstones generally varies from 1 to 500 mD (millidarcy).

Limestones vary widely in density, porosity, and permeability. When not deformed, they are usually dense and impervious. From the standpoint of water yield, secondary porosity produced as a result of fracturing and solution is more important than density and permeability. The nonuniform distribution of interstices in limestones over even short distances results because of marked differences in secondary porosity, which depends on local conditions. They are second only to sandstones as a source of groundwater. Limestones are prolific producers under suitable conditions.

Although consolidated rocks are important sources of water, the areas served by them in the United States are relatively small. Most developments lie in granular, unconsolidated sediments. Unconsolidated, sedimentary aquifers include (a) marine deposits, (b) river valleys, (c) alluvial fans, (d) coastal plains, (e) glacial outwash, and, to a much smaller degree, (f) dune sand. Materials deposited in seas are often extensive; sediments deposited on land by streams, ice, and wind are less extensive and are usually discontinuous.

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[1] One darcy is equal to the passage of 1 cm^3 of fluid of 1 centipose viscosity in 1 second (i.e., 1 mL/s) under a pressure differential of 1 atmosphere having an area of cross-section of 1 cm^2 and a length of 1 cm.

Sands and gravels are by far the best water-producing sediments. They have excellent water storage and transmission characteristics and are ordinarily so situated that replenishment is rapid, although extremely fine sands are of little value. Porosity, specific yield, and permeability depend on particle size, size distribution, packing configuration, and shape. Uniform or well-sorted sands and gravels are the most productive; mixed materials containing clay are least so. Boulder clay deposited beneath ice sheets is an example. Typical porosities lie between 25% and 65%. Gravel and coarse sands usually have specific yields greater than 20%.

Clays and silts are poor aquifers. They are highly porous but have very low permeabilities. However, the permeability is seldom zero. They are significant only when they (a) confine or impede the movement of water through more pervious soils and (b) supply water to aquifers through leakage by consolidation.

3.4 GROUNDWATER SITUATION IN THE UNITED STATES

Geologic and hydrologic conditions vary greatly in various parts of the United States. To permit useful generalizations about the occurrence and availability of groundwater, Thomas (1952) divided the United States into 10 major groundwater regions (Fig. 3.2). McGuiness (1963) provided an updated assessment of the groundwater situation in each of Thomas's regions and has also described the occurrence and development of groundwater in each of the states. The Water Resources Division (WRD) of the US Geological Survey is the principal agency of the federal government engaged in groundwater investigations. The published reports and the unpublished data of the WRD are indispensable to any groundwater investigation. In addition, many states have agencies responsible for activities in groundwater.

3.5 TYPES OF AQUIFERS

Because of the differences in the mechanism of flow, three types of aquifers are distinguished: (1) *unconfined* or *water table*, (2) *confined* or *artesian*, and (3) *semiconfined* or *leaky.*

Unconfined aquifers (also known as water table, *phreatic*, or *free* aquifers) are those in which the upper surface of the zone of saturation is under atmospheric pressure. This surface is free to rise and fall in response to the changes of storage in the saturated zone. The flow under such conditions is said to be unconfined. An imaginary surface connecting all rest or static levels in wells in an unconfined aquifer is its *water table* or *phreatic surface*. This defines the level in the zone of saturation, which is at atmospheric pressure. The

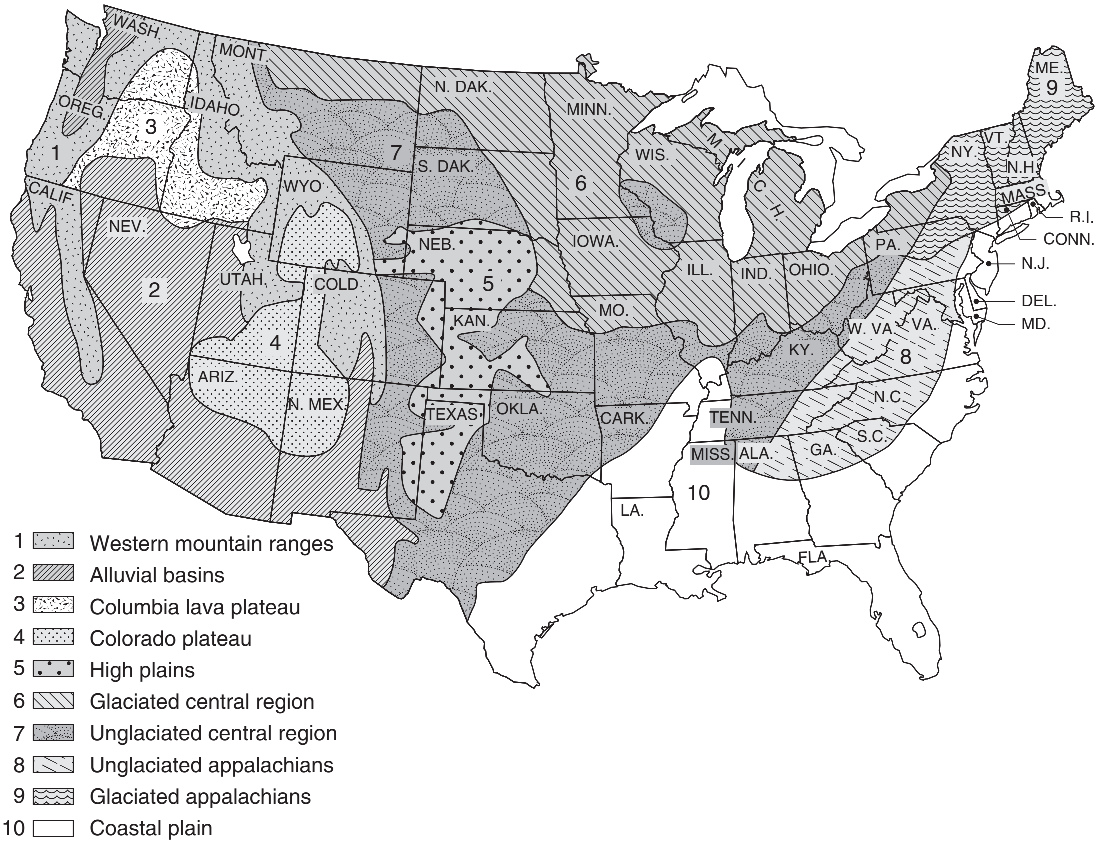


Figure 3.2 Major groundwater regions of the United States (After Thomas and McGuinness of USGS).

water held by capillary attraction at less than atmospheric pressure may fully saturate the interstices to levels above those observed in wells. Thus the upper limit of the zone of saturation and water table are not coincident. The capillary fringe may be significant for sediments with small interstices and low permeability, such as clay.

More than one zone of saturation occurs when an impervious or semipervious layer or lens in the zone of aeration supports a less extensive zone of saturation above the main water table, giving rise to the so-called *perched water table*.

If a porous stratum in the zone of saturation dips beneath an impervious layer, the flow is confined in much the same way as in a pipe that drops below the hydraulic grade line. There is no free surface in contact with the atmosphere in the area of confinement. The water level in a well tapping this confined or artesian aquifer will rise, under pressure, above the base of the confining layer to an elevation that defines the piezometric level. If the recharge areas are at a sufficiently high elevation, the pressure may be great enough to result in free-flowing wells or springs. An imaginary surface connecting the piezometric levels at all points in an artesian aquifer is called the *piezometric surface* (Fig. 3.1 depicts some of these terms). The rise and fall of water levels in artesian wells result primarily from changes in pressure rather than from changes in storage volume. The seasonal fluctuations are usually small compared with unconfined conditions.

Aquifers that are overlain or underlain by aquitards are called *leaky aquifers*. In natural materials, confining layers seldom form an absolute barrier to groundwater movement. The magnitude of flow through the semipervious layer is called *leakage*. Although the vertical permeability of the aquitard is very low and the movement of water through it extremely slow, leakage can be significant because of the large horizontal areas involved.

3.6 GROUNDWATER MOVEMENT

Groundwater in the natural state is constantly in motion. Its rate of movement under the force of gravity is governed by the frictional resistance to flow offered by the porous medium. The difference in head between any two points provides the driving force. Water moves from levels of higher energy potential (or head) to levels of lower energy potential, the loss in head being dissipated as heat. Because the magnitudes of discharge, recharge, and storage fluctuate with time, the head distribution at various locations is not stationary. Groundwater flow is both unsteady and nonuniform. Compared with surface water, the rate of groundwater movement is generally very slow. Low velocities and the small size of passageways give rise to very low *Reynolds numbers* and consequently the flow is almost always *laminar. Turbulent* flow may occur in cavernous limestones and volcanic rocks, where the passageways may be large, or in coarse gravels, particularly in the vicinity of a discharging well. Depending on the intrinsic permeability, the rate of movement can vary considerably within the same geologic formation. Flow tends to be concentrated in zones of higher permeability, that is, where the interstices are larger in size and have a better interconnection.

In aquifers of high yield, velocities of 5–60 ft/d (1.5–18.3 m/d) are associated with hydraulic gradients of 10–20 ft/mi (1.89 to 3.78 m/km). Underflow through gravel deposits may travel several hundred feet per day (m/d). Depending on requirements, flows as low as a few feet per year (m/year) may also be economically useful.

In homogeneous, isotropic aquifers, the dominant movement is in the direction of greatest slope of the water table or piezometric surface. Where there are marked nonhomogeneities and anisotropies in permeability, the direction of groundwater movement can be highly variable.

3.7 DARCY'S LAW

Although other scientists were the first to propose that the velocity of flow of water and other liquids through capillary tubes is proportional to the first power of the hydraulic gradient, credit for verification of this observation and for its application to the flow of water through natural materials or, more specifically, its filtration through sand must go to Darcy. The relationship known as *Darcy's law* may be written as

$$v = K(\mathrm{d}h/\mathrm{d}l) = KI \tag{3.1}$$

where v is the hypothetical, superficial or face velocity (darcy; not the actual velocity through the interstices of the soil) through the gross cross-sectional area of the porous medium; $I = \mathrm{d}h/\mathrm{d}l$ is the *hydraulic gradient*, or the loss of head per unit length in the direction of flow; and K is a constant of proportionality known as *hydraulic conductivity*, or the *coefficient of permeability*. The actual velocity, known as *effective velocity*, varies from point to point. The average velocity through pore space is given by

$$v_\mathrm{e} = KI/\theta \tag{3.2}$$

where θ is the effective porosity. Because I is a dimensionless ratio, K has the dimensions of velocity and is in fact the velocity of flow associated with a hydraulic gradient of unity.

The proportionality coefficient in Darcy's law, K, refers to the characteristics of both the porous medium and the fluid. By dimensional analysis:

$$K = Cd^2\gamma/\mu \tag{3.3}$$

where C is a dimensionless constant summarizing the geometric properties of the medium affecting flow, d is a representative pore diameter, μ is the viscosity, and γ is the specific weight of fluid. The product Cd^2 depends on the properties of the medium alone and is called the intrinsic or specific permeability of a water-bearing medium, $k = Cd^2$. It has the dimensions of area.

Hence,

$$K = k\gamma/\mu = k\rho g/\mu = kg/\nu \tag{3.4}$$

where ρ is the specific density and ν, the kinematic viscosity.

The fluid properties that affect the flow are viscosity and specific weight. The value of K varies inversely as the kinematic viscosity, ν, of the flowing fluid. The ratio of specific weight to viscosity is affected by changes in the temperature and salinity of groundwater. Measurements of K are generally referred to a standard water temperature such as 60°F or 15.5°C. The necessary correction factor for field temperatures other than standard is provided by the relationship

$$K_1/K_2 = \nu_1/\nu_2 \tag{3.5}$$

Most groundwaters have relatively constant temperatures, and this correction is usually ignored in practice and K is stated in terms of the prevailing water temperature. Special circumstances in which correction may be important include influent seepage into an aquifer from a surface-water body where temperature varies seasonally.

Darcy's law is applicable only to laminar flow, and there is no perceptible lower limit to the validity of the law. The volume rate of flow is the product of the velocity given by Darcy's law and the cross-sectional area A normal to the direction of motion. Thus,

$$Q = KA(\mathrm{d}h/\mathrm{d}l) \tag{3.6}$$

and solving for K,

$$K = Q/[A(\mathrm{d}h/\mathrm{d}l)] \tag{3.7}$$

Hydraulic conductivity may thus be defined as the volume of water per unit time flowing through a medium of unit cross-sectional area under a unit hydraulic gradient. In the standard coefficient used by the US Geological Survey, the rate of flow is expressed in gpd/ft^2 (m^3/d/m^2) under a hydraulic gradient of 1 ft/ft (m/m) at a temperature of 60°F (15.5°C). This unit is called the *meinzer*. For most natural aquifer materials, values of K fall in the range of 10–5000 meinzers.

3.8 AQUIFER CHARACTERISTICS

The ability of an aquifer to transmit water is characterized by its *coefficient of transmissivity*. It is the product of the saturated thickness of the aquifer, b, and the average value of the hydraulic conductivities in a vertical section of the aquifer, K. The transmissivity, $T = Kb$, gives the rate of flow of water through a vertical strip of an aquifer 1 ft wide extending the full saturated thickness of the aquifer under a unit of hydraulic gradient. It has the dimensions of (length)^2/time, that is, ft^2/d or gpd/ft. Equation (3.6) can be rewritten as

$$Q = TW(\mathrm{d}h/\mathrm{d}l) \tag{3.8}$$

where W is the width of flow.

The *coefficient of storage* is defined as the volume of water that a unit decline in head releases from storage in a vertical prism of the aquifer of unit cross-sectional area (Fig. 3.3).

The physical processes involved when the water is released from (or taken into) storage in response to head changes are quite different in cases in which free surface is present from those in which it is not. A confined aquifer remains saturated during the withdrawal of water. In the case of a confined aquifer the water is released from storage by virtue of two processes: (a) lowering of the water table in the recharge or intake area of the aquifer and (b) elastic response to pressure changes in the aquifer and its confining beds induced by the withdrawal of water. For this the storage coefficient is expressed as

$$S = \theta\gamma b[\beta + (\alpha/\theta)] \tag{3.9}$$

in which θ is the average porosity of the aquifer; γ is the specific weight of water; β is the compressibility of water; and α is the vertical compressibility of aquifer material. In most confined aquifers, storage coefficient values lie in the range of 0.00005–0.0005. These values are small and thus large pressure changes over extensive areas are required to develop substantial quantities of water.

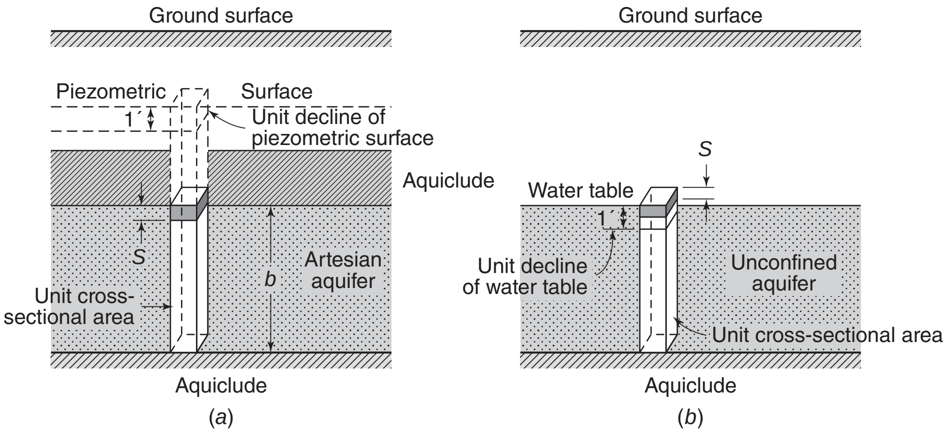


Figure 3.3 Graphical representation of storage coefficient. The volume of water that a unit decline in head releases from storage in a vertical prism of the aquifer of unit cross-sectional area. (a) Confined aquifer; (b) unconfined aquifer. Conversion factor: 1′ = 1 ft = 0.3048 m.

A confined aquifer for which S in Eq. (3.9) is 3×10^{-4} will release from 1 mi^2 64,125 gal (93,711 L/km^2) by lowering the piezometric surface by 1 ft (0.3048 m).

A water table aquifer also releases water from storage by two processes: (a) dewatering or drainage of material at the free surface as it moves downward and (b) elastic response of the material below the free surface. In general, the quantity released by elastic response is very small compared to the dewatering of the saturated material at the water table. Thus the storage coefficient is virtually equal to the specific yield of the material. In unconfined aquifers, the full complement of storage is usually not released instantaneously. The speed of drainage depends on the types of aquifer materials. Thus in water table aquifers, the storage coefficient varies with time, increasing at a diminishing rate. Ultimately it is equal to specific yield. Furthermore, since the dewatered portion of the aquifer cannot transmit water, transmissivity of the aquifer decreases with the lowering of the water table. Transmissivity is thus a function of head in an unconfined aquifer. The storage coefficient of unconfined aquifers may range from 0.01 to 0.3. A water table aquifer with a storage coefficient of 0.15 will release from a 1 mi^2 (2.59 km^2) area with an average decline in head of 1 ft (0.3048 m) $209 \times 10^6 \times 0.15$ gal = 31.30 MG (118.4 ML).

Hydraulic diffusivity is the ratio of transmissivity, T, to storage coefficient, S, or of permeability, K, to unit storage, S'. Where D is hydraulic diffusivity,

$$D = T/S = K/S' \tag{3.10}$$

In an unconfined aquifer, even if S is assumed constant, the diffusivity will vary with transmissivity, which varies with the position of the free surface.

The conductivity, the transmissivity, the storage coefficient, and the specific yield are usually referred to as *formation constants* and provide measures of the hydraulic properties of aquifers.

The capacity of an aquifer to transmit water can be measured by several methods:

1. Laboratory tests of aquifer samples
2. Tracer techniques
3. Analysis of water level maps
4. Aquifer tests

Laboratory measurements of hydraulic conductivity are obtained by using samples of aquifer material in either a constant-head or a falling-head permeameter. Undisturbed core samples are used in the case of well-consolidated materials and repacked samples in the case of unconsolidated materials. Observations are made of the time taken for a known quantity of water under a given head to pass through the sample. The application of Darcy's law enables hydraulic conductivity to be determined. The main disadvantage of this method arises from the fact that the values obtained are point measurements. Aquifers are seldom, if ever, truly homogeneous throughout their extent, and laboratory measurements are not representative of actual "in-place" values. Most samples of the material are taken in a vertical direction, whereas the dominant movement of water in the aquifer is nearly horizontal, and horizontal and vertical permeabilities differ markedly. Also, some disturbance is inevitable when the sample is removed from its environment. This method cannot, therefore, be used to give a reliable quantitative measure of hydraulic conductivity.

The measurement of hydraulic conductivity in undisturbed natural materials can be made by measurement of hydraulic gradient and determination of the speed of groundwater movement through the use of tracers. A tracer (dye, electrolyte, or radioactive substance) is introduced into the groundwater through an injection well at an upstream location, and measurements are made of the time taken by the tracer to appear in one or more downstream wells. Uranin, a sodium salt of fluorescein, is an especially useful dye because it remains visible in dilutions of $1{:}(14 \times 10^7)$ without a fluoroscope and $1{:}10^{10}$ with one. Tritium has been used as a radioactive tracer.

The time of arrival is determined by visual observation or colorimetry when dyes are added, by titration or electrical conductivity when salt solutions are injected, or by a Geiger or scintillation counter when radioactive tracers are used. The distance between the wells divided by the time required for half the recovered substance to appear is the median velocity. The observed velocity is the actual average rate of motion through the interstices of the aquifer material. The face velocity can be calculated, if effective porosity is known. The application of Darcy's law enables the hydraulic conductivity to be computed. The problems of direction of motion, dispersion and molecular diffusion, and the slow movement of groundwater limit the applicability of this method. The method is impractical for a heterogeneous aquifer that has large variations in horizontal and vertical hydraulic conductivity.

The drop in head between two equipotential lines in an aquifer divided by the distance traversed by a particle of water moving from a higher to a lower potential determines the hydraulic gradient. Changes in the hydraulic gradient may arise from either a change in flow rate, Q, hydraulic conductivity, K, or aquifer thickness, b (Eq. 3.6). If no water is being added to or lost from an aquifer, the steepening of the gradient must be due to lower transmissivity, reflecting either a lower permeability, a reduction in thickness, or both (Eq. 3.8).

Of the currently available methods for the estimation of formation constants, aquifer tests (also called pumping tests) are the most reliable. The mechanics of a test involve the pumping of water from a well at a constant discharge rate and the observation of water levels in observation wells at various distances from the pumping well at different time intervals after pumping commences. The analysis of a pumping test comprises the graphical fitting of the various theoretical equations of groundwater flow to the observed data. The mathematical model giving the best fit is used for the estimation of formation constants. The main advantages of this method are that the sample used is large and remains undisturbed in its natural surroundings. The time and expense are reasonable. The main disadvantage of the method concerns the number of assumptions that must be made when applying the theory to the observed data. Despite the restrictive assumptions, pumping tests have been successfully applied under a wide range of conditions actually encountered.

3.9 WELL HYDRAULICS

Well hydraulics deals with predicting yields from wells and in forecasting the effects of pumping on groundwater flow and on the distribution of potential in an aquifer. The response of an aquifer to pumping depends on the type of aquifer (confined, unconfined, or leaky), aquifer characteristics (transmissivity, storage coefficient, and leakage), aquifer boundaries, and well construction (size, type, whether fully or partially penetrating) and well operation (constant or variable discharge, continuous or intermittent pumping).

The first water pumped from a well is derived from aquifer storage in the immediate vicinity of the well. Water level (i.e., piezometric surface or water table) is lowered and a *cone of depression* is created. The shape of the cone is determined by the hydraulic gradients required to transmit water through aquifer material toward the pumping well. The distance through which the water level is lowered is called the *drawdown*. The outer boundary of the drawdown curve defines the *area of influence* of the well. As pumping is continued, the shape of the cone changes as it travels outward from the well. This is the dynamic phase, in which the flow is *time dependent* (*nonsteady*) and both the velocities and water levels are changing. With continued withdrawals, the shape of the cone of depression stabilizes near the well and, with time, this condition progresses to greater distances. Thereafter the cone of depression moves parallel to itself in this area. This is the depletion phase. Eventually the drawdown curve may extend to the areas of natural discharge or recharge. A new state of equilibrium is reached if the natural discharge is decreased or the natural recharge is increased by an amount equal to the rate of withdrawal from the well. A *steady state* is then reached and the water level ceases to decline.

3.10 NONSTEADY RADIAL FLOW

Solutions have been developed for nonsteady radial flow toward a discharging well. Pumping test analyses for the determination of aquifer constants are based on solutions of unsteady radial flow equations.

3.10.1 Confined Aquifers

In an effectively infinite artesian aquifer, the discharge of a well can only be supplied through a reduction of storage within the aquifer. The propagation of the area of influence and the rate of decline of head depend on the hydraulic diffusivity of the aquifer. The differential equation governing nonsteady radial flow to a well in a confined aquifer is given by

$$\frac{\partial^2 h}{\partial r^2} + \frac{1}{r}\frac{\partial h}{\partial r} = \frac{S}{T}\frac{\partial h}{\partial t} \qquad \textbf{(3.11)}$$

Using an analogy to the flow of heat to a sink, Theis (1963) derived an expression for the drawdown in a confined aquifer due to the discharge of a well at a constant rate. His equation is really a solution of Eq. (3.11) based on the following assumptions: (a) the aquifer is homogeneous, isotropic, and of infinite areal extent; (b) transmissivity is constant with respect to time and space; (c) water is derived entirely from storage, being released instantaneously with the decline in head; (d) storage coefficient remains constant with time; and (e) the well penetrates, and receives water from, the entire thickness of the aquifer. The Theis equation may be written as follows:

$$s = h_0 - h = \frac{Q}{4\pi T}\int_{\frac{r^2 S}{4Tt}}^{\infty} \frac{e^{-u}}{u}\,\mathrm{d}u = \frac{Q}{4\pi T}W(u) \quad \text{(SI units)} \qquad \textbf{(3.12)}$$

where h is the head at a distance r from the well at a time t after the start of pumping; h_0 is the initial head in the aquifer prior to pumping; Q is the constant discharge of the well; S is the storage coefficient of the aquifer; and T is the transmissivity of the aquifer. The integral in the above expression is known as the exponential integral and is a function of its lower limit. In groundwater literature, it is written symbolically as $W(u)$, which is read "well function of u," where

$$u = (r^2 S)/(4\,Tt) \quad \text{(SI units)} \qquad \textbf{(3.13)}$$

The drawdown s (m) at a distance r (m) at time t (days) after the start of pumping for a constant discharge Q (m^3/d) under the transmissivity T (m^3/d/m) is given by Eqs. (3.12) and (3.13).

Its value can be approximated by a convergent infinite series:

$$W(u) = -0.5772 - \ln u + u - u^2/2 \times 2! + u^3/3 \times 3! \cdots \qquad \textbf{(3.14)}$$

Values of $W(u)$ for a given value of u are tabulated in numerous publications. A partial listing is given in Table 3.2.

The drawdown s (ft), at a distance r (ft), at time t (days) after the start of pumping for a constant discharge Q (gpm) is given by

$$s = h_0 - h = 1{,}440\,QW(u)/(4\pi T) = 114.6\,QW(u)/T \quad \text{(US customary units)} \qquad \textbf{(3.15)}$$

Table 3.2 Values of the well function $W(u)$ for various values of u

	u							
N	$N \times 10^{-15}$	$N \times 10^{-14}$	$N \times 10^{-13}$	$N \times 10^{-12}$	$N \times 10^{-11}$	$N \times 10^{-10}$	$N \times 10^{-9}$	$N \times 10^{-8}$
1.0	33.96	31.66	29.36	27.05	24.75	22.45	20.15	17.84
1.5	33.56	31.25	28.95	26.65	24.35	22.04	19.74	17.44
2.0	33.27	30.97	28.66	26.36	24.06	21.67	19.45	17.15
2.5	33.05	30.74	28.44	26.14	23.83	21.53	19.23	16.93
3.0	32.86	30.56	28.26	25.96	23.65	21.35	19.05	16.75
3.5	32.71	30.41	28.10	25.80	23.50	21.20	18.89	16.59
4.0	32.56	30.27	27.97	25.67	23.36	21.06	18.76	16.46
4.5	32.47	30.15	27.85	25.55	23.25	20.94	18.64	16.34
5.0	32.35	30.05	27.75	25.44	23.14	20.84	18.54	16.23
5.5	32.26	29.95	27.65	25.35	23.05	20.74	18.44	16.14
6.0	32.17	29.87	27.56	25.26	22.96	20.66	18.35	16.05
6.5	32.09	29.79	27.48	25.18	22.88	20.58	18.27	15.97
7.0	32.02	29.71	27.41	25.11	22.81	20.50	18.20	15.90
7.5	31.95	29.64	27.34	25.04	22.74	20.43	18.13	15.83
8.0	31.88	29.58	27.28	24.97	22.67	20.37	18.07	15.76
8.5	31.82	29.52	27.22	24.91	22.61	20.31	18.01	15.70
9.0	31.76	29.46	27.16	24.86	22.55	20.25	17.95	15.65
9.5	31.71	29.41	27.11	24.80	22.50	20.20	17.89	15.59

	u							
N	$N \times 10^{-7}$	$N \times 10^{-6}$	$N \times 10^{-5}$	$N \times 10^{-4}$	$N \times 10^{-3}$	$N \times 10^{-2}$	$N \times 10^{-1}$	N
1.0	15.54	13.24	10.94	8.633	6.332	4.038	1.823	2.194×10^{-1}
1.5	15.14	12.83	10.53	8.228	5.927	3.637	1.465	1.000×10^{-1}
2.0	14.85	12.55	10.24	7.940	5.639	3.355	1.223	4.890×10^{-2}
2.5	14.62	12.32	10.02	7.717	5.417	3.137	1.044	2.491×10^{-2}
3.0	14.44	12.14	9.837	7.535	5.235	2.959	0.9057	1.305×10^{-2}
3.5	14.29	11.99	9.683	7.381	5.081	2.810	0.7942	6.970×10^{-3}
4.0	14.15	11.85	9.550	7.247	4.948	2.681	0.7024	3.779×10^{-3}
4.5	14.04	11.73	9.432	7.130	4.831	2.568	0.6253	2.073×10^{-3}
5.0	13.93	11.63	9.326	7.024	4.726	2.468	0.5598	1.148×10^{-3}
5.5	13.84	11.53	9.231	6.929	4.631	2.378	0.5034	6.409×10^{-4}
6.0	13.75	11.45	9.144	6.842	4.545	2.295	0.4544	3.601×10^{-4}
6.5	13.67	11.37	9.064	6.762	4.465	2.220	0.4115	2.034×10^{-4}
7.0	13.60	11.29	8.990	6.688	4.392	2.151	0.3738	1.155×10^{-4}
7.5	13.53	11.22	8.921	6.619	4.323	2.087	0.3403	6.583×10^{-5}
8.0	13.46	11.16	8.856	6.555	4.259	2.027	0.3106	3.767×10^{-5}
8.5	13.40	11.10	8.796	6.494	4.199	1.971	0.2840	2.162×10^{-5}
9.0	13.34	11.04	8.739	6.437	4.142	1.919	0.2602	1.245×10^{-5}
9.5	13.29	10.99	8.685	6.383	4.089	1.870	0.2387	7.185×10^{-6}

Source: After U.S. Geological Survey.

where

$$u = 1.87(S/T)(r^2/t) \quad \text{(US customary units)} \qquad \textbf{(3.16)}$$

and T is transmissivity in gpd/ft.

The equation can be solved for any one of the quantities involved if other parameters are given. The solution for drawdown, discharge, distance from the well, or time is straightforward. The solution for transmissivity T is difficult, because it occurs both inside and outside the integral. Theis (1963) devised a graphical method of superposition to obtain a solution of the equation for T and S.

If the discharge Q is known, the formation constants of an aquifer can be obtained as follows:

1. Plot the field or data curve with drawdown, s, as the ordinate and r^2/t as the abscissa on logarithmic coordinates on a transparent paper.
2. Plot a "type curve" with the well function, $W(u)$, as the ordinate and its argument u as the abscissa on logarithmic coordinates using the same scale as the field curve.
3. Superimpose the curves shifting vertically and laterally, keeping the coordinate axes parallel until most

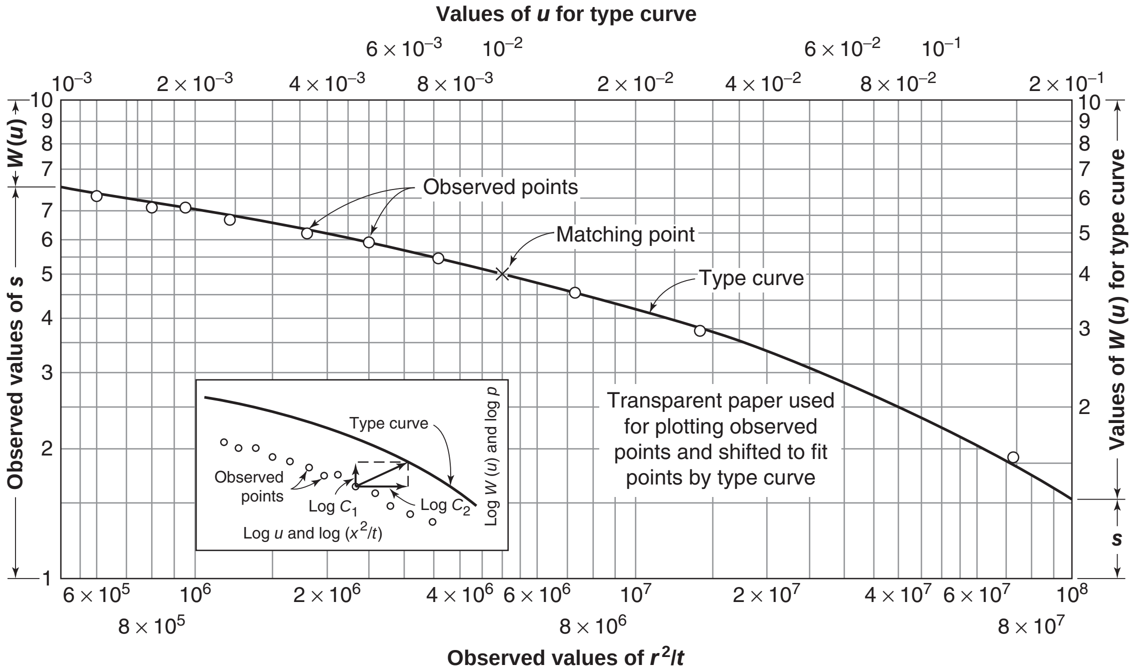


Figure 3.4 Theis-type curve determination of the formation constants of a well field using the US customary system (Data by courtesy of the US Geological Survey). Conversion factor: (r^2/t of US customary units) × 0.0929 = (r^2/t of SI units).

of the plotted points of the observed data fall on a segment of the type curve.

4. Select a convenient matching point anywhere on the overlapping portion of the sheets and record the coordinates of this common point on both graphs.
5. Use the two ordinates, s and $W(u)$, to obtain the solution for transmissivity T from Eq. (3.15).
6. Use the two abscissas, r^2/t and u, together with the value of T, to obtain the solution for the storage coefficient S from Eq. (3.16).

Values of s are related to the corresponding values of $W(u)$ by the constant factor $114.6Q/T$, whereas values of r^2/t are related to corresponding values of u by the constant factor $T/1.87S$). Thus when the two curves are superimposed, corresponding vertical axes are separated by a constant distance proportional to $\log[114.6(Q/T)] = \log C_1$ whereas the corresponding horizontal axes are separated by a constant distance proportional to $\log[T/(1.87S)] = \log C_2$ as shown in Fig. 3.4.

EXAMPLE 3.1 CALCULATION OF THE FORMATION CONSTANTS *T* AND *S* USING THE THEIS METHOD

The observed data from a pumping test are shown plotted in Fig. 3.4 along with a Theis-type curve, as if the transparency of the observed data had been moved into place over the type curve. The observation well represented by the data is 225 ft (68.58 m) from a pumping well where the rate of discharge is 350 gpm (1324.75 L/min; 1,908 m^3/d). Calculate the formation constants T and S.

Solution 1 (US Customary System):

The match-point coordinates are

$$W(u) = 4.0$$

$$s = 5.0\,\text{ft}\,(1.524\,\text{m})$$

$$u = 10^{-2}$$

$$r^2/t = 5 \times 10^6\ (4.645 \times 10^5 \text{ for SI units}).$$

Compute the formation constants:

$$T = 114.6QW(u)/s$$

$$T = 114.6 \times 350 \times 4.0/5.0$$

$$\mathbf{T = 3.2 \times 10^4\ gpd/ft}$$

$$S = uT/(1.87\, r^2/t)$$

$$S = 10^{-2}\,(3.2 \times 10^4)/[1.87(5 \times 10^6)]$$

$$\mathbf{S = 3.4 \times 10^{-5}}.$$

Solution 2 (SI System):

$$T = [Q/(4\pi s)]W(u)$$

$$T = [1808/4 \times 3.14 \times 1.524)] \times 4$$

$$\mathbf{T = 398.72\ m^3/d/m}$$

$$S = 4\,Tu/(r^2/t)$$

$$S = 4 \times 398.72 \times 10^{-2}/(4.645 \times 10^5)$$

$$\mathbf{S = 3.43 \times 10^{-5}}.$$

EXAMPLE 3.2 CALCULATIONS OF DRAWDOWN WITH TIME IN A WELL

In the aquifer represented by the pumping test in Example 3.1, a gravel-packed well with an effective diameter of 24 in. (610 mm) is to be constructed. The design flow of the well is 700 gpm (3,815 m^3/d). Calculate the drawdown at the well with total withdrawals from storage (i.e., with no recharge or leakages) after (a) 1 minute, (b) 1 hour, (c) 8 hours, (d) 24 hours, (e) 30 days, and (f) 6 months of continuous pumping, at design capacity.

Solution 1 (US Customary System):

$$\text{From Eq. (3.15): } s = [114.6\,Q/T][W(u)]$$

$$= [114.6 \times 700/(3.2 \times 10^4)][W(u)]$$

$$= 2.51\ \text{ft}\,[W(u)].$$

$$\text{From Eq. (3.16): } u = 1.87\, r^2 S/Tt$$

$$= 1.87 \times 1^2 \times 3.4 \times 10^{-5}/(3.2 \times 10^4)t$$

$$= (2.0 \times 10^{-9})/t.$$

For various values of t, compute u, then from Table 3.2 obtain the well function, $W(u)$, for the calculation of drawdown. **The values of drawdown for various values of time are given in Table 3.3**.

Table 3.3 Variation of drawdown with time for Example 3.2

	Time (days)	u	$W(u)$	Drawdown, s, ft	(m)
(a)	1/1,440	2.86×10^{-6}	12.19	30.6	(9.33)
(b)	1/24	4.8×10^{-8}	16.27	40.8	(12.44)
(c)	1/3	6.0×10^{-9}	18.35	46.0	(14.02)
(d)	1	2.0×10^{-9}	19.45	48.8	(14.87)
(e)	30	6.6×10^{-11}	22.86	57.3	(17.46)
(f)	180	1.1×10^{-11}	24.66	61.8	(18.84)

Solution 2 (SI Units):

$$\begin{aligned}\text{From Eq. (3.12): } s &= [Q/(4\pi T)][W(u)]\\ &= [3{,}815/(4 \times 3.14 \times 398.72)][W(u)]\\ &= 0.72\,\text{m}[W(u)].\\ \text{From Eq. (3.13): } u &= r^2 S/(4Tt)\\ &= (0.3048)^2 \times 3.43 \times 10^{-5}/(4 \times 398.72t)\\ &= (2.0 \times 10^{-9})/t.\end{aligned}$$

For various values of t, compute u, then from Table 3.2 obtain the well function, *W(u)*, for the calculation of drawdown. **The values of drawdown for various values of time are given in Table 3.3**.

3.10.2 Semilogarithmic Approximation

It was recognized that when u is small, the sum of the terms beyond ln u in the series expansion of $W(u)$ (Eq. 3.14) is relatively insignificant. The Theis equation (Eq. 3.12) then reduces to

$$s = [Q/(4\pi T)]\{\ln[(4Tt)/(r^2S)] - 0.5772\}$$

$$s = Q/(4\pi T)\ln[(2.25Tt)/(r^2S)] \quad \text{(SI units)} \qquad \textbf{(3.17)}$$

where Q is in m^3/d, T in m^3/d/m, t in days, and r in m.

When Q is in gpm, T in gpd/ft, t in days, and r in ft, the equation becomes

$$s = 264(Q/T)\log[(0.3Tt)/(r^2 S)] \quad \text{(US customary units)} \qquad \textbf{(3.18)}$$

A graphical solution was proposed for this equation. If the drawdown is measured in a particular observation well (fixed r) at several values of t, the equation becomes

$$s = 264(Q/T)\log(Ct)$$

where

$$C = 0.3\,T/(r^2S)$$

If, on semilogarithmic paper, the values of drawdown are plotted on the arithmetic scale and time on the logarithmic scale, the resulting graph should be a straight line for higher values of t where the approximation is valid. The graph is referred to as the time–drawdown curve. On this straight line an arbitrary choice of times t_1 and t_2 can be made and the corresponding values of s_1 and s_2 recorded. Inserting these values in Eq. (3.18), we obtain

$$s_2 - s_1 = 264(Q/T)\log(t_2/t_1) \qquad \textbf{(3.19)}$$

Solving for T,

$$T = 264\,Q\log(t_2/t_1)/(s_2 - s_1) \qquad \textbf{(3.20)}$$

Thus transmissivity is inversely proportional to the slope of the time–drawdown curve. For convenience, t_1 and t_2 are usually chosen one log cycle apart. Equation (3.20) then reduces to

$$T = 264\,Q/\Delta s \quad \text{(US customary units)} \qquad \textbf{(3.21a)}$$

where T is the transmissivity, in gpd/ft; Q is the well flow, in gpm; and Δs is the change in drawdown, in ft, over one log cycle of time.

An equivalent equation using the SI units is

$$T = 0.1833Q/\Delta s \quad \text{(SI units)} \qquad \textbf{(3.21b)}$$

where T is the transmissivity, in m^3/d/m; Q is the well flow, in m^3/d; and Δs is the change in drawdown, in m, over one log cycle of time.

The coefficient of storage of the aquifer can be calculated from the intercept of the straight line on the time axis at zero drawdown, provided that time is converted to days. For zero drawdown, Eq. (3.18) gives

$$0 = 264(Q/T)\log[0.3Tt_0/(r^2S)]$$

that is,

$$0.3Tt_0/(r^2S) = 1$$

which gives

$$S = 0.3Tt_0/r^2 \quad \text{(US customary units)} \qquad \textbf{(3.22a)}$$

where S is the coefficient of storage of an aquifier, dimensionless; T is the transmissivity, gpd/ft; t_0 is the time at zero drawdown, d; and r is the distance between an observation well and a pumping well, ft.

An equivalent equation using the SI units is

$$S = 2.24Tt_0/r^2 \quad \text{(SI units)} \qquad \textbf{(3.22b)}$$

where S is the coefficient of storage of an aquifier, dimensionless; T is the transmissivity, m^3/d/m; t_0 is the time at zero drawdown, d; and r is the distance between an observation well and a pumping well, *m*.

EXAMPLE 3.3 DETERMINATION OF THE *T* AND *S* COEFFICIENTS OF AN AQUIFER USING THE APPROXIMATION METHOD

A time–drawdown curve for an observation well at a distance of 225 ft (68.6 m) from a pumping well discharging at a constant rate of 350 gpm (1907.5 m^3/d) is shown in Fig. 3.5. Determine the transmissivity and storage coefficient of the aquifer.

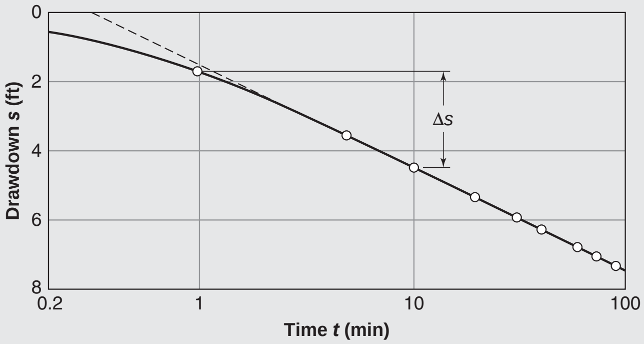


Figure 3.5 Time–drawdown curve (Data by courtesy of the US Geological Survey). Conversion factor: 1 ft = 0.3048 m.

Solution 1 (US Customary System):

To determine the slope of the straight-line portion, select two points one log cycle apart, namely,

$$t_1 = 1 \text{ min}; \quad s_1 = 1.6 \text{ ft} \, (0.49 \text{ m}).$$

$$t_2 = 10 \text{ min}; \quad s_2 = 4.5 \text{ ft} \, (1.37 \text{ m}).$$

The slope of the line per log cycle is $\Delta s = 4.5 - 1.6 = 2.9$ ft (0.88 m). The line intersects the zero drawdown axis at $t_0 = 0.3$ min. The transmissivity and the storage coefficient of the aquifers are

$$\begin{aligned} T &= 264Q/\Delta s \\ &= 264 \times 350/2.9 \\ &= \mathbf{3.2 \times 10^4 \text{ gpd/ft}} \, (\mathbf{397 \text{ m}^3/\text{d}/\text{m}}). \\ S &= 0.3\, Tt_0/r^2 \\ &= 0.3(3.2 \times 10^4)[0.3/(60 \times 24)]/(225)^2 \\ &= \mathbf{4.0 \times 10^{-5}}. \end{aligned}$$

Solution 2 (SI System):

A time–drawdown curve shown in Fig. 3.5 is used, except that the drawdown *s* (m) versus the time (min) should be plotted instead.

To determine the slope of the straight-line portion, select two points one log cycle apart, namely,

$$t_1 = 1 \text{ min}; \quad s_1 = (0.49 \text{ m}).$$

$$t_2 = 10 \text{ min}; \quad s_2 = (1.37 \text{ m}).$$

The slope of the line per log cycle is $\Delta s = 1.37 \text{ m} - 0.49 \text{ m} = (0.88 \text{ m})$. The line intersects the zero drawdown axis at $t_0 = 0.3 \text{ min} = 0.3/(60 \times 24)d$. The transmissivity and the storage coefficient of the aquifers are

$$\begin{aligned} \mathbf{T} &= 0.1833Q/\Delta s \\ &= 0.1833 \times 1907.5/0.88 \\ &= \mathbf{397.3 \text{ m}^3/\text{d}/\text{m}}. \\ \mathbf{S} &= 2.24Tt_0/r^2 \\ &= 2.24(397.3)[0.3/(60 \times 24)]/68.6^2 \\ &= \mathbf{4 \times 10^{-5}}. \end{aligned}$$

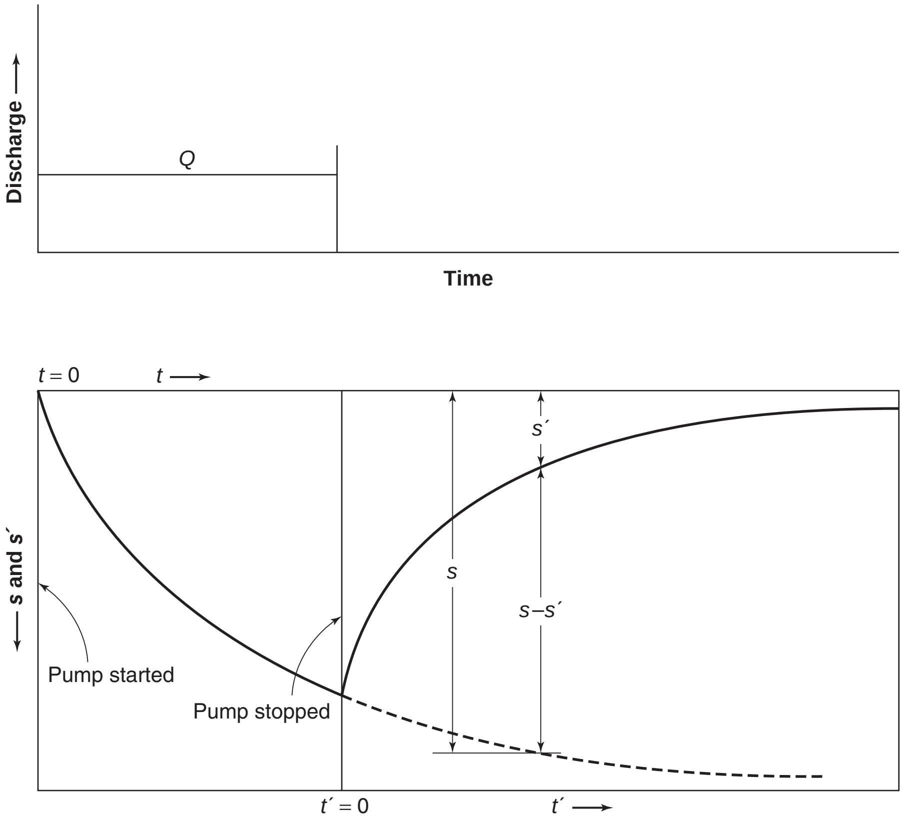


Figure 3.6 Water level recovery after pumping has stopped.

Equation (3.18) can also be used if the drawdown is measured at several observation wells at essentially the same time, that is, from the shape of the cone of depression. Drawdowns are plotted on the arithmetic scale and distance on the log scale and the resulting straight-line graph is called the distance–drawdown curve. It can be shown that the expressions for T and S in this case are

$$T = 528Q/\Delta s \qquad \textbf{(3.23)}$$

$$S = 0.3Tt/r_0^2 \qquad \textbf{(3.24)}$$

With the formation constants T and S known, Eq. (3.18) gives the drawdown for any desired value of r and t, provided that u (Eq. 3.13) is less than 0.01. The value of u is directly proportional to the square of the distance and inversely proportional to time t. The combination of time and distance at which u passes the critical value is inversely proportional to the hydraulic diffusivity of the aquifer, $D = T/S$. The critical value of u is reached much more quickly in confined aquifers than in unconfined aquifers.

3.10.3 Recovery Method

In the absence of an observation well, transmissivity can be determined more accurately by measuring the recovery of water levels in the well under test after pumping has stopped than by measuring the drawdown in the well during pumping. For this purpose, a well is pumped for a known period of time, long enough to be drawn down appreciably. The pump is then stopped, and the rise of water level within the well (or in a nearby observation well) is observed (Fig. 3.6). The drawdown after the shutdown will be the same as if the discharge had continued at the rate of pumping and a recharge well with the same flow had been superimposed on the discharge well at the instance the discharge was shut down. The residual drawdown, s', can be found from Eq. (3.15) as follows:

$$s' = (114.6Q/T)[W(u) - W(u')] \qquad \textbf{(3.25)}$$

where

$$u = 1.87r^2S/4Tt$$

$$u' = 1.87r^2S/(4Tt')$$

where r is the effective radius of the well (or the distance to the observation well), t is the time since pumping started, and t' is the time since pumping stopped. For small values of r and large values of t, the residual drawdown may be obtained from Eq. (3.19):

$$s' = (264Q/T)\log(t/t') \text{ (US customary units)} \qquad \textbf{(3.26a)}$$

Solving for T,

$$T = (264Q/s')\log(t/t') \quad \text{(US customary units)} \qquad \textbf{(3.27a)}$$

Plotting s' on an arithmetic scale and t/t' on a logarithmic scale, a straight line is drawn through the observations. The coefficient of transmissivity can be determined from the slope of the line, or for convenience, the change of residual drawdown over one log cycle can be used as

$$T = 264Q/\Delta s' \quad \text{(US customary units)} \qquad \textbf{(3.28a)}$$

where the US customary units are: s' (ft), Q (gpm), T (gpd/ft), t (min), t' (min), and $\Delta s'$ (ft). The comparable equations using the SI units of s' (m), Q (m^3/d), T (m^3/d/m), t (min), t' (min), and $\Delta s'$ (m) are

$$s' = (0.1833Q/T)\log(t/t') \quad \text{(SI units)} \qquad \textbf{(3.26b)}$$

$$T = (0.1833Q/s')\log(t/t') \quad \text{(SI units)} \qquad \textbf{(3.27b)}$$

$$T = 0.1833Q/\Delta s' \quad \text{(SI units)} \qquad \textbf{(3.28b)}$$

Strictly speaking, the Theis equation and its approximations are applicable only to situations that satisfy the assumptions used in their derivation. They undoubtedly also provide reasonable approximations in a much wider variety of conditions than their restrictive assumptions would suggest. Significant departures from the theoretical model will be reflected in the deviation of the test data from the type curves. Advances have recently been made in obtaining analytical solutions for anisotropic aquifers, for aquifers of variable thickness, and for partially penetrating wells.

3.10.4 Unconfined Aquifers

The partial differential equation governing nonsteady unconfined flow is nonlinear in h. In many cases, it is difficult or impossible to obtain analytical solutions to the problems of unsteady unconfined flow. A strategy commonly used is to investigate the conditions under which a confined flow equation would provide a reasonable approximation for the head distribution in an unconfined aquifer. These conditions are that (a) the drawdown at any point in the aquifer must be small relative to the total saturated thickness of the aquifer and (b) the vertical head gradients must be negligible. This implies that the downward movement of the water table should be very slow, that is, that sufficient time must elapse for the flow to become stabilized in a portion of the cone of depression. The minimum duration of pumping depends on the properties of the aquifer.

The observed drawdown s, if large compared to the initial depth of flow h_0, should be reduced by a factor $s^2/2h_0$ to account for the decreased thickness of flow due to dewatering before Eq. (3.15) can be applied. For an observation well at a distance greater than $0.2h_0$, the minimum duration of pumping beyond which the approximation is valid is given as

$$t_{\min} = 37.4h_0/K \quad \text{(US customary units)} \qquad \textbf{(3.29a)}$$

where t is time in days, h_0 is the saturated aquifer thickness in ft, and K is hydraulic conductivity in gpd/ft^2.

$$t_{\min} = 5h_0/K \quad \text{(SI units)} \qquad \textbf{(3.29b)}$$

where $t_{\min}$ is time in days, h_0 is saturated aquifer thickness in m, and K is hydraulic conductivity in m^3/d/m^2.

3.10.5 Leaky Aquifers

The partial differential equation governing nonsteady radial flow toward a steadily discharging well in a leaky confined aquifer is

$$\frac{\partial^2 s}{\partial r^2} + \frac{1}{r}\frac{\partial s}{\partial r} - \frac{s}{B^2} = \frac{S}{T}\frac{\partial s}{\partial t} \qquad \textbf{(3.30a)}$$

where

$$B = \sqrt{\frac{T}{K'/b'}} \qquad \textbf{(3.30b)}$$

and s is the drawdown at a distance r from the pumping well; T and S are the transmissivity and storage coefficient of the lower aquifer, respectively; and K' and b' are the vertical permeability and thickness of the semipervious confining layer, respectively. The solution in an abbreviated form is given as

$$s = 114.6Q/T[W(u, r/B)] \qquad \textbf{(3.31)}$$

where

$$W(u, r/B) = \int_u^\infty \frac{1}{y} \exp \frac{-y - r^2}{4B^2 y} \mathrm{d}y$$

and

$$u = 1.87r^2 S/Tt \qquad \textbf{(3.32)}$$

Here, $W(u, r/B)$ is the well function of the leaky aquifer, Q is the constant discharge of the well in gpm, T is transmissivity in gpd/ft, and t is the time in days.

In the earlier phases of the transient state, that is, at very small values of time, the system acts like an ideal elastic artesian aquifer without leakage and the drawdown pattern closely follows the Theis-type curve. As time increases, the drawdown in the leaky aquifer begins to deviate from the Theis curve. At large values of time, the solution approaches the steady-state condition. With time, the fraction of well discharge derived from storage in the lower aquifer decreases and becomes negligible at large values of time as steady state is approached.

The solution to the above equation is obtained graphically by the match-point technique described for the Theis solution. On the field curve drawdown versus time is plotted on logarithmic coordinates. On the type curve the values of $W(u, r/B)$ versus $1/u$ are plotted for various values of r/B as shown in Fig. 3.7. The curve corresponding to the value of r/B giving the best fit is selected. From the match-point coordinates s and $W(u, r/B)$, T can be calculated by substituting

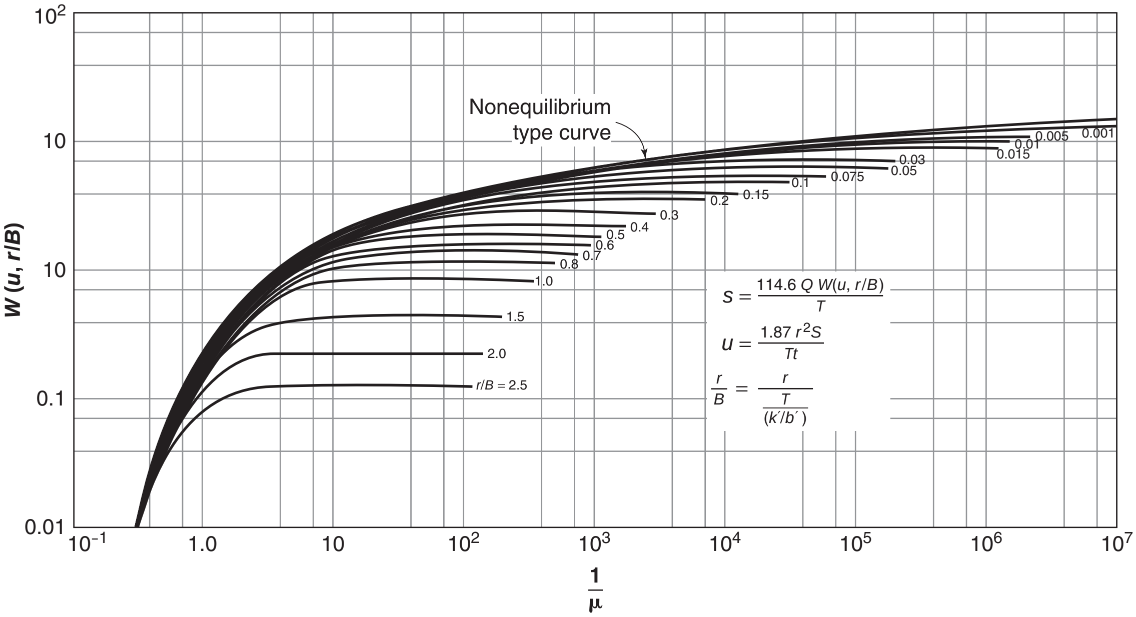


Figure 3.7 Nonsteady-state leaky artesian-type curves. Using the US customary system (After Walton of Illinois State Water Survey).

in Eq. (3.31). From the other two match-point coordinates and the value of T computed above, S is determined from Eq. (3.32). If b' is known, the value of the vertical permeability of the aquitard can be computed from Eq. (3.30), knowing r/B and T. Values of $W(u, r/B)$ for the practical range of u and r/B are given in Table 3.4.

3.11 PREDICTION OF DRAWDOWN

Predictions of drawdowns are useful when a new well field is to be established or where new wells are added to an existing field. To predict drawdowns, T, S, and proposed pumping rates must be known. Any of the several equations can be used. The Theis equation is of quite general applicability. The approximation does not accurately show drawdowns during the first few hours or first few days of withdrawals ($u > 0.01$). Because the equations governing flow are linear, the principle of superposition is valid.

3.11.1 Constant Discharge

Example 3.3 illustrated a method that can be used to evaluate the variation in drawdown with time, whereas the following example, Example 3.4, illustrates the variation in drawdown with distance when the pumping rate is constant.

EXAMPLE 3.4 DETERMINATION OF THE PROFILE OF A CONE OF DEPRESSION

Determine the profile of a quasi-steady-state cone of depression for a proposed 24 in. (61 cm) well pumping continuously at (a) 150 gpm, (b) 200 gpm, and (c) 250 gpm in an elastic artesian aquifer having a transmissivity of 10,000 gpd/ft and a storage coefficient of 6×10^{-4}. Assume that the discharge and recharge conditions are such that the drawdowns will be stabilized after 180 days.

Solution:

The distance at which drawdown is approaching zero, that is, the radius of cone of depression, can be obtained from Eq. (3.24):

$$r_0^2 = 0.3Tt/S = 0.3Dt \text{ where } D \text{ is the diffusivity of the aquifer} = T/S$$

$$= 0.3(1 \times 10^4)180/(6 \times 10^{-4}) = 9 \times 10^8$$

$$r_0 = (0.3Dt)^{0.5} = 3 \times 10^4 \text{ ft} \, (0.914 \times 10^4 \text{ m}).$$

This is independent of Q and depends only on the diffusivity of the aquifer. The change in drawdown per log cycle from Eq. (3.23) is

$$\Delta s = 528\,Q/T$$

$$\text{For 150 gpm: } \Delta s_1 = 528 \times 150/1 \times 10^4 = \mathbf{7.9\,ft\,(2.4\,m)}$$

$$\text{For 200 gpm: } \Delta s_2 = \mathbf{10.6\,ft\,(3.23\,m)}$$

$$\text{For 250 gpm: } \Delta s_3 = \mathbf{13.2\,ft\,(4.02\,m)}.$$

Using the value of $r = 30{,}000$ ft (9,144 m) as the starting point, straight lines having slopes of 7.9, 10.6, and 13.2 ft (2.4 m, 3.2 m, and 4.0 m) are drawn in Fig. 3.8.

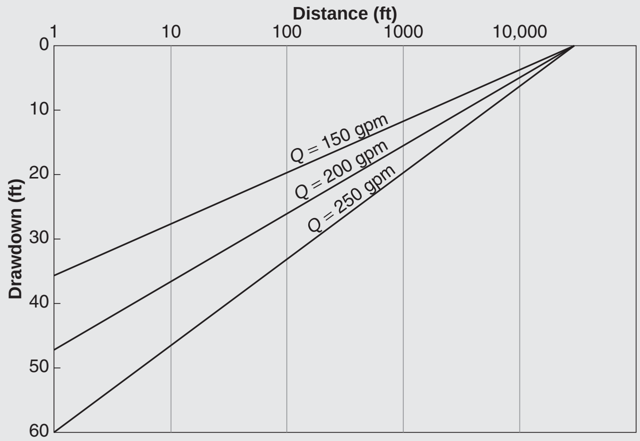


Figure 3.8 Distance–drawdown curves for various rates of pumping (Example 3.4). Conversion factors: 1 ft = 0.3048 m; 1 gpm = 5.45 m^3/d.

The contours of the piezometric surface can be drawn by subtracting the drawdowns at several points from the initial values.

3.11.2 Variable Discharge

The rate at which water is pumped from a well field in a water supply system will vary with time in response to changes in demand. The continuous rate of pumping curve can be approximated by a series of steps as shown in Fig. 3.9. Then each step can be analyzed, using one of the conventional equations. From the principle of superposition, the drawdown at any point at any specific time can be obtained as the sum of increments in drawdowns caused by the step increases up to that time:

$$s = \Delta s_1 + \Delta s_2 + \cdots + \Delta s_t \qquad \mathbf{(3.33)}$$

Using a semilogarithmic approximation,

$$s = (264\Delta Q_i/T)\log[(0.3\,Tt_i)/(r^2 S)] \qquad \mathbf{(3.34)}$$

where

$$t_i = t - t_{i-1}$$

Increments of drawdown are determined with respect to the extension of the preceding water level curve.

3.11.3 Intermittent Discharge

In a water supply system, a well (or a well field) may be operated on a regular daily cycle, pumping at a constant rate for a given time interval and remaining idle for the rest of the period. Brown (1963) gives the following expression for computing the drawdown in the pumped well after n cycles of operation:

$$s_n = (264\,Q/T)\log[(1 \cdot 2 \cdot 3 \cdots n)/(1-p)(2-p)\cdots(n-p)]$$

$$\text{(US customary units)} \qquad \mathbf{(3.35a)}$$

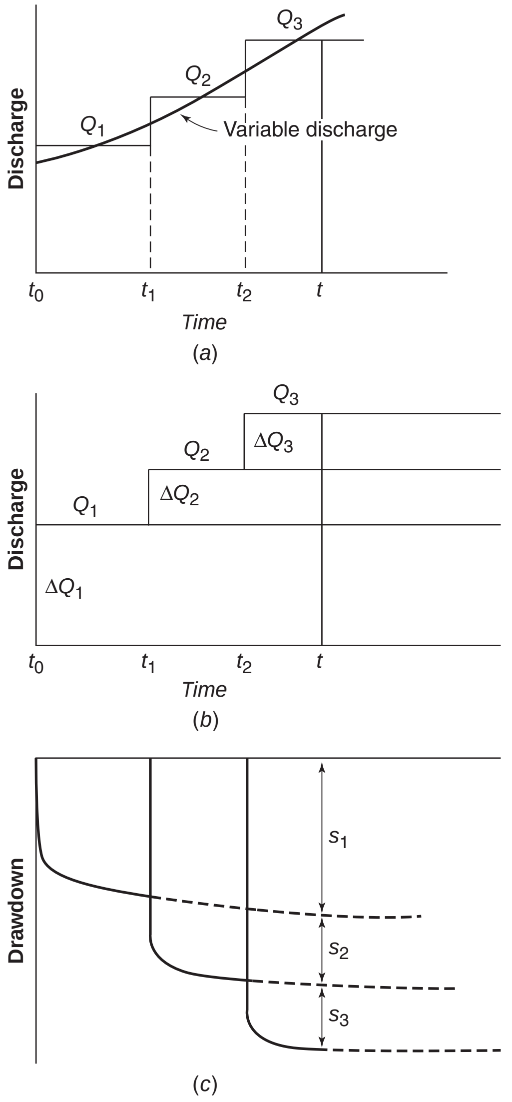
where s_n is the drawdown in the pumped well after n cycles in ft, p is the fractional part of the cycle during which the well is pumped, Q is the discharge in gpm, and T is the transmissivity in gpd/ft.

$$s_n = (380,160\,Q/T)\log[(1 \cdot 2 \cdot 3 \cdots n)/(1-p)(2-p)\cdots(n-p)] \quad \text{(SI units)} \qquad \mathbf{(3.35b)}$$

where s_n is the drawdown in the pumped well after n cycles in m, p is the fractional part of the cycle during which the

Table 3.4 Values of the function $W(u, r/B) = W(u, r/B) = \int_u^\infty \frac{1}{y} \exp \frac{-y-r^2}{4B^2 y} \mathrm{d}y$

$u^{r/B}$	0.005	0.01	0.025	0.05	0.075	0.10	0.15	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.5	2.0
0	10.8286	9.4425	7.6111	6.2285	5.4228	4.8541	4.0601	3.5054	2.7449	2.2291	1.8488	1.5550	1.3210	1.1307	0.9735	0.8420	0.4276	0.2278
0.000001	10.8283																	
0.000005	10.6822	9.4413																
0.00001	10.3963	9.4176																
0.00005	9.2052	8.8827	7.6000															
0.0001	8.5717	8.3983	7.5199	6.2282	5.4228													
0.0005	7.0118	6.9750	6.7357	6.0821	5.4062	4.8530												
0.001	6.3253	6.3069	6.1823	5.7965	5.3078	4.8292	4.0595	3.5054										
0.005	4.7249	4.7212	4.6960	4.6084	4.4713	4.2960	3.8821	3.4567	2.7428	2.2290								
0.01	4.0373	4.0356	4.0231	3.9795	3.9091	3.8150	3.5725	3.2875	2.7104	2.2253	1.8486	1.5550	1.3210	1.1307				
0.05	2.4678	2.4675	2.4653	2.4576	2.4448	2.4271	2.3776	2.3110	2.1371	1.9283	1.7075	1.4927	1.2955	1.1210	0.9700	0.8409		
0.1	1.8229	1.8227	1.8218	1.8184	1.8128	1.8050	1.7829	1.7527	1.6704	1.5644	1.4422	1.3115	1.1791	1.0505	0.9297	0.8190	0.4271	0.2278
0.5	0.5598	0.5598	0.5597	0.5594	0.5588	0.5581	0.5561	0.5532	0.5453	0.5344	0.5206	0.5044	0.4860	0.4658	0.4440	0.4210	0.3007	0.1944
1.0	0.2194	0.2194	0.2194	0.2193	0.2191	0.2190	0.2186	0.2179	0.2161	0.2135	0.2103	0.2065	0.2020	0.1970	0.1914	0.1855	0.1509	0.1139
5.0	0.0011	0.0011	0.0011	0.0011	0.0011	0.0011	0.0011	0.0011	0.0011	0.0011	0.0011	0.0011	0.0011	0.0011	0.0011	0.0011	0.0010	0.0010



(*a*)

(*b*)

(*c*)

Figure 3.9 Step function approximation of variable discharge.

well is pumped, Q is the discharge in m^3/d, and T is the transmissivity in $m^3/d/m$.

The pumping regime may involve switching on a well only during periods of peak demand. The problem of computing drawdown in a well then consists of applying one of the equations of nonsteady flow to each of the periods of pumping and recovery. The drawdown in the well, or at any other point, may be obtained by an algebraic sum of the individual values of drawdown and "buildup" resulting from each period of pumping and recovery resulting from each shutdown.

3.12 MULTIPLE-WELL SYSTEMS

Because the equations governing steady and unsteady flow are linear, the drawdown at any point due to several wells is equal to the algebraic sum of the drawdowns caused by each individual well, that is, for n wells in a well field

$$s = \sum_{i=1}^{n} s_i$$

where s is the drawdown at the point due to the ith well. If the location of wells, their discharges, and their formation constants are known, the combined distribution of drawdown can be determined by calculating drawdown at several points in the area of influence and drawing contours.

A problem of more practical interest is to determine the discharges of the wells when their drawdowns are given. This involves simultaneous solution of linear equations, which can be undertaken by numerical methods or by trial and error.

When the areas of influence of two or more pumped wells overlap, the draft of one well affects the drawdown of all others. In closely spaced wells, interference may become so severe that a well group behaves like a single well producing a single large cone of depression. When this is the case,

EXAMPLE 3.5 CALCULATION OF WELL DRAWDOWN

A well was pumped at a constant rate of 350 gpm (1907.5 m^3/d) between 7 a.m. and 9 a.m., 11 a.m. and 1 p.m., and 3 p.m. and 6 p.m., remaining idle the rest of the time. What will be the drawdown in the well at 7 a.m. the next day when a new cycle of pumping is to start? Assume no recharge or leakage. The transmissivity of the artesian aquifer is 3.2×10^4 gpd/ft (397.38 $m^3/d/m$).

Solution 1 (US Customary System):

The problem can be decomposed into three pumping and recovery periods, with Eq. (3.26a) applied to each of the subproblems:

$$s' = (264\,Q/T)\log(t/t').$$

$$264\,Q/T = 264 \times 350/3.2 \times 10^4 = 2.89.$$

For the first period of pumping:

$$t = \text{time since pumping started} = 1{,}440 \text{ min.}$$

$$t' = \text{time since pumping stopped} = 1{,}320 \text{ min.}$$

$$\log(t/t')_1 = \log(1{,}440/1{,}320) = 0.038.$$

Similarly, for the second period:

$$\log(t/t')_2 = \log(1{,}200/1{,}080) = 0.046.$$

For the third period:

$$\log(t/t')_3 = \log(960/780) = 0.090.$$

Total residual drawdown:

$$s' = (264Q/T)\left[\sum \log t/t'\right] = 2.89 \times 0.174 = \mathbf{0.5\,ft\,(0.15\,m)}.$$

Solution 2 (SI System):

The problem can be decomposed into three pumping and recovery periods, with Eq. (3.26b) applied to each of the subproblems:

$$\begin{aligned} s' &= (0.1833Q/T)\log(t/t') \\ &= (0.1833 \times 1907.5/397.38)\log(t/t') \\ &= 0.8798\log(t/t'). \end{aligned}$$

For the first period of pumping:

$$t = \text{time since pumping started} = 1{,}440\text{ min.}$$

$$t' = \text{time since pumping stopped} = 1{,}320\text{ min.}$$

$$\log(t/t')_1 = \log(1{,}440/1{,}320) = 0.038.$$

For the second period of pumping:

$$\log(t/t')_2 = \log(1{,}200/1{,}080) = 0.046.$$

For the third period of pumping:

$$\log(t/t')_3 = \log(960/780) = 0.090.$$

Total residual drawdown:

$$\begin{aligned} \mathbf{s}' &= (0.1833Q/T)\left[\sum \log t/t'\right] \\ &= 0.8798(0.038 + 0.046 + 0.090) \\ &\mathbf{= 0.15\,m}. \end{aligned}$$

EXAMPLE 3.6 DRAWDOWN IN THREE WELLS

Three 24 in. (610 mm) wells are located on a straight line 1,000 ft (304.8 m) apart in an artesian aquifer with $T = 3.2 \times 10^4$ gpd/ft (397.4 m^3/d/m) and $S = 3 \times 10^{-5}$. Compute the drawdown at each well when (a) one of the outside wells is pumped at a rate of 700 gpm (3,815 m^3/d) for 10 days and (b) the three wells are pumped at 700 gpm (3,815 m^3/d) for 10 days.

Solution 1 (US Customary System):

From Eq. (3.16),

$$\begin{aligned} u &= 1.87r^2S/Tt \\ u_{1\,\text{ft}} &= (1.87 \times 1^2 \times 3.0 \times 10^{-5})/(3.2 \times 10^4 \times 10) = 1.75 \times 10^{-10}, W(u) = 21.89 \\ u_{1{,}000\,\text{ft}} &= 10^6 u_1 = 1.75 \times 10^{-4}, W(u) = 8.08 \\ u_{2{,}000\,\text{ft}} &= 4 \times 10^6 u_1 = 7 \times 10^{-4}, W(u) = 6.69. \end{aligned}$$

Drawdown constant = $(114.6\,Q)/T = (114.6 \times 700)/(3.2 \times 10^4) = 2.51$ ft (0.765 m).

From Eq. (3.15), $s = (114.6Q/T)W(u)$.

a. Drawdown at face of pumping well:

$$s_1 = 2.51 \times 21.89 = \mathbf{54.9\ ft}\ (16.73\ \text{m}).$$

Drawdown in central well:

$$s_2 = 2.51 \times 8.08 = \mathbf{20.3\ ft}\ (6.19\ \text{m}).$$

Drawdown in the other outside well:

$$s_3 = 2.51 \times 6.69 = \mathbf{16.8\ ft}\ (5.12\ \text{m}).$$

b. Drawdown in outside wells:

$$s_1 + s_2 + s_3 = 54.9 + 20.3 + 16.8 = \mathbf{92\ ft}\ (28.04\ \text{m}).$$

Drawdown in central well:

$$s_1 + s_2 + s_2 = 54.9 + 20.3 + 20.3 = \mathbf{95.5\ ft}\ (29.11\ \text{m}).$$

Solution 2 (SI System):

From Eq. (3.13),

$$\begin{aligned}
1\ \text{ft} &= 0.3048\ \text{m},\ 1{,}000\ \text{ft} = 304.8\ \text{m},\ 2{,}000\ \text{ft} = 609.6\ \text{m}.\\
u &= r^2S/(4Tt)\\
u_{0.3048\ \text{m}} &= (0.3048)^2(3 \times 10^{-5})/(4 \times 397.4 \times 10) = 1.75 \times 10^{-10}\\
W(u) &= 21.89\\
u_{304.8\ \text{m}} &= (304.8)^2(3 \times 10^{-5})/(4 \times 397.4 \times 10) = 1.75 \times 10^{-4}\\
W(u) &= 8.08\\
u_{609.6\ \text{m}} &= (609.6)^2(3 \times 10^{-5})/(4 \times 397.4 \times 10) = 7.01 \times 10^{-4}\\
W(u) &= 6.69.
\end{aligned}$$

From Eq. (3.12),

$$\begin{aligned}
S &= (Q/4\pi T)W(u)\\
&= (3{,}815/4 \times 3.14 \times 397.4)W(u)\\
&= 0.765W(u).
\end{aligned}$$

a. Drawdown at face of pumping well:

$$s_1 = 0.765 \times 21.89 = 16.73\ \text{m}.$$

Drawdown in central well:

$$s_2 = 0.765 \times 8.08 = 6.18\ \text{m}.$$

Drawdown in other outside well:

$$s_3 = 0.765 \times 6.69 = 5.12\ \text{m}.$$

b. Drawdown in outside wells:

$$s_1 + s_2 + s_3 = 16.73 + 6.18 + 5.12 = 28.03\ \text{m}.$$

Drawdown in central well:

$$s_1 + s_2 + s_2 = 16.73 + 6.18 + 6.18 = 29.09\ \text{m}.$$

EXAMPLE 3.7 DISCHARGE FROM THREE WELLS AT A GIVEN DRAWDOWN

Suppose we want to restrict the drawdown in each of the wells from Example 3.6 to 60 ft (18.3 m). What will be the corresponding discharges for the individual wells?

Solution 1 (US Customary System):

Eq. (3.15) is $s = (114.6Q/T)W(u)$

$$\text{Well 1: } [114.6/T][Q_1 W(u_1) + Q_2 W(u_{1,000}) + Q_3 W(u_{2,000})] = 60\,\text{ft.}$$

$$\text{Well 2: } [114.6/T][Q_1 W(u_{1,000}) + Q_2 W(u_1) + Q_3 W(u_{1,000})] = 60\,\text{ft.}$$

$$\text{Well 3: } [114.6/T][Q_1 W(u_{2,000}) + Q_2 W(u_{1,000}) + Q_3 W(u_1)] = 60\,\text{ft.}$$

$$114.6/(3.2 \times 10^4)[21.89Q_1 + 8.08Q_2 + 6.69Q_3] = 60\,\text{ft.}$$

$$114.6/(3.2 \times 10^4)[8.08Q_1 + 21.89Q_2 + 8.08Q_3] = 60\,\text{ft.}$$

$$114.6/(3.2 \times 10^4)[6.69Q_1 + 8.08Q_2 + 21.89Q_3] = 60\,\text{ft.}$$

$$21.89Q_1 + 8.08Q_2 + 6.69Q_3 = 16,800.$$

$$8.08Q_1 + 21.89Q_2 + 8.08Q_3 = 16,800.$$

$$6.69Q_1 + 8.08Q_2 + 21.89Q_3 = 16,800.$$

Solving the three equations for the three unknown discharges Q_1, Q_2, and Q_3:

$$Q_1 = Q_3 = \mathbf{468\,gpm\,(2{,}550\,m^3/d)}.$$

$$Q_2 = \mathbf{420\,gpm\,(2{,}289\,m^3/d)}.$$

Solution 2 (SI System):

Eq. (3.12) is $s = (Q/4\pi T)W(u)$.

$$\text{Well 1: } [1/4\pi T][Q_1 W(u_{0.3048\,\text{m}}) + Q_2 W(u_{304.8\,\text{m}}) + Q_3 W(u_{609.6\,\text{m}})] = 18.3\,\text{m.}$$

$$\text{Well 2: } [1/4\pi T][Q_1 W(u_{304.8\,\text{m}}) + Q_2 W(u_{0.3048\,\text{m}}) + Q_3 W(u_{304.8\,\text{m}})] = 18.3\,\text{m.}$$

$$\text{Well 3: } [1/4\pi T][Q_1 W(u_{609.6\,\text{m}}) + Q_2 W(u_{304.8\,\text{m}}) + Q_3 W(u_{0.3048\,\text{m}})] = 18.3\,\text{m.}$$

Then,

$$[1/(4 \times 3.14 \times 397.4)][21.89Q_1 + 8.08Q_2 + 6.69Q_3] = 18.3\,\text{m.}$$

$$[1/(4 \times 3.14 \times 397.4)][8.08Q_1 + 21.89Q_2 + 8.08Q_3] = 18.3\,\text{m.}$$

$$[1/(4 \times 3.14 \times 397.4)][6.69Q_1 + 8.08Q_2 + 21.89Q_3] = 18.3\,\text{m.}$$

Then,

$$21.89Q_1 + 8.08Q_2 + 6.69Q_3 = 91,341.6.$$

$$8.08Q_1 + 21.89Q_2 + 8.08Q_3 = 91,341.6.$$

$$6.69Q_1 + 8.08Q_2 + 21.89Q_3 = 91,341.6.$$

Solving the three equations for the three unknowns, Q_1, Q_2, and Q_3:

$$Q_1 = Q_3 = 2{,}550\,\text{m}^3/\text{d}.$$

$$Q_2 = 2{,}289\,\text{m}^3/\text{d}.$$

discharge–drawdown relationships can be studied by replacing the group of wells by an equivalent single well having the same drawdown distribution when producing water at a rate equal to the combined discharge of the group.

The effective radius of a heavily pumped well field could be a mile or more and have a circle of influence extending over many miles. By contrast, lightly pumped, shallow wells in unconfined aquifers may show no interference when placed 100 ft (30.5 m) apart or even less. The number of wells, the geometry of the well field, and its location with respect to recharge and discharge areas and aquifer boundaries are important in determining the distribution of drawdown and well discharges. An analysis of the optimum location, spacing, and discharges should be carried out when designing a well field.

3.13 AQUIFER BOUNDARIES

Most methods of analysis assume that an aquifer is infinite in extent. In practice, all aquifers have boundaries. However, unless a well is located so close to a boundary that the radial flow pattern is significantly modified, the flow equations can be applied without appreciable error. Nevertheless, in many situations definite geologic and hydraulic boundaries limit aquifer dimensions and cause the response of an aquifer to deviate substantially from that predicted from equations based on extensive aquifers. This is especially true if the cone of depression reaches streams, outcrops, or groundwater divides; geologic boundaries, such as faults and folds; and valley fills of limited extent.

The effect of aquifer boundaries can be incorporated into analysis through the method of images. The *method of images* is an artifice employed to transform a bounded aquifer into one of an infinite extent having an equivalent hydraulic flow system. The effect of a known physical boundary (in the flow system) is simulated by introducing one or more hypothetical components, called images. The solution to a problem can then be obtained by using the equations of flow developed for extensive aquifers for this hypothetical system.

3.13.1 Recharge Boundaries

The conditions along a recharge boundary can be reproduced by assuming that the aquifer is infinite and by introducing a negative image well (e.g., a recharge image well for a discharging real well) an equal distance on the opposite side of the boundary from the real well, the line joining the two being at right angles to the boundary (Fig. 3.10). The drawdown,

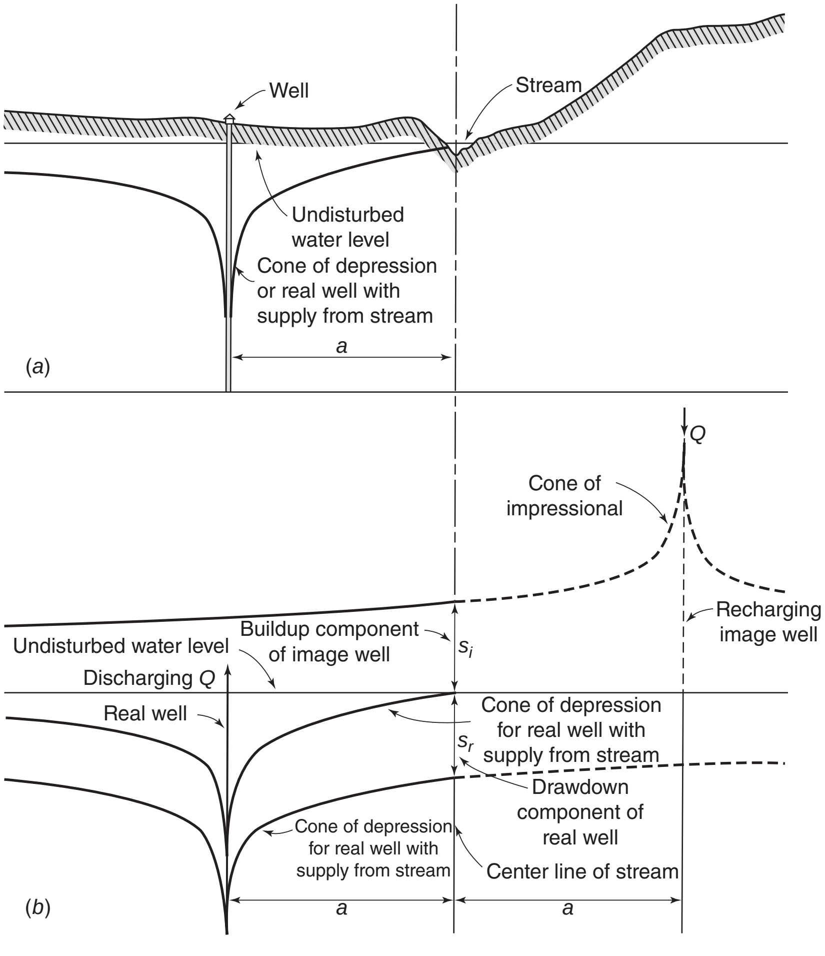


Figure 3.10 Application of the method of images to a well receiving water from a stream (idealized). (a) Real system. (b) Equivalent system in an infinite aquifer.

s, at any distance, r, from the pumping well and r_i from the image well is the algebraic sum of the drawdowns due to the real well, s_r, and buildup due to the image well, s_i:

$$s = s_r + s_i = (528Q/T)\log(r_i/r) \quad \text{(US customary units)} \tag{3.36a}$$

$$s_w = (528Q/T)\log(2a/r_w) \quad \text{(US customary units)} \tag{3.37a}$$

where s = the drawdown at any distance, r, from a pumping well, ft; s_r = the drawdown due to real well, ft; s_i = the drawdown due to image well, ft; s_w = the drawdown at the well, ft; r = any distance between the drawdown location and the pumping well, ft; r_i = the distance between the drawdown location and the image well, ft; r_w = the radius of the pumping well, ft; Q = pumping discharge rate, gpm; T = transmissivity, gpd/ft; and a = the distance between the stream and the pumping well, ft.

The following are two equivalent equations using the SI or metric units of s (m), s_r (m), s_i (m), s_w (m), r (m), r_i (m), r_w (m), Q (m^3/d), T (m^3/d/m), and a (m):

$$s = s_r + s_i = (0.366Q/T)\log(r_i/r) \quad \text{(SI units)} \tag{3.36b}$$

$$s_w = (0.366Q/T)\log(2a/r_w) \quad \text{(SI units)} \tag{3.37b}$$

EXAMPLE 3.8 DETERMINATION OF A WELL DRAWDOWN AND CONE OF DEPRESSION PROFILE

A gravel-packed well with an effective diameter of 24 in. (610 mm) pumps water from an artesian aquifer having $T = 3.2 \times 10^4$ gpd/ft (397.4 m^3/d/m) and $S = 3.4 \times 10^{-5}$. The well lies at a distance of 1,000 ft (304.8 m) from a stream that can supply water fast enough to maintain a constant head. Find the drawdown in the well after 10 days of pumping at 700 gpm (3,815 m^3/d). Determine the profile of the cone of depression with a vertical plane through the well normal to the stream.

Solution 1 (US Customary System):

In the region of interest, the use of a semilogarithmic approximation is valid:

$$s_w = (528\,Q/T)\log(2a/r_w)$$
$$s = (528Q/T)\log(r_i/r)$$
$$528\,Q/T = (528 \times 700)/(3.2 \times 10^4) = 11.55\ \text{ft.}$$

Drawdown at the well:

$$a = 1{,}000\ \text{ft}$$
$$r_w = 24\ \text{in.}/2 = 1\ \text{ft}$$
$$s_w = (528\,Q/T)\log(2a/r_w)$$
$$s_w = 11.55\ \log 2{,}000 = 38.0\ \text{ft.}$$

Drawdown at 500 ft from the stream:

$$r = 500\ \text{ft}$$
$$r_i = a + r = 1{,}000 + 500 = 1{,}500\ \text{ft}$$
$$s = 11.55\ \log(1{,}500/500) = \mathbf{5.5\ ft}.$$

Drawdown at 1,500 ft from the stream:

$$r = 1{,}500\ \text{ft}$$
$$r_i = a + r = 1{,}000 + 1{,}500 = 2{,}500\ \text{ft}$$
$$s = 11.55\ \log(2{,}500/1{,}500) = \mathbf{2.56\ ft}.$$

Drawdowns at other points can be calculated in a similar manner.

The results are shown in Fig. 3.10.

Solution 2 (SI System):

In the region of interest, the use of semilogarithmic approximation is valid:

$$s = (0.366\,Q/T)\log(r_i/r)$$
$$s_w = (0.366\,Q/T)\log(2a/r_w)$$
$$0.366\,Q/T = 0.366 \times 3{,}815/397.4 = 3.5\ \text{m.}$$

Drawdown at the well,

$$a = 304.8\,\text{m}$$
$$r_w = 610\,\text{mm}/2 = 305\,\text{mm} = 0.305\,\text{m}$$
$$s_w = 3.5\ \log(2 \times 304.8/0.305) = 11.55\,\text{m}.$$

Drawdown at 152.4 m (or 500 ft) from the stream:

$$r = 152.4\,\text{m}$$
$$r_i = a + r = 304.8\,\text{m} + 152.4\,\text{m} = 457.2\,\text{m}$$
$$s = 3.5\ \log(457.2/152.4) = 1.67\,\text{m}.$$

Drawdown at 457.2 m (or 1,500 ft) from the stream:

$$r = 457.2\,\text{m}$$
$$r_i = a + r = 304.8\,\text{m} + 457.2\,\text{m} = 762\,\text{m}$$
$$s = 3.5\ \log(762/457.2) = 0.78\,\text{m}.$$

The results are shown in Fig. 3.10.

For a well located near a stream, the proportion of the discharge of the well diverted directly from the source of recharge depends on the distance of the well from the recharge boundary, the aquifer characteristics, and the duration of pumping. Figure 3.11 shows an inflatable rubber dam that is inflated seasonally (late spring to late autumn) to raise the river stage by 3 m. The higher stage allows greater pumping rates to be maintained in the collector wells that supply water to Sonoma County residents in Maui, Hawaii. The contribution from a line source of recharge and distribution of drawdown in such a system can be evaluated and are extremely useful in determining the optimal location of well fields.

The problem of recirculation between a recharge well and a discharge well pair is of great practical importance because of the use of wells (or other devices) for underground waste disposal (or artificial recharge) and for water supply in the same area. The recirculation can be minimized by locating the recharge well directly downstream from the discharge well. The critical value of discharge and optimum spacing for no recirculation can be evaluated.

Figure 3.11 Inflatable rubber dam (Courtesy USGS).

The permissible distance, r_c, between production and disposal wells in an isotropic, extensive aquifer to prevent recirculation is given by

$$r_c = 2Q/(\pi/TI) \tag{3.38}$$

where r_c is in ft or m, Q is the equal pumping and disposal rate in gpd or m^3/d, T is the transmissivity in gpd/ft (m^3/d/m), and I is the hydraulic gradient of the water table or piezometric surface.

3.13.2 Location of Aquifer Boundaries

In many instances, the location and nature of hydraulic boundaries of an aquifer can be inferred from the analysis of aquifer test data. The effect of a boundary when it reaches an observation well causes the drawdowns to diverge from the curve or the straight-line methods. The nature of the boundary, recharge, or barrier is given by the direction of departures. An observation well closer to the boundary shows evidence of boundary effect earlier than does an observation well at a greater distance. The theory of images can be used to estimate the distance to the boundary. The analysis can be extended to locate multiple boundaries.

For the estimation of the formation constants, only those observations should be used that do not reflect boundary effects, that is, the earlier part of the time–drawdown curve. For the prediction of future drawdowns, the latter part of the curve incorporating the boundary effects is pertinent.

3.14 CHARACTERISTICS OF WELLS

The drawdown in a well being pumped is the difference between the static water level and the pumping water level. The well drawdown consists of two components:

1. *Formation loss*, that is, the head expended in overcoming the frictional resistance of the medium from the outer boundary to the face of the well, which is directly proportional to the velocity if the flow is laminar.
2. *Well loss*, which includes (a) the entrance head loss caused by the flow through the screen and (b) the head loss due to the upward axial flow of water inside the screen and the casing up to the pump intake. This loss is associated with the turbulent flow and is approximately proportional to the square of the velocity.

The well drawdown D_{w} can be expressed as

$$D_{\mathrm{w}} = BQ + CQ^2 \quad \textbf{(3.39a)}$$

where B summarizes the resistance characteristics of the formation and C represents the characteristics of the well.

For unsteady flow in a confined aquifer, from Eq. (3.18),

$$B = (264/T)\log(0.3\,Tt/r_{\mathrm{w}}^2\,S) \quad \textbf{(3.39b)}$$

This shows that the resistance of an extensive artesian aquifer increases with time as the area of influence of the well expands. For relatively low pumping rates, the well loss may be neglected, but for higher rates of discharge it can represent a sizable proportion of the well drawdown.

3.14.1 Specific Capacity of a Well

The productivity and efficiency of a well is generally expressed in terms of *specific capacity*, defined as the discharge per unit drawdown, that is, the ratio of discharge to well drawdown:

$$Q/D_{\mathrm{w}} = 1/(B + CQ) \quad \textbf{(3.40a)}$$

The specific capacity of a well depends on the formation constants and hydrogeologic boundaries of the aquifer, on well construction and design, and on test conditions. It is sometimes useful to distinguish between *theoretical specific capacity*, which depends only on formation characteristics and ignores well losses, and actual specific capacity. The former is a measure of the productivity. The difference between the two, or their ratio, is a measure of the efficiency of the well.

For unsteady flow in a confined aquifer,

$$Q/D_{\mathrm{w}} = 1/\{(264/T)\log[(0.3\,Tt)/(r_{\mathrm{w}}^2 S)] + CQ\} \quad \textbf{(3.40b)}$$

Hence the specific capacity is not a fixed quantity, but decreases with both the period of pumping and the discharge. It is important to state not only the discharge at which a value of specific capacity is obtained, but also the duration of pumping. Determination of specific capacity from a short-term acceptance test of a few hours' duration can give misleading results, particularly in aquifers having low hydraulic diffusivity, that is, low transmissivity and high storage coefficients.

3.14.2 Partial Penetration

The specific capacity of a well is affected by partial penetration. A well that is screened only opposite a part of an aquifer will have a lower discharge for the same drawdown or larger drawdown for the same discharge, that is, a smaller specific capacity. The ratio of the specific capacity of a partially penetrating well to the specific capacity of a completely penetrating well in homogeneous artesian aquifers is given by the *Kozeny formula*, which is valid for *steady-state conditions* using either the US customary units or the SI units:

$$(Q/s_{\mathrm{p}})(Q/s) = K_{\mathrm{p}}\{1 + 7[r_{\mathrm{w}}/(2K_{\mathrm{p}}b)]^{1/2}\cos(\pi K_{\mathrm{p}}/2)\} \quad \textbf{(3.41)}$$

where Q/s_{p} = specific capacity of a partially penetrating well, gpm/ft or m^3/d/m; Q/s = specific capacity of a completely penetrating well, gpm/ft or m^3/d/m; r_{w} = effective well radius, ft or m; b = aquifer thickness, ft or m; and K_{p} = ratio of length of screen to saturated thickness of the aquifer.

If the right-hand side of Eq. (3.41) is denoted by F_{p}, the equation may be written as

$$Q/s_{\mathrm{p}} = (Q/s)F_{\mathrm{p}} \quad \textbf{(3.42)}$$

The formula is not valid for small b, large K_{p}, and large r_{w}.

A graph of F_{p} versus K_{p} for various values of b/r_{w} is given in Fig. 3.12 within the valid range of the formula.

3.14.3 Effective Well Radius

The effective radius of a well is seldom equal to its nominal radius. *Effective radius* is defined as that distance, measured radially from the axis of a well, at which the theoretical drawdown equals the actual drawdown just at the surface of the well. Depending on the method of construction and development and the actual condition of the intake portion of the well, the effective radius may be greater than, equal to, or less than the nominal radius. The transmissivity of the material in the immediate vicinity of a well is the controlling factor. If the transmissivity of the material surrounding the well is higher than that of the aquifer, the effective well radius will be greater than the nominal radius. On the other hand, if the material around the well has a lower transmissivity due to caving or clogging because of faulty construction, the effective radius will be less than the nominal radius.

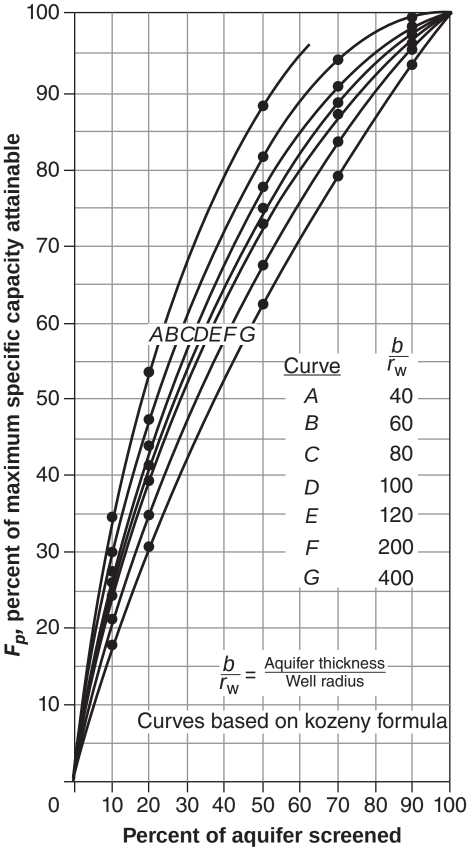


Figure 3.12 Relationship of partial penetration and specific capacity for wells in homogeneous artesian aquifers.

3.14.4 Measurement of Well Characteristics

The well-loss factor and the effective radius of a well can be determined by the multiple-step drawdown test. In this test, a well is pumped at a constant rate (generally for a few hours), after which the rate is increased and held constant at that rate for the same period. Three or four steps are used. The size of the steps depends on the yield of the well and the capacity of the pumping equipment. Another method for carrying out the test is to pump the well at a constant rate for a specified period, shut off the pump for an equal period, and then restart the pump at a higher rate. This is continued for three or four steps.

The analysis of the test data is similar to that discussed under variable discharge. Equation (3.34) is modified to include well losses. Increments of drawdown are determined at the same period of pumping in each step. Simultaneous solution of the equations gives the well-loss coefficient C, effective well radius r, and the values of formation constants. Usually a graphical procedure of solution is employed. Good results are obtained only if the value of C does not change appreciably with the rate of pumping. This indicates a stable well. A decrease in C for higher discharges may indicate development of a well during testing; an increase in C may denote clogging. The acceptance test for a well should include a step-drawdown test, because such a test permits the characteristics of the aquifer and of the well that govern the efficient performance of a water supply system to be evaluated.

3.15 YIELD OF A WELL

For the optimal design of a well (or a well field), the combination of discharge and drawdown that gives the lowest cost solution is sought. Both the capital outlays and the operation and maintenance costs need to be considered over the economic life of the structure. The interrelationship of pumping rate, drawdown, and specific yield serves as a basis for the selection of optimal design capacity. We have shown previously that the specific capacity decreases as the pumping rate is increased. Hence the earlier increments of drawdown are more effective in producing yields than the later ones, that is, each additional unit of yield is more expensive than the previous one. Increasing the yield of a well by one unit is economically justified only if the cost of developing this unit from alternate sources, such as another well or surface supplies, is higher.

The yield obtainable from a well at any site depends on (a) the hydraulic characteristics of the aquifer, which may be given in terms of a specific capacity–drawdown relationship; (b) the drawdown at the pumping well; (c) the length of the intake section of the well; (d) the effective diameter of the well; and (e) the number of aquifers penetrated by the well.

3.15.1 Maximum Available Drawdown

The maximum available drawdown at a well site can be estimated by the difference in elevations between the static water level and a conservation level below which it is undesirable to let the water levels drop. The conservation level is controlled by hydrogeologic conditions (type and thickness of the aquifer and the location of the most permeable strata), maintenance of the efficiency of the well, preservation of water quality, and pumping costs. In an artesian aquifer, good design practice requires that the drawdown not result in the dewatering of any part of the aquifer. Hence the maximum allowable drawdown is the distance between the initial piezometric level and the top of the aquifer.

In a water table aquifer, the pumping level should be kept above the top of the screen. The yield–drawdown relationship of homogeneous water table aquifers indicates that optimum yields are obtained by screening the lower one-half to one-third of the aquifer. A common practice is to limit the maximum available drawdown to one-half to two-thirds of the saturated thickness. In very thick aquifers, such as artesian or water table aquifers, the limiting factor in obtaining yields is not the drawdown but the cost of pumping. In some locations, the available drawdown may be controlled by the presence of poor quality water. The maximum drawdown should be such as to avoid drawing this poor quality water into the pumping well.

3.15.2 Specific Capacity–Drawdown Curve

A graph of specific capacity versus drawdown is prepared from the data on existing wells in the formation if such data are available. Specific capacities should be adjusted for well losses and partial penetration and should be reduced to a common well radius and duration of pumping. If no data are available, a step-drawdown test is conducted on the production well. The curve is extended to cover the maximum available drawdown. For a well receiving water from more than one aquifer, the resultant specific capacity is the sum of the specific capacities of the aquifer penetrated, reduced appropriately for partial penetration.

3.15.3 Maximum Yield

The following procedure is carried out to estimate the maximum yield of a well:

1. Calculate the specific capacity of the fully penetrating well having the proposed diameter from Eq. (3.15) or Eq. (3.18).
2. Reduce the specific capacity obtained above for partial penetration. This can be done by using Eq. (3.41).
3. Adjust the specific capacity for the desired duration of pumping from Eq. (3.40).
4. Calculate the maximum available drawdown.
5. Compute the maximum yield of the well by multiplying the specific capacity in step 3 by the maximum available drawdown.

EXAMPLE 3.9 COMPUTATION OF THE SPECIFIC CAPACITY OF A WELL AND ITS MAXIMUM YIELD

A well having an effective diameter of 12 in. (305 mm) is to be located in a relatively homogeneous artesian aquifer with a transmissivity of 10,000 gpd/ft (124.18 m^3/d/m) and a storage coefficient of 4×10^{-4}. The initial piezometric surface level is 20 ft (6.1 m) below the land surface. The depth to the top of the aquifer is 150 ft (45.72 m) and the thickness of the aquifer is 50 ft (15.24 m). The well is to be finished with a screen length of 20 ft (6.1 m). Compute the specific capacity of the well and its maximum yield after 10 days of pumping. Neglect well losses.

Solution 1 (US Customary System):

The specific capacity of a l2 in., fully penetrating well after 40 days of pumping can be calculated from Eqs. (3.15) and (3.16):

$$s = 114.6\,QW(u)/T \tag{3.15}$$

$$u = 1.87(S/T)(r^2/t) \tag{3.16}$$

$$u = 1.87(4 \times 10^{-4}/10^4)(0.5^2/10) = 1.87 \times 10^{-9}.$$

From Table 3.2: $W(u) = 19.52$.

$$(Q/s) = T/114.6\,W(u) = 10,000/(114.6 \times 19.52)$$

$$= 4.47\ \text{gpm/ft}\,(80\ \text{m}^3\ /\text{d/m}).$$

The percentage of aquifer screened is $K_p = 20/50 = 40\%$.
The slenderness of the well factor is $b/r_w = 50/0.5 = 100$.
The value of F_p from Fig. 3.12 is 0.65.
The expected **specific capacity** of the well = $0.65 \times 4.47 = 2.9$ **gpm/ft** (**51.87** m^3/d/m).
The maximum available drawdown = 150 − 20 = 130 ft (39.62 m).
The maximum **yield** of the well = 130 × 2.9 = **377 gpm** (**2,055 m^3/d**).

Solution 2 (SI System):

The specific capacity of a 305 mm, fully penetrating well after 40 days of pumping can be calculated from Eqs. (3.12) and (3.13):

$$s = (Q/4\pi T)W(u).$$

$$u = (r^2 s)/(4\,Tt).$$

Given data are

$$t = 10\ \text{days}$$

$$T = 124.18\ \text{m}^3/\text{d/m},$$

$$S = 4 \times 10^{-4}$$

$$r = (305\ \text{mm}/2) = 0.1525\ \text{m}.$$

Then

$$u = (0.1525)^2 \times (4 \times 10^{-4})/(4 \times 124.18 \times 10)$$
$$= 1.87 \times 10^{-9}.$$

From Table 3.2, $W(u) = 19.52$

$$(Q/s) = 4\pi T/W(u) = 4 \times 3.14 \times 124.18/19.52 = 80\,\mathrm{m}^3\,/\mathrm{d}/\mathrm{m}.$$

The percentage of aquifer screened is $K_\mathrm{p} = 6.1/15.24 = 40\%$.
The slenderness of the well factor is $b/r_\mathrm{w} = 15.24/0.1525 = 100$.
The value of F_p from Fig. 3.12 is 0.65.
The expected **specific capacity** of the well $(Q/s_\mathrm{p}) = (Q/s)F_\mathrm{p} = 80 \times 0.65 = 51.9\,\mathrm{m}^3/\mathrm{d}/\mathrm{m}$.
The maximum available drawdown = $45.72 - 6.1 = 39.62\,\mathrm{m}$.
The maximum **yield** of the well = $(51.9\,\mathrm{m}^3/\mathrm{d}/\mathrm{m})(39.62\,\mathrm{m}) = 2{,}057\,\mathrm{m}^3/\mathrm{d}$.
The actual maximum **yield** will be between 1,635 and 2,457 m^3/d.

3.16 WELL DESIGN

From the standpoint of well design, it is useful to think of a well as consisting of two parts: (a) the *conduit portion* of the well, which houses the pumping equipment and provides the passage for the upward flow of water to the pumping intake; and (b) the *intake portion*, where the water from the aquifer enters the well. In consolidated water-bearing materials, the conduit portion is usually cased from the surface to the top of the aquifer, and the intake portion is an uncased, open hole. In unconsolidated aquifers, a perforated casing or a screen is required to hold back the water-bearing material and to allow water to flow into the well.

The depth of a well depends on the anticipated drawdown for the design yield, the vertical position of the more permeable strata, and the length of the intake portion of the well.

The well size affects the cost of construction substantially. The well need not be of the same size from top to bottom. The diameter of a well is governed by (a) the proposed yield of the well, (b) entrance velocity and loss, and (c) the method of construction. The controlling factor is usually the size of the pump that will be required to deliver the design yield. The diameter of the casing should be two nominal sizes larger than the size of the pump bowls, to prevent the pump shaft from binding and to reduce well losses. Table 3.5 gives the casing sizes recommended for various pumping rates.

The selection of the well size may depend on the size of the open area desired to keep entrance velocities and well losses at a reasonable value. In deep-drilled wells, the minimum size of the hole may be controlled by the equipment necessary to reach the required depth. Deep wells in consolidated formations are often telescoped in size to permit drilling to required depths.

The wells are generally lined or cased with mild steel pipe, which should be grouted in place in order to prevent caving and contamination by vertical circulation and to prevent undue deterioration of the well by corrosion. If conditions are such that corrosion is unusually severe, then plastic or glass fiber pipes can be used if practicable.

The intake portion of the well should be as long as economically feasible to reduce the drawdown and the entrance velocities. In relatively homogeneous aquifers, it is not efficient to obtain more than 90% of the maximum yield. In

Table 3.5 Recommended well diameters

Anticipated well yield (gpm)	Nominal size of pump bowls (in.)	Optimum size of well casing (in.)	Smallest size of well casing (in.)
Less than 100	4	6 ID	5 ID
75–175	5	8 ID	6 ID
150–400	6	10 ID	8 ID
350–650	8	12 ID	10 ID
600–900	10	14 OD	12 ID
850–1,300	12	16 OD	14 OD
1,200–1,800	14	20 OD	16 OD
1,600–3,000	16	24 OD	20 OD

Conversion factors: 1 gpm = $5.45\,\mathrm{m}^3/\mathrm{d}$;1in. = 25.4 mm.
OD = Outside diameter; ID = Inside diameter.

nonhomogeneous aquifers, the best strategy is to locate the intake portion in one or more of the most permeable strata.

Perforated pipes or prefabricated screens are used in wells in unconsolidated aquifers. The width of the screen openings, called the slot size, depends on the critical particle size of the water-bearing material to be retained and on the grain-size distribution and is chosen from a standard sieve analysis of the aquifer material. With a relatively coarse and graded material, slot sizes are selected that will permit the fine and medium-sized particles to wash into the well during development and to retain a specified portion of the aquifer material around the screen. A graded filter is thereby generated around the well, which has higher permeability than the undisturbed aquifer material.

Perforated casings are generally used in uncemented wells when relatively large openings are permissible. If the casing is slotted in place after installation, the smallest practical opening is one-eighth inch (3.18 mm). Machine-perforated casings are also available. Fabricated well screens are available in a wide variety of sizes, designs, and materials. The choice of material is governed by water quality and cost.

For maximum efficiency, the frictional loss of the screen must be small. The head loss through a screen depends on screen length L, diameter D, percentage open area A_p, coefficient of contraction of openings C_c, velocity in the screen v, and the total flow into the screen Q (ft^3/s). It has been shown that for minimum screen loss, $CL/D > 6$,where $C = 11.31C_cA_P$. The value of CL/D may be increased by increasing C_c, A_p, or L, or by decreasing D. Thus for the screen loss to be a minimum, the percentage of open area depends on the length and diameter of the screen. The screen length is usually fixed by the considerations of hydrogeology and cost.

The screen length and diameter can be selected from the slot size and the requirement that the entrance velocity be less than that needed to move the unwanted sand particle sizes into the well. Experience has shown that, in general, a velocity of 0.1 ft/s (0.03 m/s) gives negligible friction losses and the least incrustation and corrosion.

Where the natural aquifer material is fine and uniform (effective size less than 0.01 in. (0.25 mm) and uniformity coefficient less than 3.0), it is necessary to replace it by a coarser gravel envelope next to the screen. The slot size is selected to fit this gravel pack. The gravel pack increases the effective well radius and acts as a filter and a stabilizer for the finer aquifer material. A gravel-packed well is shown in Fig. 3.13. There are no universally accepted rules for the selection of slot sizes or for the design of a gravel pack. A correctly designed well should provide a virtually sand-free operation (less than 3 mg/L). The thickness of the gravel pack should not be less than 3 in. or more than 9 in. and the particle size distribution curve of the pack should approximately parallel that of the aquifer.

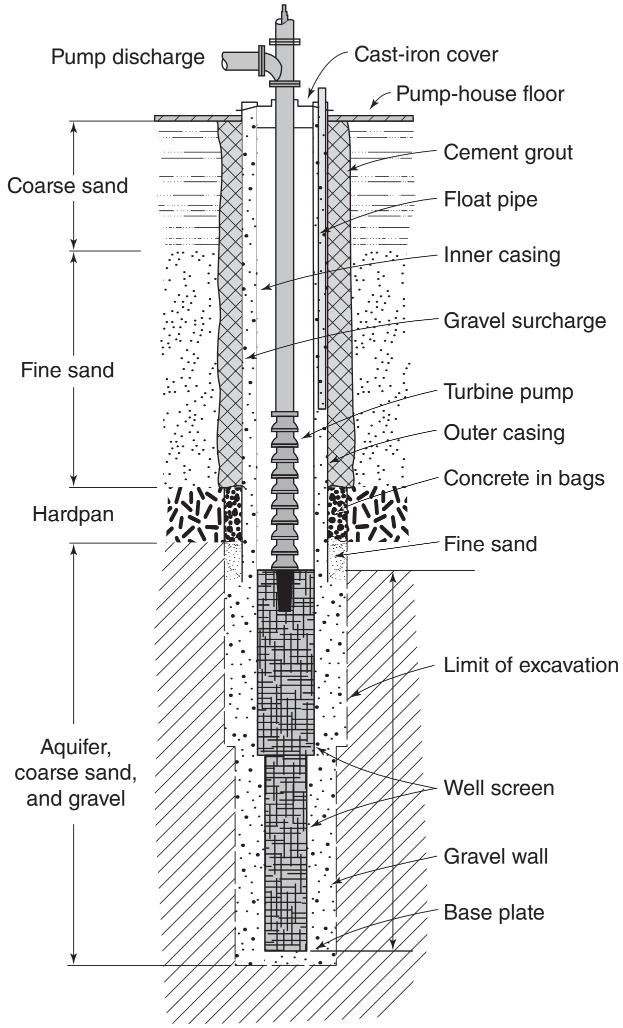


Figure 3.13 Gravel-packed well with deep-well turbine pump (After Wisconsin State Board of Health).

3.17 WELL CONSTRUCTION

There is no one optimum method of well construction. The size and depth of the hole, the rocks to be penetrated, and the equipment and experience of local drillers control the method of well sinking and determine the cost of construction. Well sinking is a specialized art that has evolved along a number of more or less regional lines. In the United States, well drillers are generally given much latitude in the choice of a suitable method. What they undertake to do is to sink a well of specified size at a fixed price per foot. Ordinarily, therefore, the engineer gives his attention not so much to drilling operations as to the adequacy, suitability, and economics of proposed developments and the location of the works.

Well categories generally take their names from the methods by which wells are constructed. Shallow wells can be dug, driven, jetted, or bored.

3.17.1 Dug Wells

Small dug wells are generally excavated by hand. In loose overburden, they are cribbed with timber; lined with brick, rubble, or concrete; or cased with large-diameter vitrified tile or concrete pipe. In rock, they are commonly left unlined. Excavation is continued until water flows in more rapidly than it can be bailed out. Dug wells should be completed when the water table is at or near its lowest level. Otherwise, they may have to be deepened at a later date.

Large and deep dug wells are often constructed by sinking their liners as excavation proceeds. The lead ring has a steel cutting edge; new rings are added as excavation progresses.

3.17.2 Driven and Jetted Wells

Wells can be driven into relatively shallow sand formations. As shown in Fig. 3.14, the driving point is attached to a strainer or perforated section of pipe. To reduce friction, the point is somewhat larger than the casing. The driving weight is commonly suspended from a block attached to a tripod. In hard ground a cylindrical shoe equipped with water jets loosens the soil and washes it to the surface. Batteries of driven wells may be connected to a suction header to supply enough water.

3.17.3 Bored Wells

Wells can be bored with hand or power augers into sufficiently cohesive (noncaving) soils. Above the water table, the soil is usually held in the auger, which must then be raised from time to time to be cleaned. Below the water table, sand may wash out of the auger and have to be removed from the bore hole by a bailer or sand pump. As the well becomes deeper and deeper, sections of rod are added to the auger stem. Bits up to 36 in. (914 mm) in diameter have been used successfully, and wells have been enlarged in diameter up to 48 in. (1219 mm) by reaming. A concrete, tile, or metal casing is inserted in the hole and cemented in place before the strainer is installed.

3.17.4 Drilled Wells

High-capacity, deep wells are constructed by drilling. Because the water-bearing materials vary so widely, no one method of drilling can be adopted under all conditions. The method of drilling is selected to suit the particular conditions of a site. The systems of drilling used in water-well construction are based on either the percussion or the rotary principle.

3.17.5 Collector Wells

A *collector well* consists of a central shaft of concrete caisson some 15 ft (4.57 m) in internal diameter and finished off below the water table with a thick concrete plug. From this shaft, perforated radial pipes 6 or 8 in. (150 or 200 mm) in diameter and 100–250 ft (30–76 m) long are jacked horizontally into a water-bearing formation through ports near the bottom of a caisson. The collector pipes may be installed and developed in the same manner as for ordinary wells.

3.17.6 Pumps

Many types of well pumps are on the market to suit the wide variety of capacity requirements, depths to water, and sources of power. Figure 3.15 shows a well pump and 190,000 gal (719,150 L) water tank. In the United States almost all well pumps are driven by electric motors.

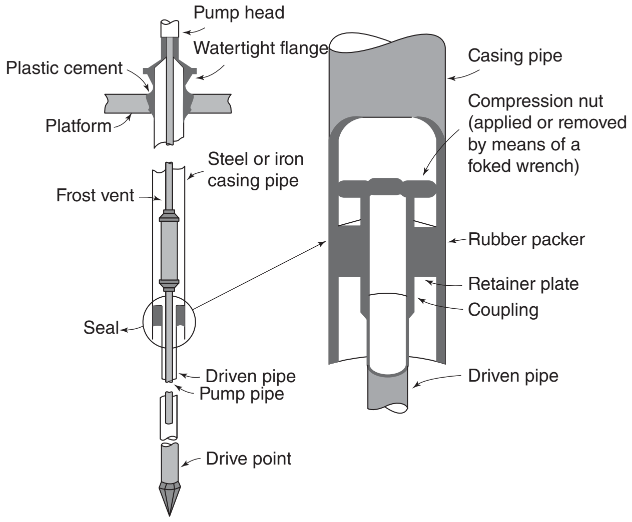


Figure 3.14 Driven well and its sanitary protection (After Iowa State Department of Health).

Figure 3.15 Well and 190,000 gal (719,150 L) Tank, Maui, HI (Courtesy of the Department of Water Supply, Maui County, HI).

Domestic systems commonly employ one of the following pumps:

1. For lifts under 25 ft (7.62 m), a small reciprocating or piston pump
2. For lifts up to 125 ft (38.10 m), a centrifugal pump to which water is lifted by recirculating part of the discharge to a jet or ejector
3. For lifts that cannot be managed by jet pumps, a cylinder pump installed in the well and driven by pump rods through a jack mounted at the well head

Systems of choice normally incorporate pressure tanks for smooth pressure-switch operation. The well itself may provide enough storage to care for differences between demand rates in the house and flow rates from the aquifer. This is why domestic wells are seldom made less than 100 ft (30 m) deep even though the water table may lie only a few feet below the ground surface. Deep wells and pump settings maintain the supply when groundwater levels sink during severe droughts or when nearby wells are drawn down steeply.

Large-capacity systems are normally equipped with centrifugal or turbine pumps driven by electric motors. A sufficient number of pump bowls are mounted one above the other to provide the pressure necessary to overcome static and dynamic heads at the lowest water levels. For moderate quantities and lifts, *submersible* motors and pumps, assembled into a single unit, are lowered into the well. The water being pumped cools the compact motors normally employed. Large-capacity wells should be equipped with suitable measuring devices. Continuous records of water levels and rates of withdrawal permit the operator to check the condition of the equipment and the behavior of the source of supply. This is essential information in the study and management of the groundwater resource.

3.17.7 Development

Steps taken to open up or enlarge flow passages in the formation in the vicinity of the well are called *development*. Thorough development of the completed well is essential regardless of the method of construction used to obtain higher specific capacities, to increase effective well radius, and to promote efficient operation over a longer period of time. This can be achieved in several ways. The method selected depends on the drilling method used and on the formation in which the well is located. The most common method employed is overpumping, that is, pumping the well at a higher capacity than the design yield. Temporary equipment can provide the required pumping rates. Pumping is continued until no sand enters the well. Other methods used include flushing, surging, high-velocity jetting, and backwashing. Various chemical treatments and explosives are used in special circumstances.

3.17.8 Testing

After a well is completed, it should be tested to determine its characteristics and productivity. Constant-rate and step-drawdown pumping tests are used for this purpose. The test should be of sufficient duration; the specific capacity of a well based on a 1-hour test may be substantially higher than that based on a 1-day test. Longer duration is also required to detect the effect of hydraulic boundaries, if any. The extent to which the specific yield would decrease depends on the nature and the effectiveness of the boundaries.

3.17.9 Sanitary Protection of Wells

The design and construction of a well to supply drinking water should incorporate features to safeguard against contamination from surface and subsurface sources. The protective measures vary with the geologic formations penetrated and the site conditions. The well should be located at such a distance from the possible sources of pollution (e.g., wells used for the disposal of liquid wastes or artificial recharge; seepage pits; and septic tanks) that there is no likelihood of contaminated water reaching the well. The casing should be sufficiently long and watertight to seal off formations that have undesirable characteristics. Failure to seal off the annular space between the casing and well hole has been responsible for bacterial contamination in many instances. The casing should be sealed in place by filling the open space around the casing with cement grout or other impermeable material down to an adequate depth. This prevents seepage of water vertically along the outside of the pipe. A properly cemented well is shown in Fig. 3.16. The well casing should extend above the ground. The top of the well should contain a watertight seal; the surface drainage should be away from it in all directions.

An essential final step in well completion is the thorough disinfection of the well, the pump, and the piping system. Although the water in the aquifer itself may be of good sanitary quality, contamination can be introduced into the well system during drilling operations and the installation of other elements of the system. Periodic disinfection of the well during the drilling is a good practice and should be encouraged. In the case of an artificially gravel-packed well, all gravel-pack material should be sterilized before being placed in the well. Solution strengths of 50–200 mg/L chlorine are commonly used for sterilizing wells. The effectiveness of disinfection should be checked after the completion of the work. Disinfection of the system is also necessary after repairs of any part of the system.

3.17.10 Maintenance

Good maintenance extends the life of a well. The maintenance of the yield of a well depends on (a) the well construction, (b) the quality of water pumped (water may be

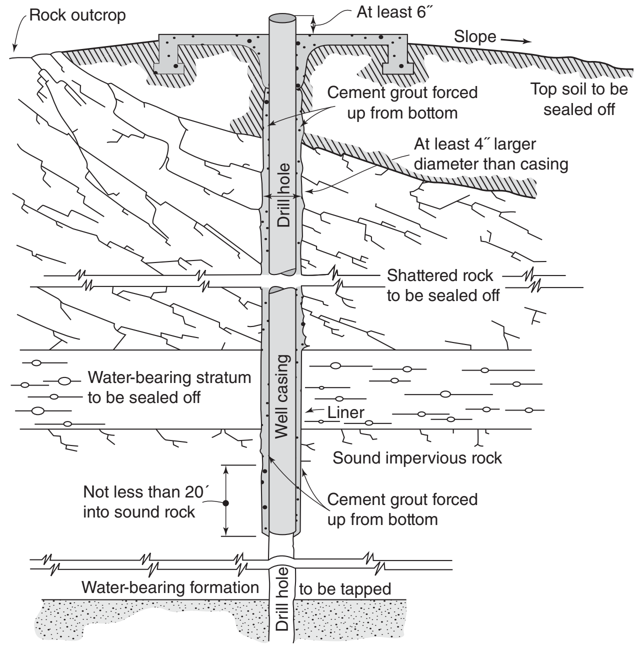


Figure 3.16 Drilled well and its sanitary protection (After Iowa State Department of Health). Conversion factor: $1' = 1\text{ ft} = 0.3048\text{ m}$; $1'' = 1\text{ in.} = 25.4\text{ mm}$.

corroding or encrusting), and (c) any interference from neighboring wells. If the performance of the well declines, renovation measures should be undertaken that may include mechanical cleaning, surging, and chemical treatment.

3.18 EVALUATION OF AQUIFER BEHAVIOR

Planning for the optimum utilization of the groundwater resource in an aquifer system requires the evaluation of the merits of alternative strategies of development. The steps involved in predicting the consequences of various plans are (a) quantitative assessment of the hydraulic and hydrologic characteristics of the aquifer system and (b) elaboration of the cause-and-effect relationships between pumping, replenishment, and water levels.

3.18.1 Hydrologic Equation

The basic groundwater balance equation is an expression of material balance:

$$\text{Inflow (or recharge)} = \text{outflow (or discharge)} \pm \text{change in storage}$$

$$I = O \pm \Delta S \tag{3.43}$$

This equation must be applied to a specific area for a specific period of time. When drawdowns imposed by withdrawals reduce the hydraulic gradient in the discharge areas, the rate of natural discharge is reduced. These flows become available for development and a new equilibrium condition is approached with the water table or the piezometric surface at a lower level.

The effect of drawdown on aquifer recharge is difficult to evaluate. Additional recharge may be induced into an aquifer through the former discharge areas by reversing the hydraulic gradient. In leaky aquifers, inflow may be induced by the reduction of heads, the contributions being roughly proportional to drawdown. Additional recharge may also be induced in the recharge areas if drawdown causes a dewatering in areas where recharge was limited because the aquifer was full. This is referred to as the capture of *rejected recharge*.

3.18.2 Safe Yield of an Aquifer

The yield of an aquifer depends on the following:

1. The characteristics of the aquifer
2. The dimensions of the aquifer and the hydraulic characteristics of its boundaries
3. The vertical position of each aquifer and the hydraulic characteristics of the overlying and underlying beds
4. The effect of proposed withdrawals on the recharge and discharge characteristics of the aquifer

Thus it is evident that the safe yield of an aquifer is not necessarily a fixed quantity and it is not strictly a characteristic of the groundwater aquifer. It is a variable quantity dependent on natural hydrogeologic conditions and on recharge and discharge regimes. Safe yield has been defined in a variety of ways, each definition placing emphasis on a particular aspect of groundwater resource development. These include, within economic limits, (a) development to the extent that withdrawals balance recharge and (b) development to the extent that change in the quality of groundwater allows.

3.18.3 Water Budget (Hydrologic Budget)

The continuity of the water flow through a system can be measured by a water budget, which is an indication of water resources sustainability. Equation (3.43) can be rearranged to be a mass balance equation in which the change of water storage (ΔS) over a time period within the water system equals the inflows or recharges (I) to the system minus the outflows or discharges (O) from the system. The water budget equation for a watershed can be derived as follows:

$$\Delta S = (G_{\text{in}} + R) - (E_{\text{t}} + E + G_{\text{out}} + Q_{\text{out}}) = I - O \tag{3.44}$$

Similarly the water budget equations for a lake can also be derived using Eq. (3.43):

$$\Delta S = (G_{\text{in}} + R + Q_{\text{in}}) - (Q_{\text{out}} + G_{\text{out}} + E + E_{\text{t}}) \tag{3.45}$$

$$G = G_{\text{in}} - G_{\text{out}} = \Delta S - R + E + E_{\text{t}} - Q_{\text{in}} + Q_{\text{out}} \tag{3.46}$$

EXAMPLE 3.10 DETERMINATION OF NET GROUNDWATER FLOW TO A LAKE

Determine the net groundwater flow to a lake G, which equals to the groundwater inflow to the lake G_{in}, minus the groundwater outflow G_{out}. Use a water budget for year 2012 in which the rainfall (precipitation) R was 44 in. (1,117.6 mm), the evaporation E was 55 in. (1,397.0 mm), the surface-water inflow Q_{in} was 1.2 in. (30.5 mm), the surface-water outflow Q_{out} was 170 in. (4,318.0 mm), the change in water storage ΔS was –3 in. (–76.2 mm), and the evapotranspiration E_{t} was in negligible amount.

Solution 1 (US Customary System):

$$\begin{aligned} G = G_{\text{in}} - G_{\text{out}} &= \Delta S - R + E + E_{\text{t}} - Q_{\text{in}} + Q_{\text{out}} \\ &= -3 - 44 + 55 + 0 - 1.2 + 170 \\ &= \mathbf{176.8}\text{ in. for the year.} \end{aligned}$$

Solution 2 (SI System):

$$\begin{aligned} G = G_{\text{in}} - G_{\text{out}} &= \Delta S - R + E + E_{\text{t}} - Q_{\text{in}} + Q_{\text{out}} \\ &= -76.2 - 1{,}117.6 + 1{,}397.0 + 0 - 30.5 + 4{,}318.0 \\ &= \mathbf{4{,}490.7}\text{ mm for the year.} \end{aligned}$$

where ΔS = change in water storage; G_{in} = groundwater inflow; G_{out} = groundwater outflow; E = evaporation; R = rainfall or precipitation; E_{t} = evapotranspiration; Q_{out} = stream outflow; Q_{in} = stream inflow; and G = groundwater flow to a lake.

3.19 GROUNDWATER QUALITY MANAGEMENT

In a majority of cases when polluted water has been drawn from wells, the contamination was introduced at the well site, indicating faulty construction. There are, however, numerous examples of contamination of groundwater caused by disposal of wastes. Once groundwater is contaminated, the impairment of the groundwater resource is long lasting and recovery is extremely slow.

To predict where the contaminating fluids will go requires a three-dimensional geologic, hydrodynamic, and geochemical analysis. The rate and extent of the spread of pollution are controlled by (a) the characteristics of the source of pollution, (b) the nature of rock formations in the unsaturated and saturated zones, and (c) the physical and chemical properties of the contaminant. The phenomena governing the disposition of the contaminant are capillary attraction, decay, adsorption, dispersion, and diffusion.

There have been numerous examples of contamination of groundwater by wastes allowed to seep into the ground, wastes discharged into pits and ponds, and leaks from holding tanks and sewers. The safe distance from a polluting source of this type is determined to a large degree by the velocity of percolation through the unsaturated zone and by the lateral movement once the contamination reaches groundwater. Water table aquifers, being near the surface and having a direct hydrologic connection to it, are more subject to contamination than are deeper lying artesian aquifers.

The discharge of wastes into streams has had both direct and indirect effects on the quality of groundwater. The polluted rivers that cross recharge areas of artesian aquifers tapped by wells have affected the quality of their discharge. The aquifers that are replenished by infiltration from polluted streams will eventually be contaminated by soluble chemical wastes carried in the stream. Induced contamination of an aquifer can result when the cone of depression of a discharging well intersects a polluted river. This is frequently the case in coastal areas in wells located near streams containing brackish water. Artificial recharge with river water of poorer quality than that found in the aquifer will ultimately result in the deterioration of the quality of groundwater.

3.19.1 Biological Contamination

Because of increasing numbers of septic tanks and growing use of effluents from wastewater treatment plants for artificial recharge of aquifers, the potential for contamination of groundwater by bacteria and viruses needs to be considered. Filtration through granular material improves the biological quality of water. A 10 ft (3 m) downward percolation in fine sand is capable of removing all bacteria from water. The length of time bacteria and viruses may survive and the distances they may travel through specific rock materials in different subsurface environmental conditions are uncertain. They seem to behave in a manner similar to the degradable and adsorbable contaminants. Under favorable conditions some bacteria and viruses may survive up to at least 5 years in the underground environment. However, the distances traveled in both the saturated and unsaturated media are surprisingly short when reasonable precautions are taken in disposal.

The principal determinant of the distance traveled seems to be the size of the media. Romero (1970) provides diagrams that may be used to evaluate the feasibility of disposing of biologically contaminated wastes in saturated and unsaturated granular media. The danger of bacterial pollution is greater in fractured rocks, cavernous limestones, and gravel deposits where the granular materials have no filtering capacity. The distances traveled will be higher in areas of influence of discharging or recharging wells because higher velocities are present. The higher rates of artificial recharge and greater permeability of artificial recharge basins enable bacteria to be carried to a greater depth.

3.19.2 Subsurface Disposal of Liquid Wastes

Subsurface space may be used to an increasing degree for the disposal of wastes. The oil industry pumps nearly 20 million barrels of salt water per day into subsurface formations from which oil has been extracted. Some highly toxic chemical wastes are disposed of underground. The use of an aquifer as a receptacle for toxic waste materials is justified only if it has little or no value as a present, or potential, source of water supply. Further, there should not be any significant risk of contaminating other aquifers or of inducing fractures in the confining formations. Recharging of groundwater by injection or spreading of reclaimed municipal wastewaters is an accepted practice that will undoubtedly be increasingly used in the future.

3.20 GROUNDWATER UNDER THE DIRECT INFLUENCE OF SURFACE WATER

The US Federal Surface Water Treatment Rule (SWTR) gives the following definition for *groundwater directly under the influence of surface water (GWUDI):* "any water beneath the surface of the ground which exhibits significant and rapid shifts in water characteristics such as turbidity, temperature/conductivity or pH which closely correlates to climatological or surface water conditions and/or which contains macroorganisms, algae, large diameter (three microns or greater) pathogens or insect parts of a surface water origin." In the United States, true groundwater, which is not directly influenced by surface water, will be monitored and/or treated under the Groundwater Rule, whereas GWUDI will be monitored and treated in accordance with the SWTR and the Long-Term 2 Enhanced Surface Water Treatment Rule (LT2-ESWTR).

The purpose of regulating groundwater sources under the direct influence of surface water in the SWTR is to protect against contamination from large-diameter pathogens associated with surface waters. Groundwater sources determined to be under the direct influence of surface water must be filtered or meet filtration avoidance criteria as contained in US federal and state sanitary codes. In some cases, it will be easier to replace the source with a properly designed and constructed well or spring or possibly to modify the source to eliminate the direct influence of surface water. Public water systems with groundwater sources under the direct influence of surface water are also subject to more stringent monitoring requirements for total coliform, turbidity, and entry point disinfection residual. The types of groundwater sources potentially regulated under the SWTR include dug wells, springs, infiltration galleries, shallow or improperly constructed wells, or other collectors in subsurface aquifers near surface waters.

Each local health department (LHD) throughout the United States is responsible for identifying which public water sources are subject to the SWTR. However, it is the responsibility of the water supplier to provide the information needed to make this determination to the LHD. The LHD is also responsible for recording and reporting the criteria used and the results of determinations. All groundwater sources used to supply public water systems must be evaluated for evidence of groundwater under the direct influence of surface-water GWUDI. This evaluation will focus on the likelihood that the groundwater source could be contaminated with large-diameter pathogens, such as *Giardia lamblia* and *Cryptosporidium*, through a hydraulic connection with surface water. If a drinking water source has been identified as GWUDI, the source must meet the criteria established under the SWTR.

3.20.1 GWUDI Determination: Source Screening Phase

Information gathered during sanitary surveys is important when making GWUDI determinations. In addition, information such as compliance monitoring data, topographic maps, geologic reports, well logs, and data on potential contaminant source(s) is useful.

A two-phase methodology is being used in the United States to determine whether or not a groundwater source is under the direct influence of surface water: the *source screening phase* is used first to separate those sources that are clearly not subject to surface-water influences from those sources in need of further evaluation. Then the second phase, the *detailed evaluation phase*, is applied to sources identified for testing to evaluate their degree of hydraulic connection with surface water.

The source screening phase should be used to separate those sources that are clearly not subject to surface-water influences from those sources in need of further evaluation. A schematic of the screening procedure is shown in Fig. 3.17, and the overall methodology for the detailed evaluation phase is presented as a flowchart in Fig. 3.18.

Box 1 in Fig. 3.17 includes criteria that will immediately select a groundwater source for further review. These source screening water criteria include the following: (a) a

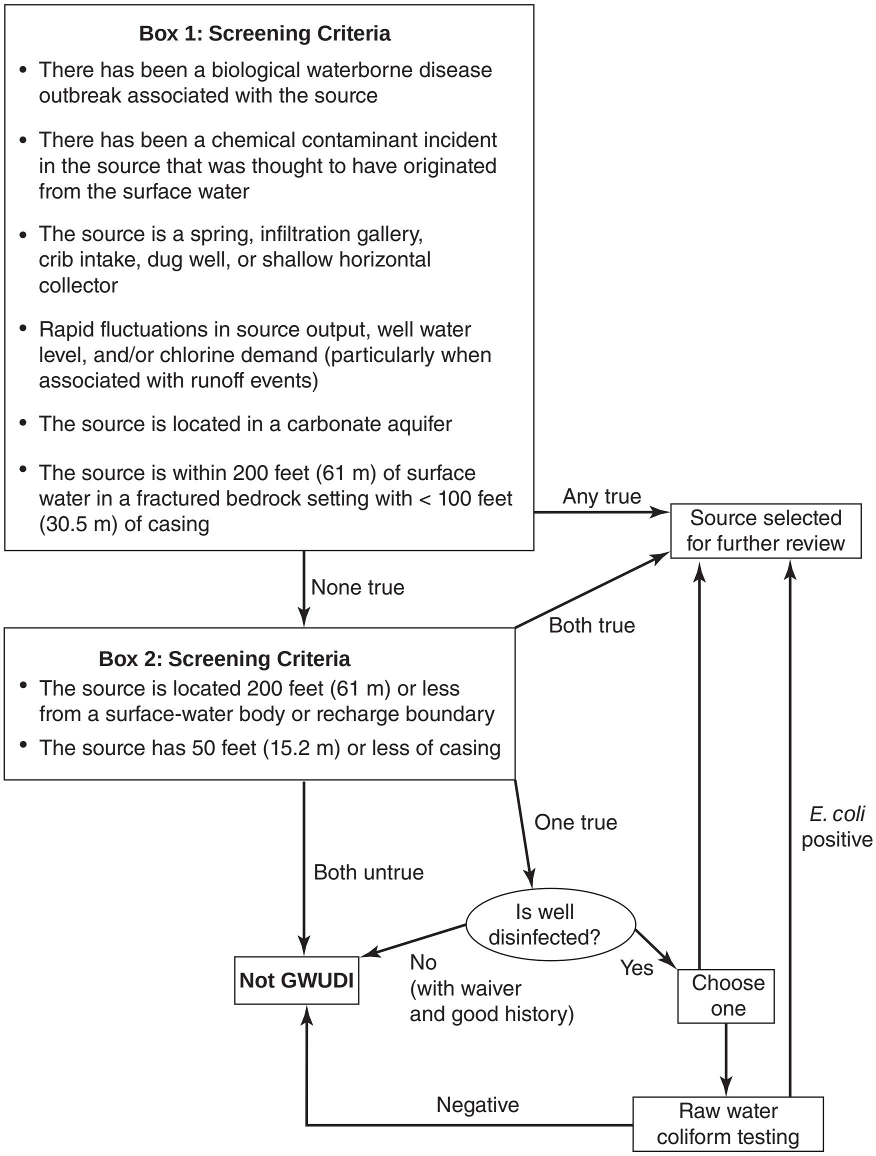


Figure 3.17 Identification of groundwater sources under the direct influence of surface water (GWUDI)—source screening phase methodology.

biological waterborne disease outbreak has been associated with the source; (b) a chemical contaminant incident has occurred in the source that was thought to have originated from the surface water; (c) the source is a spring, infiltration gallery, crib intake, dug well, or shallow horizontal collector; (d) rapid fluctuations have been noted in the source output, well water level, and/or chlorine demand (particularly when associated with runoff events); (e) the source is located in a carbonate aquifer; and (f) the source is within 200 ft (60 m) of surface water in a fractured bedrock setting with <100 ft(30 m) of casing. If none of these criteria are met, then the screening process continues in Box 2 of Fig. 3.17. If any of these criteria is met, then the screening process continues to the box "Source selected for further review."

The criteria in Box 2 of Fig. 3.17 ask if the source is located within 200 ft (60 m) of surface water, and/or if the well has 50 ft (15 m) or less of casing. When both criteria are met, the source is selected for further review under the box "Source selected for further review." If neither of these conditions is met, the source is designated as not being under the direct influence of surface water, or not-GWUDI. When only one of the criteria in Box 2 of Fig. 3.17 is met, the next step in the GWUDI determination is dependent on whether or not the water source is currently disinfected. Undisinfected wells that have met the criteria for a disinfection waiver and have an adequate coliform monitoring history (typically 5 years of quarterly monitoring) are designated as not being under the direct influence of surface water. Disinfected wells are subject to the options of either performing 1 year of monthly raw water coliform monitoring or moving directly into the detailed evaluation phase. Any raw water sample that is positive for *E. coli* would require the system to perform a detailed evaluation. Conversely, once 1 year of satisfactory monthly raw water coliform samples have been obtained,

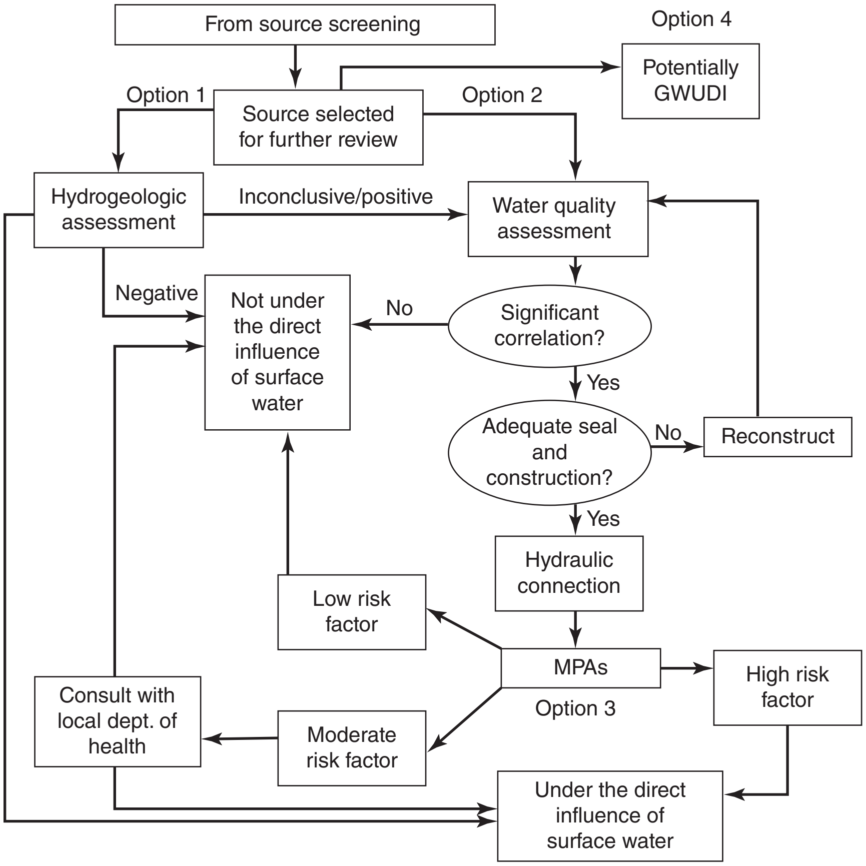


Figure 3.18 Identification of groundwater sources under the direct influence of surface water—detailed evaluation phase methodology.

the source can be designated as not being under the direct influence of surface water, or not-GWUDI.

3.20.2 GWUDI Determination: Detailed Evaluation Phase

Once a groundwater source has been selected for further review, as a result of the source screening phase, the procedure described in Fig. 3.18 should be closely followed.

The detailed evaluation phase has three components, as shown in Fig. 3.18: (a) hydrogeologic assessment, (b) water quality assessment, and (c) microscopic particulate analysis (MPA). The water supplier makes the decision about whether to begin with option 1 (hydrogeologic assessment), option 2 (water quality assessment), or option 3 (MPAs). Either the hydrogeologic assessment or water quality assessment is capable of providing the information required to determine that no surface-water influence is present. Sometimes, for example, when the project time is extremely important, the project engineer may want to choose option 3 (MPAs) directly or even option 4 (accept GWUDI determination).

If the hydrogeologic assessment option was chosen, but its results were inconclusive, the project engineer who represents the groundwater source owner/developer may request further water quality assessment (option 2) or MPAs (option 3) to complete the detailed evaluation phase, or simply accept the GWUDI determination (option 4).

Most project engineers would like to choose the straightforward option of water quality assessment instead of the complex option 1, hydrogeologic assessment, for their clients. If the option 2 test fails, then option 3 follows.

The MPA option is usually chosen, conducted during times of worst-case GWUDI conditions, as indicated by the water quality assessment or as predicted by historical information.

3.20.3 Hydrogeologic Assessment

As discussed and shown in Fig. 3.18, a water system has four evaluation options. Option 1 requires a detailed hydrogeologic assessment that addresses the potential of surface water to move quickly to the subsurface collection device. However, at any time during the hydrogeologic assessment, the water system's engineer can halt an ongoing evaluation by accepting a GWUDI designation (option 4) and making the appropriate modifications to bring the system into compliance with the SWTR, or changing it to option 2 or option 3.

If the hydrogeologic assessment is selected, and results of the assessment indicate that the aquifer supplying the source is not in hydraulic connection with surface water, no further analysis will be required. However, if the LHD determines that the hydrogeologic assessment does not contain enough information to establish whether there is a hydraulic connection between surface water and the source-water collection device, the water supplier should collect additional hydrogeologic information or proceed with a water quality assessment or MPAs.

The hydrogeologic assessment should be designed to provide the following information:

1. Well construction details, which provide (a) a well log, well construction diagram, and well description; and (b) its installation methods for comparison with current well standards
2. Aquifer characteristics, data which include (a) aquifer geometry and texture, including the unsaturated zone; (b) saturated thickness; (c) hydraulic conductivity; and (d) transmissivity
3. Prepumping and pumping groundwater flow characteristics data, which include (a) water table/potentiometric surface elevations; (b) groundwater flow directions; (c) groundwater flow velocity; and (d) zone of contribution/influence of the well
4. Degree of hydraulic connection between surface-water source and aquifer, which is indicated by (a) geology underlying surface-water body; (b) characterization of bottom sediments in surface-water body; (c) determination of vertical hydraulic gradient in surface-water body; (d) hydraulic relationship between the surface-water body and the well; and (e) calculations of travel times between the surface-water body and the well
5. Seasonal variations in hydrogeologic characteristics, such as the changes in flow patterns during seasonal fluctuations or periods of drought

The hydrogeologic assessment should include, as a minimum, geologic logs and construction details for the pumping well and any observation wells or piezometers; aquifer pumping test(s); a survey of the elevations of water level monitoring measuring points; water level monitoring of groundwater and surface sources; and preparation of detailed maps of water table/potentiometric surface and geologic cross-sections. The owner/developer may need to install observation/monitoring wells or piezometers if these do not already exist.

In addition to requiring an assessment of hydrogeologic factors, the hydrogeologic assessment should include a description and review of the collection device (i.e., type, age) and a summary of any current or historical sanitary conditions. Any information available from previous sanitary surveys or field investigations should be included in the assessment, as appropriate. Table 3.6 and Fig. 3.19 provide examples of sanitary and field survey reports that can be used for information gathering.

To be definitive, a hydrogeologic assessment needs to include an interpretation of the information collected with respect to the potential for a hydraulic connection between a surface-water body and the aquifer. If the hydrogeologic assessment indicates a potential hydraulic connection, the water system should be required to initiate a water quality assessment.

3.20.4 Water Quality Assessment

Option 2, a water quality assessment, entails either (a) an evaluation of water quality parameters in terms of conductivity and temperature daily over a 12-month period or (b) an evaluation of water quality parameter in terms of monthly coliform levels for 12 consecutive months.

Conductivity, or specific conductance, is the measure of water's ability to carry an electric current. This ability depends on the presence of ions in the water and the water's temperature. Groundwater is generally higher in conductivity than surface water, because groundwater dissolves minerals from substrates through which it moves. Generally, the longer the contact times between groundwater and its aquifer, the higher the conductivity. However, there are exceptions to this generalization (e.g., surface-water bodies receiving large amounts of groundwater recharge or surface-water bodies contaminated with salts, clays, metals, or polar organics). Conductivity data are especially important when making determinations for springs (and other situations where large seasonal fluctuations in temperature are expected). Overall, conductivity tends to be a more sensitive parameter than temperature and more difficult to interpret.

Once water quality assessment data have been collected and analyzed, a determination must be made regarding whether there is a significant hydraulic connection between the surface-water body and the groundwater source. A significant hydraulic connection exists when water movement from the surface-water body to the groundwater source allows for the transport of *Giardia lamblia* cysts or *Cryptosporidium* oocysts. Dilution and time of travel estimates should be considered when determining the significance of the hydraulic connection. If the time of travel estimate for the source is less than 100 days, a significant hydraulic connection should be assumed and the supplier should proceed to the next step, MPAs (option 3).

3.20.5 Microscopic Particulate Analyses

When the hydrogeologic assessment (option 1) and/or water quality assessment (option 2) results suggest that the groundwater source is probably under the direct influence of surface water, then the water supplier may either accept the GWUDI determination or choose to conduct MPAs, as shown

Table 3.6 Example of sanitary and field survey report for hydrogeologic assessment

Well location and characteristics
Water system name/address/Fed ID# ______________________
____
____
Name of site ______________________
Description of device ______________________
Name and description of nearby surface source(s) ______________________
____
____
Describe datum for elevations ______________________
Is there a USGS gauging station nearby, or are other flow records available?
____
____
Static water level
  As a depth ____________ Date ____________
  As an elevation ____________ Date ____________
Pumping water level
  As a depth ____________ Date ____________
  As an elevation ____________ Date ____________
Characterize pumping practices or spring flow (rates, seasonally, etc.)
____
____
Describe the vertical distance between the aquifer and the surface water under pumping conditions
____
____
Describe the horizontal distance between the aquifer and the surface-water source under pumping conditions ____________
____
Well construction
Construction date ______________________
Does construction conform to the current standards (i.e., Ten State)? ______________________
Construction methods/materials ______________________
____
  Casing ______________________
  Grouting ______________________
  Screening ______________________
  Well house ______________________
  Other information ______________________
Is a detailed plan/drawing available? ______________________
Hydrogeology
Is a geologic log for this well available? ______________________
What is the thickness of the unsaturated zone? ______________________
What is the hydraulic conductivity of the unsaturated zone and the aquifer? ______________________
Is there a confining layer? At what elevation(s)? ______________________
Describe ______________________
Does the surface-water body penetrate the aquifer? ______________________
Draw the aquifer(s), confining unit(s), and water table using one or more cross-sections with at least one cross-section parallel to the flow direction ______________________
Summary of sanitary conditions
Describe the results of past sanitary surveys and give the location of written reports ______________________
____
____

in Fig. 3.18. Information collected as part of the characterization of hydraulic communication and time of travel will be very important in determining the correct timing of MPA sample collection. MPA samples must be collected at least twice, and the dates of sampling should represent worst-case conditions, when maximum potential recharge from the nearby surface water is taking place (usually during extremely wet or dry periods).

Basically, the use of MPA in a GWUDI determination involves the careful enumeration of microscopic organisms

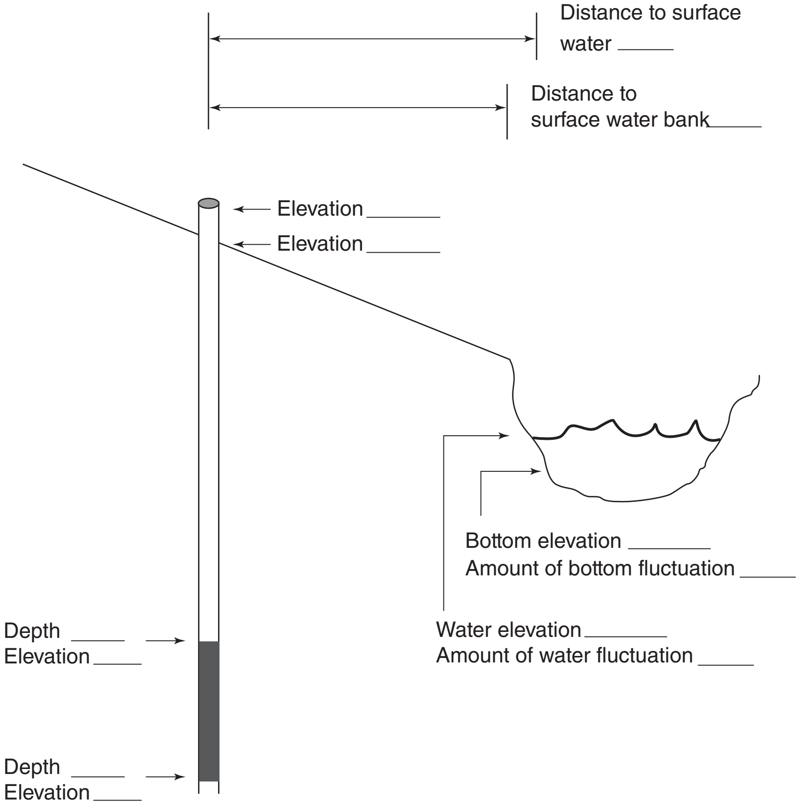


Figure 3.19 Investigation for hydrogeological assessment.

(and other particulates) in the raw drinking water. These data are then systematically evaluated to determine if the particles found are more indicative of surface water or groundwater. Some of the organisms that are considered to be characteristic of surface water include *Giardia*, *Cryptosporidium*, algae, diatoms, and rotifers.

Note that the hydrogeologic information and the results of the water quality assessment must be used in conjunction with MPAs to make the GWUDI determinations. Most importantly, MPA samples should be collected at a time when the water quality assessment and hydrogeologic data indicate the greatest probability that surface water is impacting the groundwater source, as indicated by hydrogeologic and water quality data. It is relatively safe to conclude from a "high" MPA rating that the groundwater source is under the direct influence of surface water, particularly when considered along with corroborating information collected in the earlier phases of a GWUDI evaluation. Moreover, it is more difficult to conclude from a "low" MPA rating that the groundwater source is not under the direct influence of surface water, because surface-water influences often only occur intermittently under particular hydrologic conditions (usually during very wet or dry periods).

PROBLEMS/QUESTIONS

3.1 Differentiate between porosity and permeability and between specific retention and specific yield.

3.2 Differentiate among aquifers, aquicludes, and aquitards.

3.3 Explain the classification of aquifers, and why the unconfined aquifers can be easily contaminated.

3.4 Demonstrate the relatively slow rate of groundwater movement by using Eq. (3.2) to determine the rate of movement through an aquifer and a confining bed. The following data are assumed:

- Aquifer composed of coarse sand: $K = 60\,\mathrm{m/d}, \mathrm{d}h/\mathrm{d}l = I = 1\,\mathrm{m}/1{,}000\,\mathrm{m}$ and $\theta = 0.20$.
- Confining bed composed of clay: $K = 0.0001\ \mathrm{m/d}, \mathrm{d}h/\mathrm{d}l = I = 1\,\mathrm{m}/10\,\mathrm{m}$ and $\theta = 0.50$.

3.5 The observed data from a pumping test were plotted in a figure similar to Fig. 3.4 along with a Theis-type curve, as if the transparency of the observed data had been moved into place over the type curve. The observation well represented by the data is 400 ft (121.92 m) from a pumping well where the rate of discharge is 275 gpm (1498.75 m^3/d). Calculate the formation constants T and S using these match-point coordinates:

$$\begin{aligned} W(u) &= 3.0 \\ s &= 3.7\,\mathrm{ft}\,(1.13\,\mathrm{m}) \\ u &= 2.3 \times 10^{-2} \\ r^2/t &= 1.4 \times 10^7\ (\mathrm{SI}\,1.3 \times 10^6). \end{aligned}$$

3.6 A time–drawdown curve for an observation well at a distance of 400 ft (121.92 m) from a pumping well discharging at a constant rate of 275 gpm (1498.75 m^3/d) is shown in Fig. 3.5. Determine the transmissivity and storage coefficient of the aquifer.

3.7 In the aquifer represented by the pumping test in Problem 3.5 ($T = 2.6 \times 10^4$ gpd/ft $= 322.87$ m^3/d/m and $S = 2.3 \times 10^{-5}$), a gravel-packed well with an effective diameter of 36 in. (914 mm) is to be constructed. The design flow of the well is 1,000 gpm (5450 m^3/d). Calculate the drawdown at the well with total withdrawals from storage (i.e., with no recharge or leakages) after (a) 1 min, (b) 1 h, (c) 1 day, (d) 1 month, and (e) 1 yr of continuous pumping, at design capacity.

3.8 Determine the drawdown of a quasi-steady state for a proposed 36 in. (914 mm) effective diameter well pumping continuously at 350 gpm (1907 m^3/d) in an elastic artesian aquifer having a transmissivity of 5000 gpd/ft (62 m^3/d/m) and a storage coefficient of 4×10^{-4}. Assume that the discharge and recharge conditions are such that the drawdown will be stabilized after 180 days.

3.9 A well was pumped at a constant rate of 500 gpm (2,725 m^3/d) between 8 a.m. and 11 a.m., 2 p.m. and 5 p.m., and remained idle the rest of the time. What will be the drawdown in the well at 8 a.m. the next day when a new cycle of pumping is to start? Assume no recharge or leakage. The transmissivity of the artesian aquifer is 4.1×10^4 gpd/ft $(509.14\,\text{m}^3/\text{d/m})$.

3.10 Two wells with effective diameters of 36 in. (914 mm) are located 1,500 ft (457.2 m) apart in an artesian aquifer with $T = 2.5 \times 10^4$ gpd/ft (310.45 m^3/d/m) and $S = 4 \times 10^{-5}$. Compute the drawdown at each well when the two wells are pumped at 500 gpm (2725 m^3/d) for 20 days.

3.11 Suppose it is desired to restrict the drawdown in each of the wells to 40 ft in Problem 3.10. What will be the corresponding discharges for individual wells?

3.12 A gravel-packed well with an effective diameter of 36 in. (914 mm) pumps water from an artesian aquifer having $T = 4.2 \times 10^4$ gpd/ft (521.56 m^3/d/m) and $S = 4.4 \times 10^{-5}$. The well lies at a distance of 2,000 ft (610 m) from a stream that can supply water fast enough to maintain a constant head. Find the drawdown in the well after 20 days of pumping at 550 gpm (2725 m^3/d).

3.13 The interrelationship of pumping rate, drawdown, and specific yield serves as a basis for the selection of optimal design capacity. List the factors on which the obtainable yield from a well depends.

3.14 A well having an effective diameter of 36 in. (914 mm) is to be located in a relatively homogeneous artesian aquifer with a transmissivity of 15,000 gpd/ft (186.27 m^3/d/m) and a storage coefficient of 3×10^{-4}. The initial piezometric surface level is 30 ft (9.14 m) below the land surface. The depth to the top of the aquifer is 140 ft (42.67 m) and the thickness of the aquifer is 60 ft (18.28 m). The well is to be finished with a screen length of 30 ft (9.14 m). Compute the specific capacity of the well and its maximum yield after 10 days of pumping. Neglect well losses.

3.15 Discuss the sanitary protection of wells constructed for supplying drinking water.

3.16 Visit the US Environmental Protection Agency website (www.epa.org) and find the Groundwater Rule (GWR). Explain the background of the GWR and outline briefly the most recent final GWR final requirements.

3.17 About 25% of Earth's freshwater supply is stored beneath the surface of the land, where it remains for thousands of years. Only a relatively small proportion of this groundwater is located in soil formations (aquifers) from which it can be withdrawn in significant amounts. Explain the following briefly:

(a) The reasons why groundwater is the principal source of water for small systems,

(b) The US EPA's Groundwater Rule promulgation,

(c) The meaning of "groundwater under the direct influence of surface water" (GWUDI) and the water monitoring and treatment requirements of GWUDI.

3.18 Define the technical terms of (a) well capacity; (b) unconfined aquifer; (c) confined aquifer; (d) zone of aeration; and (e) zone of saturation.

3.19 Define the technical terms of (a) groundwater recharge; (b) groundwater storage; (c) groundwater safe yield; (d) impounding reservoir safe yield; and (e) river safe yield.

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Chapter 4

Quantities of Water Demand

Knowledge of the required quantities of water flows is fundamental to systems design and management. In the United States the volumes of water supplied to cities and towns or removed from them are expressed in US gallons per year (gal/yr), month (gal/month), day (gpd), or minute (gpm). The US gallon occupies a volume of 0.1337 cubic feet (ft^3) or 3.785 liters (L) and weighs 8.344 pounds (lb) or 3.785 kilograms (kg). The fundamental metric (SI) unit in engineering work is the cubic meter (m^3), weighing 1,000 kilograms (kg) or 1 metric tonne (T) and equaling 1,000 liters (L). In the United States, annual water or wastewater volumes are conveniently recorded in million gallons (MG) or million liters (ML). Daily volumes are generally expressed in million gallons per day (MGD) or million liters per day (MLD) if more than 100,000 gpd (378,500 L/d). The gallons per capita daily volumes are stated in gpcd while the liters per capita daily volumes are stated in Lpcd. Connected or tributary populations and numbers of services or dwelling units may take the place of total populations.

Per capita and related figures generalize the experience. They permit comparison of the experience and practices of different communities and are helpful in estimating future requirements of specific communities. Fluctuations in flow are usefully expressed as ratios of maximum or minimum annual, seasonal, monthly, weekly, daily, hourly, and peak rates of flow to corresponding average rates of flow.

Most water and wastewater systems include massive structures (dams, reservoirs, and treatment works) that have long construction timelines and are not readily expanded; they also include pipes and other conduits sunk into city streets, which disrupt traffic while they are being laid. Accordingly, the principal system components are purposely made large enough to satisfy community needs for a reasonable number of years. For this reason, selecting the initial or design capacity is not simple. It calls for skill in interpreting social and economic trends and sound judgment in analyzing past experience and predicting future requirements. Among needed estimates are the following:

1. The number of years, or *design period*, for which the proposed system and its component structures and equipment are to be adequate
2. The number of people, or *design population*, to be served
3. The rates of water use and wastewater release, or *design flows*, in terms of per capita water consumption and wastewater discharge as well as industrial and commercial requirements
4. The area to be served, or *design area*, and the allowances to be made for population density and areal water consumption as well as water supply and wastewater release from residential, commercial, and industrial districts
5. The rates of rainfall and runoff, or *design hydrology*, for storm and combined systems

4.1 DESIGN PERIOD

New water and wastewater works are normally made large enough to meet the needs and wants of growing communities for an economically justifiable number of years in the future. Choice of a relevant design period is generally based on

1. The useful life of component structures and equipment, taking into account obsolescence as well as wear and tear
2. The ease or difficulty of enlarging contemplated works, including consideration of their location
3. The anticipated rate of population growth and water use by the community and its industries
4. The going rate of interest on bonded indebtedness
5. The performance of contemplated works during their early years when they are expected to be under minimum load

Design periods often employed in practice are shown in Table 4.1.

Water Engineering: Hydraulics, Distribution and Treatment, First Edition. Nazih K. Shammas and Lawrence K. Wang.

Table 4.1 Design periods for water and wastewater structures

Type of structure	Special characteristics	Design period (years)
	Water supply	
Large dams and conduits	Hard and costly to enlarge	25–50
	Easy to extend	
Wells, distribution systems, and filter plants	When growth and interest rates are low[a]	20–25
	When growth and interest rates are high[a]	10–15
Pipes more than 12 in. (304.8 mm) in diameter	Replacement of smaller pipes is more costly in the long run	20–25
Laterals and secondary mains less than 12 in. (304.8 mm) in diameter	Requirements may change fast in limited areas	Full development
	Sewerage	
Laterals and submains less than 15 in. (381 mm) in diameter	Requirements may change fast in limited areas	Full development
Main sewers, outfalls, and interceptors	Hard and costly to enlarge	40–50
Treatment works	When growth and interest rates are low[a]	20–25
	When growth and interest rates are high[a]	10–15

Conversion factor: 1 in. = 25.4 mm.

[a] The dividing line is in the vicinity of 3% per annum.

4.2 DESIGN POPULATION

4.2.1 Population Data

For information on the population of given communities or regions at a given time, engineers turn to the records of official censuses or enumerations. The US government has conducted a decennial census since 1790. Some state and local enumerations provide additional information, usually for years ending in 5, and results of special surveys sponsored by public authorities or private agencies for political, social, or commercial purposes may also be available. US census dates and intervals between censuses are listed in Table 4.2.

The information obtained in the decennial censuses is published by the US Bureau of the Census, Department of Commerce. Political or geographic subdivisions for which population data are collated vary downward in size from the country as a whole, to its coterminous portion only, individual states and counties, metropolitan districts, cities and wards, townships and towns, and—in large communities—census tracts. The tracts are areas of substantially the same size and large enough to house 3,000–6,000 people.

Table 4.2 US census dates and intervals between censuses

Year	Date	Census interval (years)
1790–1820	First Monday in August	Approximately 10
1830–1900	June 1	Exactly 10, except between 1820 and 1830
1910	April 15	9.875
1920	January 1	9.708
1930	April 1	10.250
1940–2010	April 1	Exactly 10

4.2.2 Population Growth

Populations increase by births, decrease by deaths, and change with migration. Communities also grow by annexation. Urbanization and industrialization bring about social and economic changes as well as growth. Educational and employment opportunities and medical care are among the desirable changes. Among unwanted changes are the creation of slums and the pollution of air, water, and soil. Least predictable of the effects on growth are changes in commercial and industrial activity. Examples are furnished in Table 4.3 (a) for Detroit, MI, where the automobile industry was responsible for a rapid rise in population between 1910 and 1950; (b) for Providence, RI, where competition with southern textile mills was reflected in low rates of population

Table 4.3 Census populations of Detroit, MI; Providence, RI; and Miami, FL; 1910–2006

	City		
Census year	(a) Detroit	(b) Providence	(c) Miami
1910	466,000	224,000	5,500
1920	994,000	235,000	30,000
1930	1,569,000	253,000	111,000
1940	1,623,000	254,000	172,000
1950	1,850,000	249,000	249,000
1960	1,670,000	207,000	292,000
1970	1,493,000	177,000	332,000
1980	1,203,000	157,000	347,000
1990	1,028,000	161,000	359,000
2000	951,000	174,000	363,000
2010	714,000	178,000	401,000

Source: After US Bureau of the Census.

All population values have been rounded to the nearest thousand.

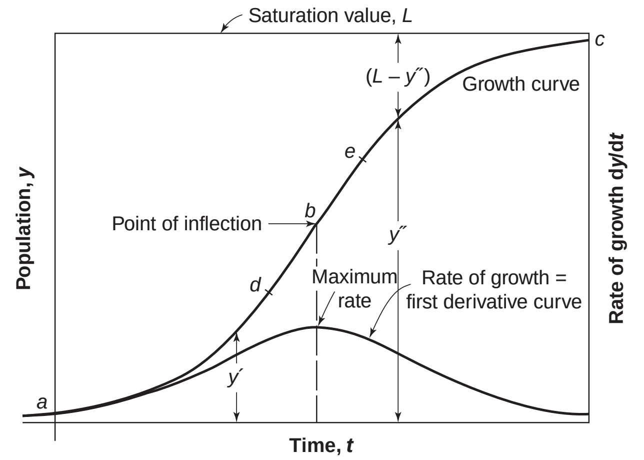

Figure 4.1 Population growth idealized. Note geometric increase from *a* to *d*, straight-line increase from *d* to *e*, and first-order increase from *e* to *c*.

growth after 1910; and (c) for Miami, FL, where recreation added a new and important element to prosperity from 1910 onward.

Were it not for industrial vagaries of the Providence type, human population kinetics would trace an S-shaped growth curve in much the same way as spatially constrained microbial populations. As shown in Fig. 4.1, the trend of seed populations is progressively faster at the beginning and progressively slower toward the end as a saturation value or upper limit is approached. What the future holds for a given community, therefore, is seen to depend on where on the growth curve the community happens to be at a given time.

The growth of cities and towns and characteristic portions of their growth curves can be approximated by relatively simple equations that derive historically from chemical kinetics. The equation of a first-order chemical reaction, possibly catalyzed by its own reaction products, is a recurring example. It identifies also the kinetics of biological growth and other biological reactions including population growth, kinetics, or dynamics. This widely useful equation can be written as

$$\mathrm{d}y/\mathrm{d}t = ky(L - y) \tag{4.1}$$

where y is the population at time t, L is the saturation or maximum population, and k is a growth or rate constant with the dimension $1/t$. It is pictured in Fig. 4.1 together with its integral, Eq. (4.6).

Three related equations apply closely to characteristic portions of this growth curve: (a) a first-order progression for the terminal arc *ec* of Fig. 4.1, (b) a logarithmic or geometric progression for the initial arc *ad*, and (c) an arithmetic progression for the transitional intercept *de*, or

$$\text{For arc } ec\text{:}\quad \mathrm{d}y/\mathrm{d}t = k(L - y) \tag{4.2}$$

$$\text{For arc } ad\text{:}\quad \mathrm{d}y/\mathrm{d}t = ky \tag{4.3}$$

$$\text{For arc } de\text{:}\quad \mathrm{d}y/\mathrm{d}t = k \tag{4.4}$$

If it is assumed that the initial value of k, namely k_0, decreases in magnitude with time or population growth rather than remaining constant, k can be assigned the following value:

$$k = k_0/(1 + nk_0t) \tag{4.5}$$

in which n, as a coefficient of retardance, adds a useful concept to Eqs. (4.2)–(4.4).

On integrating Eqs. (4.1)–(4.4) between the limits $y = y_0$ at $t = 0$ and $y = y$ at $t = t$ for unchanging k values, they change as shown next.

For autocatalytic first-order progression (arc *ac* in Fig. 4.1),

$$\ln[(L - y)/y] - \ln[(L - y_0)/y_0] = -kLt$$

or

$$y = L/\{1 + [(L - y_0)/y_0]\,\exp(-kLt)\} \tag{4.6}$$

For first-order progression without catalysis (arc *ec* in Fig. 4.1),

$$\ln[(L - y)/(L - y_0)] = -kt$$

or

$$y = L - (L - y_0)\exp(-kt) \tag{4.7}$$

For geometric progression (arc *ad* in Fig. 4.1),

$$\ln(y/y_0) = kt$$

or

$$y = y_0\exp(kt) \tag{4.8}$$

For arithmetic progression (arc *de* in Fig. 4.1),

$$y - y_0 = kt \tag{4.9}$$

Substituting Eq. (4.5) into Eqs. (4.2)–(4.4) yields the retardant expressions shown next. For retardant first-order progression,

$$y = L - (L - y)(1 + nk_0t)^{-1/n} \tag{4.10}$$

For retardant, geometric progression,

$$\ln(y/y_0) = (1/n)\ \ln(1 + nk_0t)$$

or

$$y = y_0(1 + nk_0t)^{-1/n} \tag{4.11}$$

For retardant, arithmetic progression,

$$y - y_0 = (1/n)\ \ln(1 + nk_0t) \tag{4.12}$$

These and similar equations are useful in water and wastewater practice, especially in water and wastewater treatment kinetics.

4.2.3 Short-Term Population Estimates

Estimates of midyear populations for current years and the recent past are normally derived by arithmetic from census data. They are needed perhaps most often for (a) computing per capita water consumption and wastewater release and (b) calculating the annual birth and general death rates per 1000 inhabitants, or specific disease and death rates per 100,000 inhabitants.

Understandably, morbidity and mortality rates from waterborne and otherwise water-related diseases are of deep concern to sanitary engineers.

For years between censuses or after the last census, estimates are usually interpolated or extrapolated as arithmetic or geometric progressions. If t_i and t_j are the dates of two sequent censuses and t_m is the midyear date of the year for which a population estimate is wanted, the rate of arithmetic growth is given by Eq. (4.9) as

$$k_{\text{arithmetic}} = y_j - y_i/(t_j - t_i)$$

and the midyear populations, y_m, of intercensal and postcensal years are as follows:

Intercensal:

$$y_m = y_i + (t_m - t_i)(y_j - y_i)/(t_j - t_i) \qquad \textbf{(4.13)}$$

Postcensal:

$$y_m = y_j + (t_m - t_j)(y_j - y_i)/(t_j - t_i) \qquad \textbf{(4.14)}$$

In similar fashion, Eq. (4.8) states that

$$k_{\text{arithmetic}} = (\log y_i - \log y_j)/(t_j - t_i)$$

and the logarithms of the midyear populations, log y_m, for intercensal and postcensal years are as follows:

Intercensal:

$$\log\, y_m = \log\, y_i + (t_m - t_i)(\log\, y_j - \log\, y_i)/(t_j - t_i) \qquad \textbf{(4.15)}$$

Postcensal:

$$\log\, y_m = \log\, y_j + (t_m - t_j)(\log\, y_j - \log\, y_i)/(t_j - t_i) \qquad \textbf{(4.16)}$$

Geometric estimates, therefore, use the logarithms of the population parameters in the same way as the population parameters themselves are employed in arithmetic estimates; moreover, the arithmetic increase corresponds to capital growth by simple interest, and the geometric increase to capital growth by compound interest. Graphically, arithmetic progression is characterized by a straight-line plot against arithmetic scales for both population and time on double-arithmetic coordinate paper and, thus, geometric as well as first-order progression by a straight-line plot against a geometric (logarithmic) population scale and an arithmetic timescale on semilogarithmic paper. The suitable equation and method of plotting are best determined by inspection from a basic arithmetic plot of available historic population information.

EXAMPLE 4.1 ESTIMATION OF POPULATION

As shown in Table 4.3, the rounded census population of Miami, FL, was 249,000 in 1950 and 292,000 in 1960. Estimate the midyear population (1) for the fifth intercensal year and (2) for the ninth postcensal year by (a) arithmetic and (b) geometric progression. The two census dates were both April 1.

Solution:

Intercensal estimates for 1955:

t_m = 1955.25 (there are 3 months = 0.25 years, from April 1 to midyear, June 30)

$t_m - t_i$ = 1955.25 − 1950 = 5.25 years

$t_j - t_i$ = 1960 − 1950 = 10.00 years

$(t_m - t_i)/(t_j - t_i)$ = 5.25/10.00 = 0.525

	(a) Arithmetic	(b) Geometric
1960	y_j = 292,000	log y_j = 5.4654
1950	y_i = 249,000	log y_i = 5.3962
	$y_j - y_i$ = 43,000	log y_j − log y_i = 0.0692
	0.525($y_j - y_i$) = 23,000	0.525(log y_j − log y_i) = 0.03633
1955	**y_m = 272,000**	**y_m = 268,000**

Postcensal estimate for 1969:

$t_m - t_i$ = 9.25 yr $\quad$ $t_j - t_i$ = 10.00 yr $\quad$ $(t_m - t_i)/(t_j - t_i)$ = 0.925

	(a) Arithmetic	(b) Geometric
1960	$y_j = 292{,}000$	$\log y_j = 5.4654$
1950	$y_i = 249{,}000$	$\log y_i = 5.3962$
	$y_j - y_i = 43{,}000$	$\log y_j - \log y_i = 0.0692$
	$0.925(y_j - y_i) = 40{,}000$	$0.925(\log y_j - \log y_i) = 0.0620$
1969	$\mathbf{y_m = 332{,}000}$	$\mathbf{y_m = 337{,}000}$

Geometric estimates are seen to be lower than arithmetic estimates for intercensal years and higher for postcensal years.

The US Bureau of the Census estimates the current population of the whole nation by adding to the last census population the intervening differences (a) between births and deaths, that is, the natural increases; and (b) between immigration and emigration. For states and other large population groups, postcensal estimates can be based on the apportionment method, which postulates that local increases will equal the national increase times the ratio of the local to the national intercensal population increase. Intercensal losses in population are normally disregarded in postcensal estimates; the last census figures are used instead.

Supporting data for short-term estimates can be derived from sources that reflect population growth in ways different from, yet related to, population enumeration. Examples are records of school enrollments; house connections for water, electricity, gas, and telephones; commercial transactions; building permits; and health and welfare services. These are translated into population values by ratios derived for the recent past. The following ratios are not uncommon:

1. Population:school enrollment = 5:1
2. Population:number of water, gas, or electricity services = 3:1
3. Population:number of land-line telephone services = 4:1

4.2.4 Long-Range Population Forecasts

Long-range forecasts, covering design periods of 10–50 years, make use of available and pertinent records of population growth. Again dependence is placed on mathematical curve fitting and graphical studies. The logistic growth curve is an example.

The logistic growth equation is derived from the autocatalytic, first-order equation (Eq. 4.6) by letting

$$p = (L - y_0)/y_0$$

and

$$q = kL$$

or

$$y = L/[1 + p\exp(-qt)]$$

$$y = L/[1 + \exp(\ln p - qt)]$$

and equating the first derivative of Eq. (4.1) to zero, or

$$\mathrm{d}(\mathrm{d}y/\mathrm{d}t)/\mathrm{d}t = kL - 2ky = 0$$

It follows that the maximum rate of growth $\mathrm{d}y/\mathrm{d}t$ is obtained when

$$y = 1/2L$$

and

$$t = (-\ln p)/q = (-2.303\log p)/q$$

It is possible to develop a logistic scale for fitting a straight line to pairs of observations as in Fig. 4.2. For general use of this scale, populations are expressed in terms

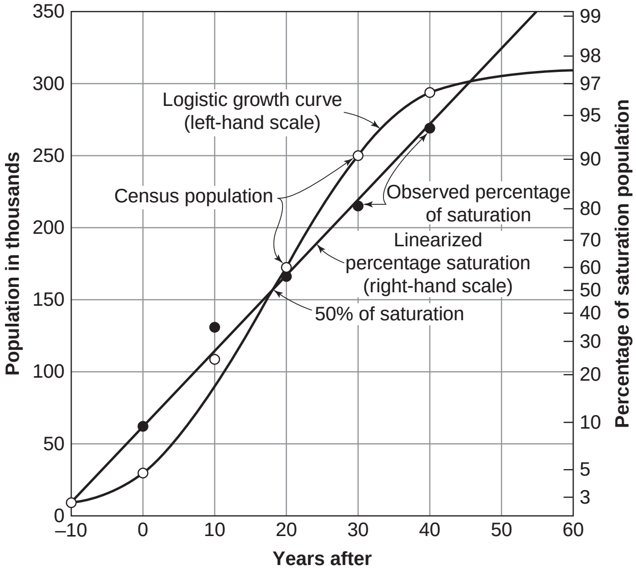

Figure 4.2 Logistic growth of a city. Calculated saturation population, confirmed by graphical good straight-line fit, is 313,000. Right-hand scale is plotted as log [(100 – P)/P] about 50% at the center.

of successive saturation estimates L, which are eventually verified graphically by lying closely in a straight line on a logistic-arithmetic plot. The percentage saturation P is

$$P = 100y/L = 100/[1 + p\ \exp(-qt)]$$

and

$$\ln[(100 - P)/P] = \ln p - qt$$

The straight line of best fit by eye has an ordinate intercept ln p and a slope $-q$ when $\ln[(100 - P)/P]$ is plotted against t or values of $n[(100 - P)/P]$ are scaled in either direction from a 50th percentile or middle ordinate.

Graphical forecasts offer a means of escape from mathematical forecasting. However, even when mathematical forecasting appears to give meaningful results, most engineers seek support for their estimates from plots of experienced and projected population growth on arithmetic or semilogarithmic scales. Trends in rates of growth rather than growth itself may be examined arithmetically, geometrically, or graphically with fair promise of success. Estimates of arithmetic and semilogarithmic straight-line growth of populations and population trends can be developed analytically by applying least-squares procedures, including the determination of the coefficient of correlation and its standard error.

At best, since forecasts of population involve great uncertainties, the probability that the estimated values turn out to be correct can be quite low. Nevertheless, the engineer must select values in order to proceed with planning and design of works. To use uncertainty as a reason for low estimates and short design periods can lead to capacities that are even less adequate than they otherwise frequently turn out to be. Because of the uncertainties involved, populations are sometimes projected at three rates—high, medium, and low. The economic and other consequences of designing for one rate and having the population grow at another can then be examined.

4.2.5 Simplified Method for Population Forecasts

The following are two simple equations that are used by consulting engineers for computing the rate of population increase and the future population forecast:

$$P_n = P_o(1 + R)^n \qquad \textbf{(4.17a)}$$

$$P_o = P_p(1 + R)^n \qquad \textbf{(4.17b)}$$

where

P_n = future population

P_o = present population

P_p = past population

R = probable rate of population increase per year

n = number of years considered

When population data (P_p, P_o, and n) for the past are available, the value of R in the preceding equations can be computed. In case the rate of population increase per year is changing, several R values can be calculated in order to obtain an average for the future population forecast (P_n) using the known data (R, P_o, and n).

Table 4.4 Common population densities

	Persons/acre	Persons/ha
1. Residential areas		
(a) Single-family dwellings, large lots	5–15	12–37
(b) Single-family dwellings, small lots	15–35	37–87
(c) Multiple-family dwellings, small lots	35–100	87–247
(d) Apartment or tenement houses	100–1,000 or more	247–2,470
2. Mercantile and commercial areas	15–30	37–74
3. Industrial areas	5–15	12–37
4. Total, exclusive of parks, playgrounds, and cemeteries	10–50	25–124

Conversion factors: 1 acre = 0.4046 ha = 4,047 m^2 = 0.004047 km^2; 1 person/acre = 2.4716 person/ha.

4.2.6 Population Distribution and Area Density

Capacities of water collection, purification, and transmission works and of wastewater outfall and treatment works are a matter of areal as well as population size. Within communities their individual service areas, populations, and occupancy are the determinants. A classification of areas by use and of expected population densities in persons per acre is shown in Table 4.4. Values of this kind are founded on analyses of present and planned future subdivisions of typical blocks. Helpful, in this connection, are census tract data; land-office, property, zoning, fire insurance, and aerial maps; and other information collected by planning agencies.

4.3 WATER CONSUMPTION

Although the draft of water from distribution systems is commonly referred to as *water consumption,* little of it is, strictly speaking, consumed; most of it is discharged as spent or wastewater. *Use of water* is a more exact term. True consumptive use refers to the volume of water evaporated or transpired in the course of use—principally in sprinkling lawns and gardens, in raising and condensing steam, and in bottling, canning, and other industrial operations.

Service pipes introduce water into dwellings, mercantile and commercial properties, industrial complexes, and public buildings. The water delivered is classified accordingly. Table 4.5 shows approximate per capita daily uses in the United States. Wide variations in these figures must be expected because of differences in

1. Climate
2. Standards of living

Table 4.5 Water consumption

Class of consumption	Quantity, gpcd (Lpcd) Normal range	Average
Domestic or residential	20–90 (75–340)	55 (210)
Commercial	10–130 (40–500)	20 (75)
Industrial	22–80 (80–300)	50 (185)
Public	5–20 (20–75)	10 (40)
Water unaccounted for	5–30 (20–115)	15 (60)
Total	60–350 (235–1,330)	150 (570)

Conversion factor: 1 gpcd = 3.785 Lpcd.

3. Type of residences
4. Extent of sewerage
5. Type of mercantile, commercial, and industrial activity
6. Water pricing
7. Use of private supplies
8. Water quality for domestic and industrial purposes
9. Distribution system pressure
10. Completeness of metering
11. Systems management

4.3.1 Domestic Consumption

Although domestic water use is about 50% of the water drawn in urban areas, 90% of the consumers are domestic. A breakdown of household flows apportions the various uses as follows:

1. 41% to flushing toilets
2. 37% to washing and bathing
3. 6% to kitchen use
4. 5% to drinking water
5. 4% to washing clothes
6. 3% to general household cleaning
7. 3% to watering lawns and gardens
8. 1% to washing family cars

Although domestic use is commonly expressed in gpcd (or Lpcd), the daily draft per dwelling unit, gpud (or Lpud), may offer more meaningful information.

Extremes of heat and cold increase water consumption: hot and arid climates by frequent bathing, air conditioning, and heavy sprinkling; and cold climates by bleeding water through faucets to keep service pipes and internal water piping from freezing during cold spells. In metered and sewered residential areas, the observed average daily use of water for lawns and gardens, $Q_{\text{sprinkling}}$, in gpd during the growing season is about 60% of the estimated average potential evapotranspiration E, reduced by the average daily precipitation, P, effective in satisfying evapotranspiration during the period, or

$$Q_{\text{sprinkling}} = 1.63 \times 10^4\, A\,(E - P) \quad \text{(US customary units)} \tag{4.18a}$$

where

$Q_{\text{sprinkling}}$ = average daily use of water for lawns and gardens (gpd)

1.63×10^4 = number of gallons in an acre-inch considering a 60% factor

A = average lawn and garden acreage per dwelling unit

E = average potential evapotranspiration expressed (in./d)

P = average daily precipitation (in./d)

The following is a sister equation using SI units:

$$Q_{\text{sprinkling}} = 6003\, A\,(E - P) \quad \text{(SI units)} \tag{4.18b}$$

where

$Q_{\text{sprinkling}}$ = average daily use of water for lawns and gardens (L/d)

A = average lawn and garden area per dwelling unit (ha)

E = average potential evapotranspiration expressed (mm/d)

P = average daily precipitation also expressed (mm/d)

The average lawn and garden area per dwelling unit A is given by the observational relationship

$$A = 0.803\, D^{-1.26} \quad \text{(US customary units)} \tag{4.19a}$$

where A is the average lawn and garden area, in acres per dwelling unit, and D is the gross housing density, in dwelling units per acre. The following is its sister equation using SI units:

$$A = 1.016\, D^{-1.26} \quad \text{(SI units)} \tag{4.19b}$$

where A is the average lawn and garden area, in ha per dwelling unit, and D is the gross housing density, in dwelling units per ha.

In arid climates, Shammas (1991) studied the relationship between actual water application and evapotranspiration rates for various categories of green areas in Riyadh, Saudi Arabia (Table 4.6). As expected, irrigation water application rates increased with rising evapotranspiration rates for every category of green areas. For private residences, as evapotranspiration rates rose from 3.4 to 5.3 to 8.0 mm/d (L/m^2/d), the corresponding water application rates increased from 7 to 10 to 23 mm/d (L/m^2/d). Similar behavior can be noticed for the other categories of planted areas.

High standards of cleanliness, large numbers of water-connected appliances, oversized plumbing fixtures, and frequent lawn and garden sprinkling—all of which are associated with wealth—result in heavy drafts. For sewered

Table 4.6 Relationship of irrigation water application rates to evapotranspiration in arid areas

	Period									
	December–January			February–March			April–May			
Category	I^a (L/m^2/d)	E^b (mm/d)	I/E ratio	I^a (L/m^2/d)	E^b (mm/d)	I/E ratio	I^a (L/m^2/d)	E^b (mm/d)	I/E ratio	Overall I/E ratio
Private residences	7.0	3.4	2.1	10	5.3	1.9	23	8.0	2.9	2.3
Road medians	15	3.4	4.4	15	5.3	2.8	33	8.0	4.1	3.8
Public parks	3.0	3.4	0.9	7.0	5.3	1.3	12	8.0	1.5	1.2
Special developments	1.0	3.4	0.3	3.0	5.3	0.6	6.0	8.0	0.8	0.6

Conversion factors: 1 gpd/ft^2 = 0.0408 m^3/m^2/d = 40.8 L/m^2/d; 1 in./d = 25.4 mm/d.
[a] I = irrigation rate (L/m^2/d).
[b] E = evapotranspiration rate (mm/d equivalent to L/m^2/d).

properties, the average domestic use of water Q_{domestic}, in gpud or Lpud for each dwelling unit, is related to the average market value M of the units in thousands of current US dollars by the following observational equations:

$$Q_{\text{domestic}} = 157 + 3.46MF \quad \text{(US customary units)} \tag{4.20a}$$

$$Q_{\text{domestic}} = 594 + 13.1MF \quad \text{(SI units)} \tag{4.20b}$$

Where F is the site/time factor to be determined by a planning engineer periodically. It is important to note that these mathematical models do vary with time and locations; therefore, they must be modified and updated for a specific site/time situations.

4.3.2 General Urban Water Demands

Some commercial enterprises—hotels and restaurants, for instance—draw much water; so do industries such as breweries, canneries, laundries, paper mills, and steel mills. Industries, in particular, draw larger volumes of water when it is cheap than when it is dear. Industrial draft varies roughly inversely as the manufacturing rate and is likely to drop by about half the percentage increase in cost when rates are raised. Hospitals, too, have high demands. Although the rate of draft in firefighting is high, the time and annual volume of water consumed in extinguishing fires are small and seldom identified separately for this reason.

Water of poor quality may drive consumers to resort to uncontrolled, sometimes dangerous, sources, but the public supply remains the preferred source when the product water is clean, palatable, and of unquestioned safety; soft for washing and cool for drinking; and generally useful to industry. The availability of groundwater and nearby surface sources may persuade large industries and commercial enterprises to develop their own process and cooling waters.

Hydraulically, leaks from mains and plumbing systems and flows from faucets and other regulated openings behave like orifices. Their rate of flow varies as the square root of the pressure head, and high distribution pressures raise the rate of discharge and with it the waste of water from fixtures and leaks. Ordinarily, systems pressures are not raised above 60 psig (lb/in.^{2} gauge), or 416 kPa, in American practice, even though it is impossible to employ direct hydrant streams in firefighting when hydrant pressures are below 75 psig (520 kPa).

Metering encourages thrift and normalizes the demand. The costs of metering and the running expense of reading and repairing meters, however, are substantial. They may be justified in part by accompanying reductions in waste and possible postponement of otherwise needed extensions. Under study and on trial here and there is the encouragement of off-peak-hour draft of water by large users. To this purpose, rates charged for water drawn during off-peak hours are lowered preferentially. The objective is to reap the economic benefits of a relatively steady flow of water within the system and the resulting proportionately reduced capacity requirements of systems components. The water drawn during off-peak hours is generally stored by the user at ground level even when this entails repumping.

Distribution networks are seldom perfectly tight. Mains (see Fig. 4.3), valves, hydrants, and services of well-managed


Figure 4.3 Water leakage from a crack in a water pipe.

systems are therefore regularly checked for leaks. Superficial signs of controllable leakage are

1. High night flows in mains
2. Water running in street gutters
3. Moist pavements
4. Persistent seepage
5. Excessive flows in sewers
6. Abnormal pressure drops
7. Unusually green vegetation (in dry climates)

Leakage is detected by (a) driving rods into the ground to test for moist earth, (b) using a sounding method in which listening devices amplify the sound of running water, (c) conducting a leak noise correlation that is based on a sonic technique, (d) tracing leaks by injecting a gas into the network and detecting its escape by a suitable instrument to determine the location of the leak, and (e) inspecting premises for leaky plumbing and fixtures. Leakage detection of well-managed waterworks may be complemented by periodic and intensive but, preferably, routine and extensive water-waste surveys. Generally involved is the isolation of comparatively small sections of the distribution system by closing valves on most or all feeder mains and measuring the water entering the section at night through one or more open valves or added piping on fire hoses. Common means of measurement are pitot tubes, bypass meters around controlling valves, or meters on one or more hose lines between hydrants that straddle closed valves. Table 4.7 summarizes reported water losses ranging from 2% to 50% in several cities including developed as well as developing countries.

Table 4.7 Water losses in various cities, 1980–1990

City	Water loss (%)
Toronto, Canada	18
Munich, Germany	12
Hamburg, Germany	2
Hong Kong, China	30
Bombay, India	33
Delhi, India	18
Deagu, Korea	37
Manila, Philippines	51
Riyadh, Saudi Arabia	30
Colombo, Sri Lanka	30
Stockholm, Sweden	19
Kalmar, Sweden	5
Bangkok, Thailand	49
United Kingdom	24
Boston, MA, USA	33
Cambridge, MA, USA	18.5
Springfield, IL, USA	25
Westchester, NY, USA	16
46 communities in Massachusetts, USA	2–50

4.3.3 Industrial Water Consumption

The amounts of water used by industry vary widely. Some industries draw in excess of 50 MGD (190 MLD); others, no more than comparably sized mercantile establishments. On average, US industry satisfies more than 60% of its water requirements by internal reuse and less than 40% by draft through plant intakes from its own water sources or through service connections from public water systems. Only about 7% of the water taken in is consumed; 93% is returned to open waterways or to the ground, whence it may be removed again by downstream users. On balance, industry's consumptive use is kept down to 2% of the draft of all water users in the United States. Table 4.8 shows the relative amounts of water consumed by different industries. Not brought out is the fact that once-through cooling, particularly by the power industry, is by far the biggest use component and the principal contributor to the thermal pollution of receiving waters.

To draw comparisons between the water uses of different industries and of plants within the same industrial category, it is customary to express plant or process use in volumes of water—gallons, for instance—per unit of production (Table 4.9). For the chemical industry, however, this may not be meaningful, because of the diversity of chemicals produced.

Rising water use can be arrested by conserving plant supplies and introducing efficient processes and operations. Most important, perhaps, are the economies of multiple reuse through countercurrent rinsing of products, recirculation of cooling and condensing waters, and reuse of otherwise spent water for secondary purposes after their partial purification or reunification.

About two-thirds of the total water intake of US manufacturing plants is put to use for cooling. In electric-power generation, the proportion is nearly 100%; in manufacturing industries, it ranges from 10% in textile mills to 95% in beet sugar refineries. It averages 66% in industries as reported by the National Association of Manufacturers.

Industry often develops its own supply. Chemical plants, petroleum refineries, and steel mills, for example, draw on

Table 4.8 Water percentage of total water intake consumed by industry

Industry	Percent of intake	Industry	Percent of intake
Automobile	6.2	Meat	3.2
Beet sugar	10.5	Petroleum	7.2
Chemicals	5.9	Poultry processing	5.3
Coal preparation	18.2	Pulp and paper	4.3
Corn and wheat milling	20.6	Salt	27.6
		Soap and detergents	8.5
Distillation	10.4	Steel	7.3
Food processing	33.6	Sugar, cane	15.9
Machinery	21.4	Textiles	6.7

Table 4.9 Water requirements of selected industries

Industry	Unit of production	Gal per unit
Food products		
Beet sugar	Ton of beets	7,000[a]
Beverage alcohol	Proof gal	125–170
Meat	1,000 lb live weight	600–3,500[b]
Vegetables, canned	Case	3–250
Manufactured products		
Automobiles	Vehicle	10,000
Cotton goods	1,000 lb	20,000–100,000
Leather	1,000 ft^2 of hide	200–64,000
Paper	ton	2,000–100,000
Paper pulp	ton	4,000–60,000
Mineral products		
Aluminum (electrolytic smelting)	ton	56,000 (max)
Copper		
Smelting	ton	10,000[c]
Refining	ton	4,000
Fabricating	ton	200–1,000
Petroleum	Barrel of crude oil	800–3000[d]
Steel	ton	1,500–50,000

Conversion factors: 1 gal = 3.785 L; 1 ton = 2000 lb = 0.9072 metric ton; 1 lb = 0.4536 kg; 1 ft^2 = 0.0929 m^2; 1 barrel = 5.615 ft^3 = 0.159 m^3 = 42 gal.

[a] Includes 2,600 gal (9,841 L) of flume water and 2,000 gal (7,570 L) of barometric condenser water.

[b] Lower values for slaughterhouses; higher for slaughtering and packing.

[c] Total, including recycled water; water consumed is 1,400 gal (5,390 L).

[d] Total, including recycled water; water consumed is 30–60 gal (113.55–227.1 L).

public or private utilities for less than 10% of their needs. Food processors, by contrast, purchase about half of their water from public supplies, largely because the bacterial quality of drinking water makes it *de facto* acceptable.

About 90% of the industrial draft is taken from surface sources. Groundwaters may be called into use in the summer because their temperature is then seasonally low. They may be prized, too, for their clarity and their freedom from color, odor, and taste.

Available sources may be drawn on selectively: municipal water for drinking, sanitary purposes, and delicate processes, for example; and river water for rugged processes and cooling and for emergency uses such as fire protection. Treatment costs as well as economic benefits are the determinants.

4.3.4 Rural Water Consumption

The minimum use of piped water in rural dwellings is about 20 gpcd (75 Lpcd); the average about 50 gpcd (190 Lpcd). Approximate drafts of rural schools, overnight camps, and rural factories (exclusive of manufacturing uses) are 25 gpcd (95 Lpcd); of wayside restaurants, 10 gpcd (38 Lpcd) on a patronage basis; and of work or construction camps, 45 gpcd (170 Lpcd). Resort hotels need about 100 gpcd (380 Lpcd), and rural hospitals and the like, nearly twice this amount.

Farm animals have the following approximate requirements: dairy cows, 20 gpcd (75 Lpcd); horses, mules, and steers, 12 gpcd (45 Lpcd); hogs, 4 gpcd (15 Lpcd); sheep, 2 gpcd (8 Lpcd); turkeys, 0.07 gpcd (0.26 Lpcd); and chickens, 0.04 gpcd (0.15 Lpcd). Cleansing and cooling water add about 15 gpcd (57 Lpcd) for cows to the water budget of dairies. Greenhouses may use as much as 70 gpd per 1,000 ft^2 (2.856×10^{-3} m^3/d/m^2) and garden crops about half this amount.

Military requirements vary from an absolute minimum of 0.5 gpcd (1.9 Lpcd) for troops in combat through 2–5 gpcd (7.6–19 Lpcd) for soldiers on the march or in bivouac, and 15 gpcd (57 Lpcd) for temporary camps, up to 50 gpcd (190 Lpcd) or more for permanent military installations.

4.4 VARIATIONS OR PATTERNS OF WATER DEMAND

A pattern is a function relating water use to time of day. Patterns allow the user to apply automatic time-variable changes within the system. Most patterns are based on a multiplication factor versus time relationship, whereby a multiplication factor of 1.0 represents the base value (often the average value, Q_{avg}). This relationship is written as $Q_t = kQ_{\text{avg}}$, where Q_t = demand at time t and k = multiplier for time.

Water consumption changes with the seasons, the days of the week, and the hours of the day. Fluctuations are greater in small rather than large communities and during short rather than long periods of time. Variations are usually expressed as ratios to the average demand. Estimates for the United States are as follows:

Ratio of rates, k	Normal range	Average
Maximum day: average day	(1.5–3.5):1	2.0:1
Maximum hour: average day	(2.0–7.0):1	4.5:1

Where existing water quantity data are not available to accurately determine the instantaneous peak demand for the design year, the following criteria may be used as a minimum for estimating the instantaneous peak demand:

$$\text{For 220 people or less: } Q_{\text{ins peak}} = 9(Q_{\text{avg day}})$$

$$\text{For more than 220 people: } Q_{\text{ins peak}} = 7(Q_{\text{avg day}})\big/P_k^{0.167}$$

where

$Q_{\text{ins peak}}$ = instantaneous peak water demand (gpm or L/min)

$Q_{\text{avg day}}$ = average daily water demand (gpm or L/min)

P_k = design year population (thousands)

Table 4.10 Variations in average daily rates of water drawn by dwelling units

	gpd per dwelling unit and (ratios to average day)		
	Average day	Maximum day	Peak hour
Federal Housing Administration standards	400 (1.0)	800 (2.0)	2,000 (5.0)
Observed drafts—metered dwellings			
National average	400 (1.0)	870 (2.2)	2,120 (5.3)
West	460 (1.0)	980 (2.1)	2,480 (5.2)
East	310 (1.0)	790 (2.5)	1,830 (5.9)
Unmetered dwellings			
National average	690 (1.0)	2,350 (3.4)	5,170 (7.6)
Unsewered dwellings			
National average	250 (1.0)	730 (2.9)	1,840 (7.5)

Conversion factors: 1 gpd per dwelling unit = 1 gpud = 3.785 Lpd per dwelling unit = 3.785 Lpud.

4.4.1 Domestic Variations

Observations in a Johns Hopkins University study and standards of the Federal Housing Administration for domestic drafts are brought together in Table 4.10. Damping effects produced by network size and phasing of commercial, industrial, and domestic drafts explain the differences between community-wide and domestic demands. Observational values for peak hourly demands $Q_{\text{peak h}}$ in gpud for US customary units and Lpud for SI units are given by the regression equation

$$Q_{\text{peak h}} = 334 + 2.02\,Q_{\text{max day}} \quad \text{(US customary units)} \tag{4.21a}$$

$$Q_{\text{peak h}} = 1{,}264 + 2.02\,Q_{\text{max day}} \quad \text{(SI units)} \tag{4.21b}$$

where $Q_{\text{max day}}$ is the maximum daily water demand in gpud for US customary units and Lpud for SI units.

Calculation of the confidence limit of expected demands makes it possible to attach suitably higher limits to these values. For design this may be the 95% confidence limit, which lies above the expected rate of demand by standard deviation, that is, by σ where σ is the standard deviation of the observed demands. Approximate values of this variance are shown in Table 4.11 for gross housing densities of 1, 3, and 10 dwelling units per acre (1 acre = 0.4046 ha) and a potential daily evapotranspiration of 0.28 in. (7.1 mm) of water.

Example: For 200 dwelling units, the maximum water demand for the housing density of 3 dwelling units/acre is $0.4 \times 1{,}000$ gpud = 400 gpud. The peak water demand for the housing density of 3 dwelling units/acre is $0.7 \times 1{,}000$ gpud = 700 gpud.

In another study in Riyadh, Saudi Arabia, Quraishi et al. (1990) reported similar daily variations in water consumption as well as variations in average daily water consumption as a function of type of residence and household income (see Table 4.12).

Rates of rural water use and wastewater production are generally functions of the water requirements and discharge capacities of existing fixtures.

Design demands determine the size of hydraulic components of the system; expected demands determine the rate structure and operation of the system.

Table 4.11 Increase in design demands for dwelling units above expected maximum day and peak hour

Number of dwelling units	Maximum day (1,000 gpud) for stated density in dwelling units per acre housing			Peak hour (1,000 gpud) for stated housing density in dwelling units per acre		
	1	3	10	1	3	10
1	5.0	4.8	4.6	7.0	7.0	7.0
10	1.6	1.5	1.5	3.5	2.3	2.2
10^2	0.7	0.5	0.5	1.5	0.8	0.7
10^3	0.6	0.3	0.2	1.5	0.4	0.3
10^4	0.5	0.2	0.1	1.4	0.3	0.2

Conversion factors: 1,000 gpud = 3,785 Lpud; 1 dwelling unit/acre = 2.4716 dwelling units/ha; 1 acre = 0.4046 ha.

Table 4.12 Variations in per capita domestic water consumption in Riyadh, Saudi Arabia

Description	Consumption (Lpcd)	Ratio to average day
Overall population		
Average day	310	1.00
Maximum day	1,323	4.27
Minimum day	41	0.13
Income level		
Low income (average day)	339	1.09
Medium income (average day)	219	0.71
High income (average day)	432	1.39
House occupants (average day)	343	1.11
Building occupants (average day)	221	0.71

Conversion factor: 1 gpcd = 3.785 Lpcd.

4.4.2 Fire Demands

Height, bulk, area, congestion, fire resistance, type of construction, and building occupancy determine the rate at which water should be made available at neighboring hydrants, either as hydrant or engine streams, to extinguish localized fires and prevent their spread into areal or citywide conflagrations. The needed fire flow (NFF) is defined as the water flow rate, measured at a residual pressure of 20 psi (138 kPa) and for a given duration, which is required for fighting a fire in a specific building. Analysis of water demands actually experienced during fires in communities of different sizes underlies the formulation of the general standards. The American Water Works Association (1998) in its M31 Manual describes three methods for calculating the fire flow requirements that were developed by these organizations:

1. Insurance Services Office Inc. (ISO)
2. Illinois Institute of Technology Research Institute (IITRI)
3. Iowa State University (ISU)

The following general information is taken from the 2006 edition of the ISO's *Guide for Determination of Needed Fire Flow*:

1. For one- and two-family dwellings not exceeding two stories in height, the following needed fire flows should be used:

Distance between buildings, ft (m)	Fire flow, gpm (L/min)
Over 100 (over 30)	500 (1,893)
31–100 (9.1–30)	750 (2,839)
11–30 (3.1–9)	1,000 (3,785)
10 or less (3 or less)	1,500 (5,678)

2. For wood-shingle roof coverings on the building or on exposed buildings, add 500 gpm (1,893 Lpm) to the above flows.
3. High-risk areas. Multifamily, commercial, and industrial areas are considered high-risk areas. The fire flows available for these areas require special consideration. The distribution and arterial mains in the high-risk areas are to accommodate the required fire flows in those areas.
4. Typically the water main system must be able to meet the flow requirements of (a) peak day demands plus fire flow demands as a minimum for any water main design and (b) instantaneous peak demands for special water mains from water source, water treatment plant, and/or water storage facilities.
5. Water storage facilities should have sufficient capacity, as required by the *Recommended Standards for Water Works* (2007 edition), to meet domestic average daily water demands and, where fire protection is provided, fire flow demands.

Fire demand can also be estimated according to the community size and realty subdivision development by using the following empirical equation from the National Board of Fire Underwriters:

$$Q_{\text{fire}} = 1{,}020(P_k)^{0.5}[1 - 0.01(P_k)^{0.5}] \quad \text{(US customary units)} \tag{4.22a}$$

$$Q_{\text{fire}} = 3{,}860.7(P_k)^{0.5}[1 - 0.01(P_k)^{0.5}] \quad \text{(SI units)} \tag{4.22b}$$

where Q_{fire} is the fire demand, in gpm for US customary units and L/min for SI units, and P_{k} is the population, in thousands.

The National Board of Fire Underwriters requires provision for a 5-hour fire flow in places with populations of less than 2,500 and provision for a 10-hour flow in larger places.

The ISU method is relatively simple and quick to use but yields low fire flow requirements, whereas the IITRI method produces excessively high rates. The ISO methodology calculates values in between the other two. The most recent *International Fire Code* (International Code Council [ICC], 2006) recommends a minimum fire flow of 1,000 gpm (3,785 L/min) for one- and two-family dwellings having an area that does not exceed 3,600 ft^2 (344 m^2). Fire flow and flow durations for larger buildings having areas in excess of 3,600 ft^2 (344 m^2) are not to be less than what is shown in Table 4.13. Type I-A structures are typically concrete-frame buildings made of noncombustible materials. All of the building elements—structural frame, bearing walls, floors, and roofs—are fire resistance rated. Type V-B construction is typically wood-frame construction, which is very common because it does not require a fire rating.

The benefits of early fire suppression are acknowledged by the acceptance that the firefighting water requirement can be reduced by 50% for one- and two-family dwellings and

Table 4.13 Required fire flow for buildings

Area[a] ft^2 (m^2) Type of building construction			
Type I-A[b] noncombustible, protected	Type V-B[c] combustible, unprotected	Fire flow gpm (L/min)	Duration (h)
0–22,700 (0–2,109)	0–3,600 (0–334)	1,500 (5,678)	2
22,701–30,200 (2,110–2,806)	3,601–4,800 (335–446)	1,750 (6,624)	
30,201–38,700 (2,807–3,595)	4,801–6,200 (447–576)	2,000 (7,520)	
38,701–48,300 (3,596–4,487)	6,201–7,700 (577–715)	2,250 (8,516)	
48,301–59,000 (4,488–5,481)	7,701–9,400 (716–873)	2,500 (9,463)	
59,001–70,900 (5,482–6,587)	9,401–11,300 (874–1,050)	2,750 (10,409)	
70,901–83,700 (6,588–7,776)	11,301–13,400 (1,051–1,245)	3,000 (11,355)	3
83,701–97,700 (7,777–9,076)	13,401–15,600 (1,246–1,449)	3,250 (12,301)	
97,701–112,700 (9,077–10,470)	15,601–18,000 (1,450–1,672)	3,500 (13,248)	
112,701–128,700 (10,471–11,956)	18,001–20,600 (1,673–1,913)	3,750 (14,144)	
128,701–145,900 (11,957–13,554)	20,601–23,300 (1,914–2,165)	4,000 (15,140)	4
145,901–164,200 (13,555–15,254)	23,301–26,300 (2,166–2,443)	4,250 (16,086)	
164,201–183,400 (15,255–17,038)	26,301–29,300 (2,444–2,722)	4,500 (17,033)	
183,401–203,700 (17039–18,927)	29,301–32,600 (2,723–3,029)	4,750 (17,979)	
203,701–225,200 (18,928–20,921)	32,601–36,000 (3,030–3,344)	5,000 (18,925)	
225,201–247,700 (20,922–23,011)	36,001–39,600 (3,345–3,679)	5,250 (19,871)	
247,701–271,200 (23,012–25,194)	39,601–43,400 (3,680–4,032)	5,500 (20,818)	
271,201–295,900 (25,195–27,489)	43,401–47,400 (4,033–4,403)	5,750 (21,764)	
295,901–greater (27,490–greater)	47,401–51,500 (4,404–4,784)	6,000 (22,710)	
— (—)	51,501–55,700 (4,785–5,175)	6,250 (23,656)	
— (—)	55,701–60,200 (5,176–5,593)	6,500 (24,603)	
— (—)	60,201–64,800 (5,594–6,020)	6,750 (25,549)	
— (—)	64,801–69,600 (6,021–6,466)	7,000 (26,495)	
— (—)	69,601–74,600 (6,467–6,930)	7,250 (27,441)	
— (—)	74,601–79,800 (6,931–7,413)	7,500 (28,388)	
— (—)	79,801–85,100 (7,414–7,906)	7,750 (29,334)	
— (—)	85,101–greater (7,907–greater)	8,000 (30,280)	

Source: Data from International Fire Code (ICC, 2006).

Conversion factors: 1 ft^2 = 0.0929 m^2, 1 gpm = 3.785 L/min.

[a] The specified area shall be the area of the three largest successive floors.

[b] Typically these are concrete-frame buildings made of noncombustible materials. "A" stands for protected.

[c] Typically these are wood-frame buildings. "B" stands for unprotected.

by up to 75% for larger buildings (exceeding 3600 ft^2 or 344 m^2) when buildings are fitted with automatic sprinkler systems.

Standard fire requirements take into account probable loss of water from connections broken in the excitement of a serious fire. Coincident draft of water for purposes other than firefighting is rarely assumed to equal the maximum hourly rate. Depending on local conditions, the maximum daily rate may be a reasonably safe assumption instead.

EXAMPLE 4.2 DETERMINATION OF CAPACITIES OF WATERWORKS SYSTEMS

The four typical waterworks systems shown in Fig. 4.4 supply a community with an estimated future population of 120,000. Determine the required capacities of the constituent structures for an average consumption of 150 gpcd (568 Lpcd) and a distributing reservoir so sized that it can balance out differences between hourly and daily flows, fire demands, and emergency requirements. Assume a requirement for fire protection that is needed to control two simultaneous fires in two unprotected wood-frame construction, multistory buildings having a 12,000 ft^2 (1,115 m^2) area per floor.

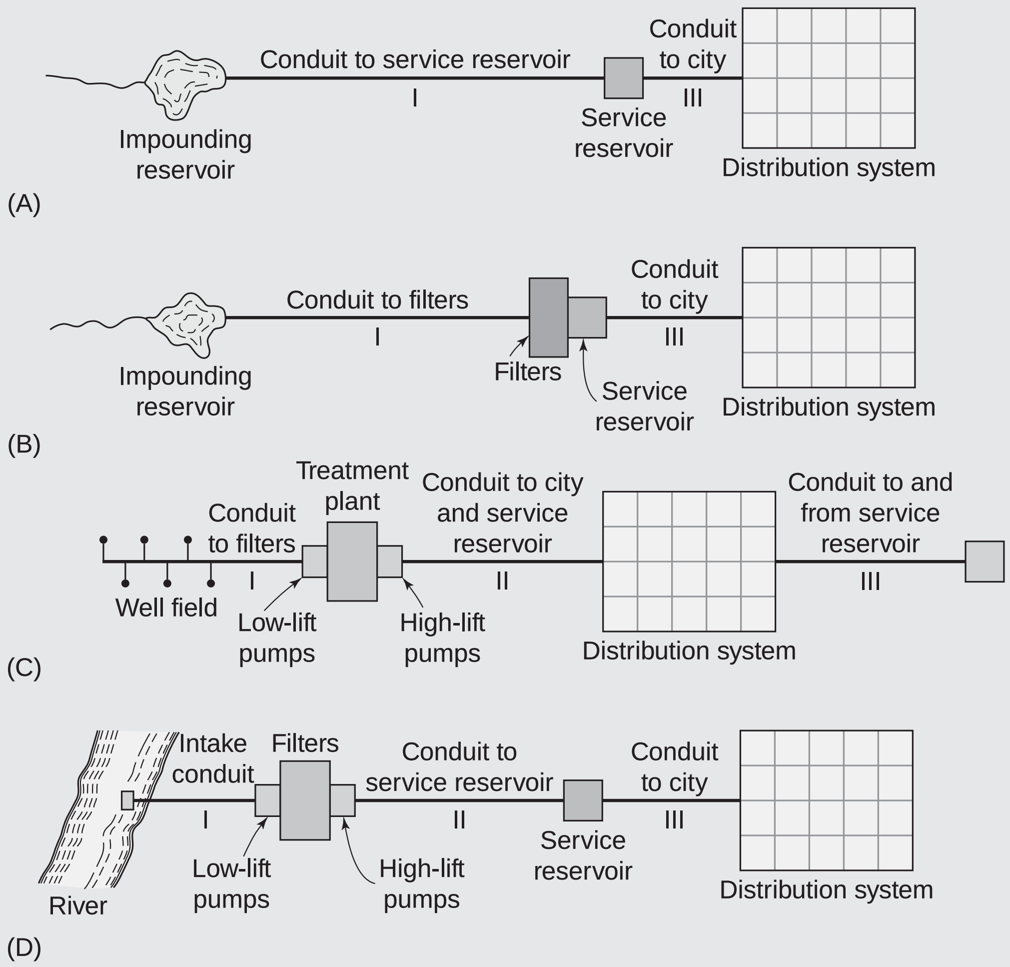

Figure 4.4 Required capacities of four typical waterworks systems. The service reservoir is assumed to compensate for fluctuations in draft and fire drafts and to hold an emergency reserve.

Solution 1 (US Customary System):

Required capacities for waterworks systems of Fig. 4.4:

Average daily draft = $150 \times 1.2 \times 10^5/10^6 = 18$ MGD.
Maximum daily draft = coincident draft = $2 \times 18 = 36$ MGD.
Maximum hourly draft = $4.5 \times 18 = 81$ MGD.

Fire flow from Table 4.13:

Area of three floors = $3 \times 12{,}000$ ft^2/floor = 36,000 ft^2.
Flow for one fire = 5,000 gpm.
Flow for two simultaneous fires = $2 \times 5{,}000 = 10{,}000$ gpm = 14.4 MGD.
Coincident draft plus fire flow = $36 + 14.4 = 50.4$ MGD.

Provision for breakdowns and repair of pumps and water purification units by installing one reserve unit, as shown in Table 4.14, assuming total units = $3 + 1 = 4$:

Low-lift pumps: $4/3 \times$ maximum daily draft = $(4/3) \times 36 = 48$ MGD.
High-lift pumps: $4/3 \times$ maximum hourly draft = $(4/3) \times 81 = 108$ MGD.
Treatment works: $4/3 \times$ maximum daily draft = 48 MGD.

The resultant capacities of systems components are summarized in Table 4.14.

Table 4.14 Capacities of the four systems of Example 4.2

		Capacity of system (MGD)			
Structure	Required capacity	A	B	C	D
1. River or well field	Maximum day			36.0	36.0
2. Conduit I	Maximum day	36.0	36.0	36.0	36.0
3. Conduit II	Maximum day	…	…	81.0	36.0
4. Conduit III	Maximum hour	81.0	81.0	81.0	81.0
5. Low-lift pumps	Maximum day plus reserve	…	…	48.0	48.0
6. High-lift pumps	Maximum hour plus reserve	…	…	108.0	108.0
7. Treatment plant	Maximum day plus reserve	…	48.0	48.0	48.0
8. Distribution system high-value district	Maximum hour	81.0	81.0	81.0	81.0

Conversion factor: 1 MGD = 3.785 MLD.

Solution 2 (SI System):

Required capacities for waterworks systems of Fig. 4.4:

Average daily draft $Q_{\text{avg day}} = (568 \text{ Lpcd})(120{,}000)/10^6 = 68.2$ MLD.

Maximum daily draft $Q_{\text{max day}} = \text{coincident draft} = 2 \times Q_{\text{avg day}} = 2 \times 68.2 = 136.4$ MLD.

MGD maximum hourly draft $Q_{\text{max h}} = 4.5 \times Q_{\text{avg day}} = 4.5 \times 68.2 = 306.9$ MLD.

Fire flow from Table 4.13:

Area of three floors = $3 \times 12{,}000 \text{ ft}^2/\text{floor} = 36{,}000 \text{ ft}^2 = 3{,}344.4 \text{ m}^2$.

Flow for one fire = 5,000 gpm = 18,925 Lpm.

Flow for two simultaneous fires = $2 \times 18{,}925 = 37{,}850$ Lpm = 54.5 MLD.

Coincident draft plus fire flow = $136.4 + 54.5 = 190.9$ MLD.

Provision for breakdowns and repair of pumps and water purification units by installing one reserve unit, as shown in Table 4.14 assuming total units = $3 + 1 = 4$:

Low-lift pumps: $4/3 \times$ maximum daily draft = $(4/3) \times 136.4 = 181.9$ MLD.

High-lift pumps: $4/3 \times$ maximum hourly draft = $(4/3) \times 306.9 = 409.2$ MLD.

Treatment works: $4/3 \times$ maximum daily draft = 181.9 MLD.

The resultant capacities of systems components are summarized in Table 4.14.

EXAMPLE 4.3 ESTIMATION OF WATER DEMAND TO A RESIDENTIAL AREA

For management and design purposes, find the expected maximum day and peak hour demand and daily rates of water to be supplied to a residential area of 200 houses with a gross housing density of three dwellings per acre (7.415 unit/ha), an average market value of USD 125,000 per dwelling unit, and a potential evaporation of 0.28 in./d (7.11 mm/d) on the maximum day. Assume the site/time factor F is equal to 1.0.

Solution 1 (US Customary System):

Average maximum and peak daily domestic demands per dwelling unit:

By Eq. (4.20a) $Q_{\text{domestic}} = 157 + 3.46\, MF = 157 + 3.46 \times 125 \times 1.0 = 590$ gpud.

By Eq. (4.19a): $A = 0.803\, D^{-1.26} = 0.803/3^{1.26} = 0.20$ acre per dwelling unit.

By Eq. (4.18a): $Q_{\text{sprinkling}} = 1.63 \times 10^4\, A\, (E - P) = 1.63 \times 10^4 \times 0.20 \times (0.28 - 0)$

$= 913$ gpud excluding precipitation.

For management:

$$Q_{\text{max day}} = Q_{\text{domestic}} + Q_{\text{sprinkling}} = 590 + 913 = 1{,}503 \text{ gpud.}$$

By Eq. (4.21a): $Q_{\text{peak h}} = 334 + 2.02Q_{\text{max day}} = 334 + 2.02 \times 1{,}503 = 3{,}640$ gpud.

For design based on 200 dwellings:

From Table 4.11: $Q_{\text{max day}} = 1{,}503 + 0.4(1{,}000) = 1{,}903$ gpud.

From Table 4.11: $Q_{\text{peak h}} = 3{,}640 + 0.7(1{,}000) = 4{,}340$ gpud.

Solution 2 (SI System):

Average maximum and peak daily domestic demands per dwelling unit:

By Eq. (4.20b): $Q_{\text{domestic}} = 594 + 13.1\, MF = 594 + 13.1 \times 125 \times 1.0 = 2{,}232$ Lpud.

By Eq. (4.19b): $A = 1.016\, D^{-1.26} = 1.016/(7.415)^{1.26} = 1.016/12.4836 = 0.08138$ ha/unit.

By Eq. (4.18b): $Q_{\text{sprinkling}} = 6{,}003\, A(E - P) = 6{,}003 \times 0.08138 \times (7.11 - 0)$

$= 3{,}473$ Lpud excluding precipitation.

For management:

$$Q_{\text{max day}} = Q_{\text{domestic}} + Q_{\text{sprinkling}} = 2{,}232 + 3{,}473 = 5{,}705 \text{ Lpud.}$$

By Eq. (4.21b): $Q_{\text{peak h}} = 1{,}264 + 2.02\, Q_{\text{max day}} = 1{,}264 + 2.02 \times 5{,}705.2 = 12{,}789$ Lpud.

For design based on 200 dwellings:

From Table 4.11: $Q_{\text{max day}} = 5{,}705 + 0.4\,(1{,}000) \times 3.785 = 7{,}219$ Lpud.

From Table 4.11: $Q_{\text{peak h}} = 12{,}789 + 0.7\,(1{,}000) \times 3.785 = 15{,}438$ Lpud.

EXAMPLE 4.4 DETERMINATION OF FIRE FLOW AND DURATION FOR BUILDINGS

A four-story wooden-frame building (combustible, unprotected) has each floor of area 5,490 ft^2 (510 m^2). This wooden building is adjacent to a five-story building of commercial construction (noncombustible, protected) with 9,800 ft^2 (910 m^2) per floor. Determine the fire flow and duration (a) for four-story building; (b) for five-story building; and (c) for both buildings assuming they are connected together.

Solution 1 (US Customary System):

a. Area occupied by the four-story building = 4 (5,490) = 21,960 ft^2.

From Table 4.13, column Type V-B,
Fire flow = 4,000 gpm and duration = 4 h.

b. Area occupied by the five-story building = 5 (9,800) = 49,000 ft^2.

From Table 4.13, column Type I-A,
Fire flow = 2,500 gpm and duration = 2 h.

c. Total area of two buildings = 21,960 + 49,000 = 70,960 ft^2.

Fractional area of the four-story building = 21,960/70,960 = 0.3095.
Fractional area of the five-story building = 49,000/70,960 = 0.6905.

From Table 4.13, column Type I-A, for 70,960 ft^2,

Fire flow = 3000 gpm and duration = 3 h.

From Table 4.13, column Type I-B, for 70,960 ft^2,

Fire flow = 7,250 gpm and duration = 4 h.

Combined fire flow for the two buildings connected together

$= 0.3095 \times 3{,}000 + 0.6905 \times 7{,}250 = 5{,}934$ gpm

Select duration = 4 h.

Solution 2 (SI System):

a. Area occupied by the four-story building = 4 (510) = 2,040 m^2 = 21,960 ft^2

From Table 4.13, column Type V-B,

Fire flow = 4,000 gpm = 15.14 m^3/min and duration = 4 h.

b. Area occupied by the five-story building = 5 (910) = 4,550 m^2 = 49,000 ft^2

From Table 4.13, column Type I-A,

Fire flow = 2,500 gpm = 9.46 m^3/min and duration = 2 h.

c. Total area of two buildings = 2,040 + 4,550 = 6,590 m^2 = 70,960 ft^2.

Fractional area of the four-story building = 2,040/6,590 = 0.3095.

Fractional area of the five-story building = 4,550/6,590 = 0.6905.

From Table 4.13, column Type I-A, for 70,960 ft^2,

Fire flow = 3,000 gpm and duration = 3 h.

From Table 4.13, column Type I-B, for 70,960 ft^2,

Fire flow = 7,250 gpm and duration = 4 h.

Combined fire flow for the two buildings connected together

$0.3095 \times 3{,}000 + 0.6905 \times 7{,}250 = 5{,}934$ gpm = 22.46 m^3/min.

Select duration = 4 h.

EXAMPLE 4.5 DETERMINATION OF FIRE FLOW BASED ON COMMUNITY POPULATION

Estimate the fire flow requirement of a community with a population of 121,000.

Solution 1 (US Customary System):

$$\begin{aligned} Q_{\text{fire}} &= 1{,}020(P_k)^{0.5}[1 - 0.01(P_k)^{0.5}] \\ &= 1{,}020(121)^{0.5}[1 - 0.01(121)^{0.5}] \\ &= \mathbf{9{,}986\ gpm}. \end{aligned}$$

Solution 2 (SI System):

$$\begin{aligned} Q_{\text{fire}} &= 3{,}860.7(P_k)^{0.5}[1 - 0.01(P_k)0.5] \\ &= 3{,}860.7(121)^{0.5}[1 - 0.01(121)^{0.5}] \\ &= \mathbf{37{,}796.25\ L/min(or 37.796\ m^3/min)}. \end{aligned}$$

EXAMPLE 4.6 ESTIMATION OF THE RATIOS OF MAXIMUM, WEEK, DAY, AND HOUR DEMAND IN THE ABSENCE OF ACTUAL WATER DEMAND DATA

Determine the "percent of annual average water demand" for (a) daily average in maximum month; (b) daily average in maximum week; (c) maximum day in a year; and (d) peak hour within a day assuming the peak hour demand is distributed over 2 hours, all using the following equation:

$$P = 180\,(t)^{-0.1} \tag{4.23}$$

where P = percent of the annual average day water demand for time t and t = time in days, which varies from 2/24 to 365.

Solution:

a. For the daily average in maximum month, $t = 30$ days.

$P = 180(t)^{-0.1} = 180(30)^{-0.1} = 180 \times 0.7117 = 128.$

Daily average water demand in maximum month = 1.28 $Q_{\text{ave-day}}$.

b. For the daily average in maximum week, $t = 7$ days.

$P = 180(t)^{-0.1} = 180(7)^{-0.1} = 180 \times 0.8232 = 148.$

Daily average water demand in maximum week = 1.48 $Q_{\text{ave-day}}$.

c. For the maximum day in a year, $t = 1$ day.

$P = 180(t)^{-0.1} = 180(1)^{-0.1} = 180 \times 1 = 180.$

Maximum day water demand in a year = 1.8 $Q_{\text{ave-day}}$.

d. For the peak hour within a day assuming the peak hour demand is distributed over 2 hours, $t = 2/24$ days.

$P = 180(t)^{-0.1} = 180(2/24)^{-0.1} = 180 \times 1.2821 = 231.$

Peak hour demand within a day = 2.31 $Q_{\text{ave-day}}$.

4.5 DEMAND AND DRAINAGE LOADS OF BUILDINGS

The demand load of a building depends on the number and kinds of fixtures and the probability of their simultaneous operation. Different fixtures are furnished with water at different rates as a matter of convenience and purpose. Expressed in cubic feet per minute (ft^3/min), or liter per second (L/s) for fixture units, these rates become whole numbers of small size. Common demand rates are shown in Table 4.15.

It is quite unlikely that all fixtures in a building system will draw water or discharge it at the same time. A probability study of draft demands leads to the relationships plotted in Fig. 4.5. In practice, the values shown are modified as follows:

1. Demands for service sinks are ignored in calculating the total fixture demand.
2. Demands of supply outlets, such as sill cocks, hose connections, and air conditioners, through which water flows more or less continuously for a considerable length of time, are added to the probable flow rather than the fixture demand.
3. Fixtures supplied with both hot and cold water exert reduced demands on main hot-water and cold-water branches (not fixture branches). An allowance of three-fourths of the demand shown in Table 4.15 for individual fixtures is suggested.

Table 4.15 Fixture rates

	Rates of supply and discharge ft^3/min (L/min)	
Type of fixture and supply (1) and discharge (2)	Private buildings	Public and office buildings
Wash basin, faucet—(1) and (2)	1 (28)	2 (57)
Water closet, (2) only	6 (170)	8 (227)
Flush tank, (1) only	3 (85)	5 (142)
Flush valve, (1) only	6 (170)	10 (283)
Urinal		
(Stall or wall), flush tank (1) only	… (…)	3 (85)
(Stall or wall), flush valve	… (…)	5 (142)
(Pedestal), flush valve	… (…)	10 (283)
Bathtub or shower, faucet or mixing valve	2 (57)	4 (113)
Bathroom group (2) only	8 (227)	… (…)
Flush tank for closet, (1) only	6 (170)	… (…)
Flush valve for closet, (1) only	8 (227)	… (…)
Separate shower head, (1) only	2 (57)	… (…)
Separate shower stall, (2) only	10 (283)	… (…)
Kitchen sink, faucet 2	2 (57)	4 (113)
Laundry trays (1–3), faucet	3 (85)	… (…)
Combination fixture, faucet	3 (85)	… (…)
Service sink, faucet (hotel or restaurant)	… (…)	3 (85)
Sill cock	2/3 (19)	2/3 (19)

Conversion factor: 1 ft^3/min = 28.32 L/min = 0.02832 m^3/min.

EXAMPLE 4.7 FINDING THE DEMAND LOAD

A two-story dwelling contains the following fixtures: a pair of bathroom groups, an additional water closet, an additional washbasin, a kitchen sink, a laundry tray, and a sill cock. All water closets are served by flush tanks. Find the demand load.

Solution:

From Table 4.15: 2 bathroom groups at 6 = 12; 1 water closet = 3; 1 washbasin = 1; 1 kitchen sink = 2; 1 laundry tray = 3; total = **21 ft^3/min, or 0.5943 m^3/min.**

From Fig. 4.5, the probable flow in the building main will be 15 gpm plus [(2/3) ft^3/min = (2/3) × 7.48 = 5 gpm] for the sill cock, or a **total of 20 gpm, or 75.7 L/min.**

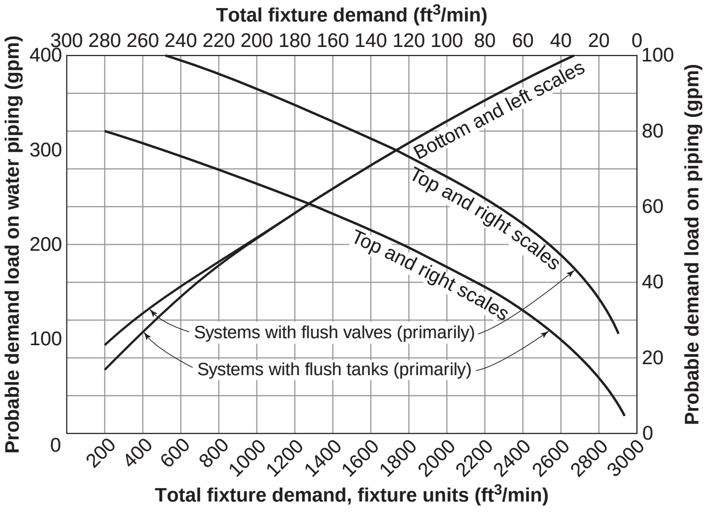

Figure 4.5 Demand load on water piping. Conversion factor: 1 ft^3/min = 7.48 gpm = 0.0283 m^3/min. 1 gpm = 3.785 L/min.

PROBLEMS/QUESTIONS

4.1 As shown in Table 4.3, the April census population of Detroit, MI, was 1,028,000 in 1990 and decreased to 951,000 in 2000. Estimate the population for the following periods:

1. For the fifth intercensal year in April using (a) arithmetic and (b) geometric progression.
2. For the sixth postcensal year in April using (a) arithmetic and (b) geometric progression.

4.2 As shown in Table 4.3, the census population of Providence, RI, was 161,000 in 1990 and 174,000 in 2000. Estimate the 2009 population by (a) arithmetic and (b) geometric progression.

4.3 As shown in Table 4.3, the city of Miami recorded a population of 111,000 in its 1930 and 172,000 in its 1940 consecutive censuses. Estimate the midyear (July 1) populations for the following periods:

1. For the fifth intercensal year by (a) arithmetic increase and (b) geometric increase.
2. For the ninth postcensal year by (a) arithmetic increase and (b) geometric increase.

Assume a census date of April 1.

4.4 Each of the four waterworks systems shown in Fig. 4.4 serves a community with an estimated future population of 100,000. Estimate the required capacities of their constituent structures for an average water consumption of 150 gpcd (568 Lpcd), a fire demand of 9,160 gpm (34,671 L/min), and a distributing reservoir so sized that it can provide enough water to care for differences between hourly and daily flows and for fire demands and emergency water requirements.

4.5 An old town in an established agricultural area with no prospects for extensive development is located along a river that is used as the source of water supply for the community. The conditions of the river are such that it receives its water from a drainage area of 1,000 mi^2 (2,590 km^2) and its low-water flow is 0.1 $\text{ft}^3\text{/s/mi}^2$ (0.00109 $\text{m}^3\text{/s/km}^2$). The regulations are that only 10% of the river flow can be used at any time for water supply.

Past records reveal that the population increased as follows:

Year	Population
1940	10,000
1950	12,500
1960	15,200
1970	17,500
1980	19,700
1990	22,000
2000	24,400

If you are asked to design a water supply system for this community,

1. What would be the expected population in the year 2040? Justify your method of choice for estimating the future population. (Give the figure to the nearest 1,000.)
2. Do you think that the amount of water available in the river would be enough of a water supply and how much water can be supplied to each person in the community?
3. Assuming that you decide to supply the community at an annual average demand of 70 gpcd, or 264 Lpcd, and that the maximum daily demand is two times the annual average and the maximum hourly demand is 1.5 times the daily demand, would you be able to supply this demand directly from the river? If not, what would you recommend in order to be able to supply the peak demand?

4.6 A small village is located near a stream that is to be used as the source of water supply for the community. The minimum flow in the stream is 550 $\text{m}^3\text{/h}$. The population analysis reveals that the population is expected to increase linearly from a population of 10,000 in 2008 to 30,000 in 2058.

The water consumption of the community is estimated to reach an annual average demand of 300 Lpcd, or 79.26 gpcd, a maximum daily demand of 1.8 times the annual average, and peak hourly demand of 1.5 times the daily average.

Would you be able to supply these demands directly from the stream? If not, what would you recommend (1) in the year 2030 and (2) in the year 2058?

4.7 A well that is the source of water supply for a community has a uniform production capacity of 350 $\text{m}^3\text{/h}$ (79.52 gpm). Currently the community has a population of 5,000 people with an anticipated linear growth rate of 10% per year.

If the average water consumption for the community is 400 Lpcd (105.68 gpcd), would you be able to supply the water demand (a) 5 years, (b) 10 years, and (c) 20 years from now, directly from this well? If not, what would you recommend for each of the above periods? Show all of your calculations and explain the reasons behind your assumptions and recommendations.

4.8 Estimate the number of people who can be supplied with water from (a) 12 in. (304.8 mm) and (b) 24 in. (609.6 mm) water main (1) in the absence of fire service for a maximum draft of 200 gpcd (757 Lpcd) and (2) with a residential fire flow requirement of 500 gpm (1,892.5 Lpm) and a coincident draft of 150 gpcd (567.75 Lpcd). Also, find the hydraulic gradient.

Note: Consider the most economical design velocity in the system piping to be 3 ft/s (0.91 m/s).

4.9 Estimate the number of people who can be supplied with water from (a) a 300 mm water main and (b) a 600 mm water main.

Case 1: In the absence of fire service for a maximum draft of 800 Lpcd.

Case 2: With a residential fire flow requirement of 1,890 Lpm and a simultaneous domestic draft of 600 Lpcd.

Note: Consider the most economical design velocity in the system piping to be 1 m/s.

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Chapter 5

Water Hydraulics, Transmission, and Appurtenances

5.1 FLUID MECHANICS, HYDRAULICS, AND WATER TRANSMISSION

5.1.1 Fluid Mechanics and Hydraulics

Fluids are substances that are capable of flowing and that conform to the shape of the vessel in which they are contained. Fluids may be divided into liquids and gases. Liquids are practically incompressible, whereas gases are compressible. Liquids occupy definite volumes and have free surfaces, whereas gases expand until they occupy all portions of any containing vessels. Water, wastewater, oil, and so on are liquids.

Fluid mechanics is a branch of applied mechanics dealing with the behavior of fluids (liquids and/or gases) at rest and in motion. The principles of thermodynamics must be included when an appreciable compressibility does occur. Hydraulics is a branch of fluid mechanics dealing with the behavior of particularly incompressible water, wastewater, liquid sludge, and so on.

The characteristics of gases are determined by Boyle's and Charles's laws:

$$P_a V_s / T_a = R_g \tag{5.1}$$

where P_a = absolute pressure = $P_{gauge} + P_{atm}$, lb/ft^2; V_s = $1/\gamma$ = specific volume, ft^3/lb; T_a = absolute temperature = $T + 460$°F; R_g = gas constant, ft/°F;T = temperature, °F; γ = specific weight, lb/ft^3; and

$$\gamma = 1/V_s = P_a / R_g T_a \tag{5.2}$$

The specific weight, γ, of a substance is the weight of a unit volume of the substance. The specific (unit) weight of water is 62.4 lb/ft^3 for ordinary temperature variations.

The specific gravity of a substance is that pure number, which denotes the ratio of the weight of a substance to the weight of an equal volume of a substance taken as a standard. Solids and liquids are referred to water (at 39.2°F = 4°C) as standard, whereas gases are referred to air free of hydrogen and carbon dioxide (at 32°F = 0°C and at 1 atmosphere = 14.7 lb/in.^{2} pressure) as standard. For example, if the specific gravity of a given oil is 0.755, its specific weight is $0.755 \times$ (62.4 lb/ft^3) = 47.11 lb/ft^3.

The mass of a body ρ (rho), which is termed density, can be expressed by the following equation:

$$\rho = \frac{\gamma}{g} \tag{5.3}$$

In the US customary system of units, the density of water is (62.4 lb/ft^3)/(32.2 ft/s^2) = 1.94 slug/ft^3. In the metric system, the density of water is 1 g/cm^3 = 1 kg/L = 1,000 kg/m^3 = 1 tonne/m^3 at 4°C.

Fluid flow may be steady or unsteady, uniform or nonuniform. Steady flow occurs at any point if the velocity of successive fluid particles is the same at successive instants, that is, if the fluid velocity is constant with time. The uniform flow occurs when the velocity of successive fluid particles does not change with distance. This book introduces mainly the fundamentals of the steady and uniform flows involving water and wastewater.

In steady flow, the mass of fluid passing any and all sections in a stream of fluid per unit of time is the same. For incompressible fluids, such as water and wastewater, the following principle of conservation of mass governs:

$$A_1 v_1 = A_2 v_2 = \text{Constant} = Q \tag{5.4}$$

where A_1 and A_2 are, respectively, the cross-sectional areas (ft^2 or m^2) at section 1 and section 2; v_1 and v_2 are, respectively, the average velocity of the stream (ft/s or m/s) at section 1 and section 2; and Q is the flow rate (ft^3/s or m^3/s).

The Bernoulli equation results from application of the principle of conservation of energy to fluid flow, and is written between two points in a hydraulic system:

$$H_A + H_a - H_l - H_e = H_B \tag{5.5a}$$

$$H_A + H_a = H_B + (H_l + H_e) = H_B + h_f \tag{5.5b}$$

$$H_A = P_A/\gamma + v_A^2/2g + Z_A \tag{5.6a}$$

$$H_B = P_B/\gamma + v_B^2/2g + Z_B \tag{5.6b}$$

Water Engineering: Hydraulics, Distribution and Treatment, First Edition. Nazih K. Shammas and Lawrence K. Wang.

where

H_A = energy at section A

H_B = energy at section B

H_a = energy added

H_l = energy lost

H_e = energy extracted. (The units of H_A, H_B, H_a, H_l, and H_e are feet or meters of the fluid.)

P_A, P_B = pressures at sections A and B (lb/ft^2 or kN/m^2 [1,000 Newton/m^2])

γ = specific weight of water (lb/ft^3 or kN/m^3)

g = acceleration due to gravity (32.2 ft/s^2 or 9.81 m/s^2)

V_A, V_B = velocities at sections A and B (ft/s or m/s)

Z_A, Z_B = heights of stream tube above any assumed datum plane at sections A and B (ft or m)

h_f = head loss (ft or m)

It is important to determine the magnitude, direction, and sense of the hydraulic force exerted by fluids in order to design the constraints of the structures satisfactorily. The force P_f (lb or kg) exerted by a fluid on a plane area A (ft^2 or m^2) is equal to the product of the specific weight γ (lb/ft^{3}orkN/m^3) of the liquid, the depth (ft and m) of the center of gravity h_{cg} of the area, and the area A (ft^2 or m^2). The engineering equation is

$$P_f = (\gamma h_{cg})A = I_p A \quad \textbf{(5.7)}$$

where I_p is the intensity of pressure at center of gravity of the area. The line of action of the force passes through the center of pressure, which can be determined by the following equation:

$$y_{cp} = [I_{cg}/y_{cg}A] + y_{cg} \quad \textbf{(5.8)}$$

where

I_{cg} = moment of inertia of the area about its center of gravity axis

y_{cp} = distance of the center of pressure measured along the plane from an axis located at the intersection of the plane and the water surface, extended if necessary

y_{cg} = distance of the center of gravity measured along the plane from an axis located at the intersection of the plane and the liquid surface, extended if necessary

The moment of inertia of the most common rectangular shape (width b and height h) can be calculated by the following equation:

$$\begin{aligned} I_{cg} &= (1/12)Ah^2 \\ &= (1/12)(bh)h^2 \\ &= (1/12)bh^3 \end{aligned} \quad \textbf{(5.9)}$$

EXAMPLE 5.1 EQUATION OF CONTINUITY

For incompressible fluids, such as water, the equation of continuity shown in Eq. (5.4) can be used for all practical purposes. Derive the equations of continuity for compressible fluids, such as air, carbon dioxide, and chlorine, which are frequently used in water and wastewater treatment plants.

Solution:

The following equations of continuity should be used:

$$\rho_1 A_1 v_1 = \rho_2 A_2 v_2 \quad \textbf{(5.4a)}$$

$$\gamma_1 A_1 v_1/g = \gamma_2 A_2 v_2/g \quad \textbf{(5.4b)}$$

$$\gamma_1 A_1 v_1 = \gamma_2 A_2 v_2 \quad \textbf{(5.4c)}$$

where ρ_1, ρ_2, A_1, A_2, v_1, v_2, γ_1, γ_2, and g have been defined in Eqs. (5.2)–(5.4). The above equations of continuity result from the basic principle of conservation of mass. For a steady flow, the mass of fluid passing any and all sections in a stream of fluid per unit of time is the same.

EXAMPLE 5.2 SIZING AIR PIPE DIAMETER

Determine the minimum diameter of an air pipe to carry 0.55 lb/s (0.25 kg/s) of air with a maximum velocity of 19 ft/s (5.8 m/s). The air is at 82°F (27.8°C) and under an absolute pressure of 34.5 lb/in.^{2} (239.4 kPa). Gas constant for air is 53.3 ft/°R (0.287 kJ/kg-K).

Solution 1 (US Customary System):

$P_a = P_{gauge} + P_{atm} = 34.5$ lb/in.^{2} $= 34.5 \times 144$ lb/ft^2.

$R_g = 53.3$ ft/°R.

T_a = absolute temperature = 82 + 460 = 542°R.

$\gamma_a = P_a/[R_g T_a] = (34.5 \times 144 \text{ lb/ft}^2)/[(53.3 \text{ ft/°R})(542\text{°R})] = 0.172 \text{ lb/ft}^3$.

$W = 0.55 \text{ lb/s} = \gamma_a Q = (0.172 \text{ lb/ft}^3)(Q)$.

$Q = 3.19 \text{ ft}^3\text{/s} = Av = A(19 \text{ ft/s})$.

$A = 3.19/19 = 0.168 \text{ ft}^2 = D^2 \times 0.785$.

$\boldsymbol{D} = (0.168/0.785)^{0.5} = \mathbf{0.46\ ft = 5.55\ in}$.

Solution 2 (SI System):

$P_a = 239.4 \text{ kPa} = 239.4 \text{ kN/m}^2$.

$R_g = 0.287 \text{ kJ/kg-K} = 29.26 \text{ m/K}$.

$\gamma_a = P_a/[R_g T_a] = (239.4 \text{ kN/m}^2)/[(29.26 \text{ m/K})(27.8 + 273)\text{K}]$.

$\gamma_a = 0.027 \text{ kN/m}^3 = 0.027 \times 1{,}000/9.81 \text{ kg/m}^3 = 2.752 \text{ kg/m}^3$.

Here 1 kg = 9.81 N.

$W = 0.25 \text{ kg/s} = \gamma_a Q = (2.752 \text{ kg/m}^3)Q$.

$Q = 0.09084 \text{ m}^3\text{/s} = Av = A(5.8 \text{ m/s})$.

$A = 0.01788 \text{ m}^2 = D^2 \times 0.785$.

$\boldsymbol{D} = (0.01788/0.785)^{0.5} = \mathbf{0.15\ m = 15\ cm}$.

EXAMPLE 5.3 COMPRESSIBLE CARBON DIOXIDE GAS FLOW CALCULATIONS

Carbon dioxide passes point A in a 3.5 in. (0.0889 m) pipe at a velocity of 16 ft/s (4.88 m/s). The pressure at point A is 32 psi (222 kPa) and the temperature is 70°F (21°C). At point B downstream the pressure is 22 psi (152.7 kPa) and the temperature is 85°F (29.4°C). For a barometric reading of 14.7 psi (102 kPa), determine the gas velocity at point B, and the gas flows at points A and B. The gas constant of carbon dioxide is 38.86 ft/°R (0.209 kJ/kg-K).

Solution 1 (US Customary System):

P_a = absolute pressure = $P_{\text{gauge}} + P_{\text{atm}} = 32 + 14.7 = 46.7$ psi.

$P_a = (46.7 \text{ lb/in.}^2)(144 \text{ in.}^2\text{/ft}^2) = 6{,}724.8 \text{ lb/ft}^2$.

T_a = absolute temperature = $T + 460 = 70 + 460 = 530$°R.

$\gamma_A = P_a/[R_g T_a] = [(32 + 14.7) \times 144]/[38.86(70 + 460)]$.

$\gamma_A = 6{,}724.8 \text{ lb/ft}^2/[(38.86 \text{ ft/°R})(530\text{°R})] = 0.326 \text{ lb/ft}^3$.

Similarly γ_B can be determined:

$\gamma_B = P_a/[R_g T_a] = [(22 + 14.7) \times 144]/[38.86(85 + 460)] = 0.249 \text{ lb/ft}^3$.

$W \text{ (lb/s)} = (\gamma_A)(A_A)(v_A) = (\gamma_B)(A_B)(v_B) = (\gamma_A)(Q_A) = (\gamma_B)(Q_B)$.

Since $A_A = A_B$, then $(\gamma_A)(v_A) = (\gamma_B)(v_B)$.

$(0.326 \text{ lb/ft}^3)(16 \text{ ft/s}) = (0.249 \text{ lb/ft}^3)(v_B)$.

$\boldsymbol{v_B} = \mathbf{20.9\ ft/s}$.

$A_A = A_B = (3.5/12)^2(3.14/4) = 0.0668 \text{ ft}^2$.

$\boldsymbol{Q_A} = (A_A)(v_A) = (0.0668 \text{ ft}^2)(16 \text{ ft/s}) = \mathbf{1.068\ ft^3/s}$.

$\boldsymbol{Q_B} = (A_B)(v_B) = (0.0668 \text{ ft}^2)(20.9 \text{ ft/s}) = \mathbf{1.396\ ft^3/s}$.

Solution 2 (SI System):

P_a = absolute pressure = $P_{\text{gauge}} + P_{\text{atm}} = 222 + 102 = 324 \text{ kPa} = 324 \text{ kN/m}^2$.

$R_g = 0.209 \text{ kJ/kg-K} = 209 \text{ J/kg-K} = 209 \times 0.101972 \text{ kg-m/kg-K} = 21.31 \text{ m/K} = 38.86 \text{ ft-lb/lb-°R}$.

T_a = absolute temperature = $T + 273 = 21 + 273 = 294$ K.

$\gamma_A = P_a/[R_g T_a] = (324 \text{ kN/m}^2)/[(21.31 \text{ m/K})(294 \text{ K})] = 0.0517 \text{ kN/m}^3$.

$\gamma_A = (0.0517 \times 1{,}000)/9.81 \text{ kg/m}^3 = 5.27 \text{ kg/m}^3$.

Similarly γ_B can also be determined:

$\gamma_B = P_a/[R_g T_a] = (152.7 + 102)/[21.31(29.4 + 273)] = 0.0395\ \text{kN/m}^3$.

$\gamma_B = (0.0395 \times 1{,}000)/9.81\ \text{kg/m}^3 = 4.029\ \text{kg/m}^3$.

$W\ (\text{kg/s}) = (\gamma_A)(A_A)(v_A) = (\gamma_B)(A_B)(v_B) = (\gamma_A)(Q_A) = (\gamma_B)(Q_B)$.

Since $A_A = A_B$, then $(\gamma_A)(v_A) = (\gamma_B)(v_B)$.

$(5.27\ \text{kg/m}^3)(4.88\ \text{m/s}) = (4.029\ \text{kg/m}^3)(v_B)$.

$\boldsymbol{v_B = 6.38\ \textbf{m/s}}$.

$A_A = A_B = (0.0889)^2(3.14/4) = 0.0062\ \text{m}^2$.

$\boldsymbol{Q_A} = (A_A)(v_A) = (0.0062\ \text{m}^2)(4.88\ \text{m/s}) = 0.03\ \text{m}^3/\text{s}$.

$\boldsymbol{Q_B} = (A_B)(v_B) = (0.0062\ \text{m}^2)(6.38\ \text{m/s}) = 0.04\ \text{m}^3/\text{s}$.

EXAMPLE 5.4 COMPRESSIBLE AIR FLOW CALCULATIONS

Air flows in a 6.5 in. (165 mm) pipe at a pressure of 35 psi (242.9 kPa) gauge and a temperature of 100°F (37.78°C). If the barometric pressure is 14.7 psi (102 kPa) and the velocity is 10.8 ft/s (3.29 m/s), determine the air mass flow (lb/s and kg/s) when air is flowing. Gas constant for air is 53.3 ft/°R absolute (0.287 kJ/kg-K).

Solution 1 (US Customary System):

The gas laws require absolute units for temperature and pressure, when using Eq. (5.2).

P_a = absolute pressure $= P_{\text{gauge}} + P_{\text{atm}} = 35 + 14.7 = 49.7$ psi.

$P_a = (49.7\ \text{lb/in.}^2)\ (144\ \text{in.}^2/\text{ft}^2) = 7{,}156.8\ \text{lb/ft}^2$.

$R_g = 53.3$ ft/°R absolute.

T_a = absolute temperature $= T + 460 = 100 + 460 = 560$°R absolute.

$\gamma = P_a/[R_g T_a] = (7{,}156.8\ \text{lb/ft}^2)/[(53.3\ \text{ft/°R})(560\text{°R})] = 0.24\ \text{lb/ft}^3$ for air.

W = mass flow $= (\gamma)(A)(v) = (\gamma)(\underline{Q})$.

$\boldsymbol{W} = (0.24\ \text{lb/ft}^3)[(6.5/12)^2(3.14/4)\ \text{ft}^2](10.8\ \text{ft/s}) =$ **0.597 lb/s**.

Solution 2 (SI System):

P_a = absolute pressure $= P_{\text{gauge}} + P_{\text{atm}} = 242.9 + 102 = 344.9\ \text{kPa} = 344.9\ \text{kN/m}^2$.

$R_g = 53.3$ ft/°R absolute $= 0.287$ kJ/kg-K $= 287 \times 0.101972$ kg-m/kg-K $= 29.26$ m/K.

Here 1 J = 0.101972 kg-m.

T_a = absolute temperature $= T + 273 = 37.78 + 273 = 310.8$ K absolute.

$\gamma = P_a/[R_g T_a] = (344.9\ \text{kN/m}^2)/[(29.26\ \text{m/K})(310.8\ \text{K})] = 0.0379\ \text{kN/m}^3$ for air $= 3.87\ \text{kg/m}^3$.

Here 1 kg = 9.81 N.

W = mass flow $= (\gamma)(A)(v) = (\gamma)(Q)$.

$\boldsymbol{W} = (3.87\ \text{kg/m}^3)[(0.165\ \text{m})^2(3.14/4)](3.29\ \text{m/s}) =$ **0.272 kg/s**.

EXAMPLE 5.5 CALCULATION OF VELOCITIES IN WATER PIPES

When 550 gpm (2,082 Lpm) water flows through a 12 in. (30.48 cm) pipe, which later reduces to a 6 in. (15.24 cm) pipe, calculate the average velocities in the two pipes.

Solution 1 (US Customary System):

$Q = (A_1)(v_1) = (A_2)(v_2) = 550\ \text{gpm} = 1.224\ \text{ft}^3/\text{s}$.

$A_1 = (12/12)^2 \times 0.785\ \text{ft}^2 = 0.785\ \text{ft}^2$.

$A_2 = (6/12)^2 \times 0.785\ \text{ft}^2 = 0.196\ \text{ft}^2$.

$Q = 1.224\ \text{ft}^3/\text{s} = (A_1)(v_1) = (A_2)(v_2)$.

$1.224\ \text{ft}^3/\text{s} = (0.785\ \text{ft}^2)(v_1) = (0.196\ \text{ft}^2)(v_2)$.

$\boldsymbol{v_1} = \mathbf{1.6\ ft/s}$.

$\boldsymbol{v_2} = \mathbf{6.2\ ft/s}$.

Solution 2 (SI System):

$Q = (A_1)(v_1) = (A_2)(v_2) = 2{,}082\ \text{Lpm} = 0.0347\ \text{m}^3/\text{s}$.

$A_1 = (30.7/100)^2 \times 0.785\ \text{m}^2 = 0.0725\ \text{m}^2$.

$A_2 = (15.24/100)^2 \times 0.785\ \text{m}^2 = 0.0182\ \text{m}^2$.

$Q = 0.0347\ \text{m}^3/\text{s} = (A_1)(v_1) = (A_2)(v_2)$.

$0.0347\ \text{m}^3/\text{s} = (0.0725\ \text{m}^2)(v_1) = (0.0182\ \text{m}^2)(v_2)$.

$\boldsymbol{v_1} = \mathbf{0.479\ m/s}$.

$\boldsymbol{v_2} = \mathbf{1.907\ m/s}$.

EXAMPLE 5.6 DETERMINATION OF HYDRAULIC PRESSURE IN A WATER PIPE

Water flows through the nozzle shown in Fig. 5.1 and deflects the mercury in the U-tube gauge at a treatment plant. Determine the value of *h* if the pressure at point D is 20 psi (138.8 kN/m^2), and *m* is 2.75 ft (0.84 m).

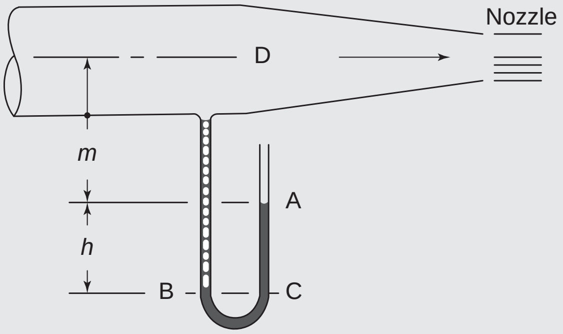


Figure 5.1 Fluid flow through a nozzle.

Solution 1 (US Customary System):

Method A. Pressure balance method:

$20\ \text{lb/in.}^2 + [(62.4\ \text{lb/ft}^3)(2.75 + h)\ \text{ft}]/(144\ \text{in.}^2/\text{ft}^2) = [(13.57 \times 62.4\ \text{lb/ft}^3)(h)\ \text{ft}]/(144\ \text{in.}^2/\text{ft}^2) + P_A$.

$P_A = 0\ \text{psi} = 0\ \text{lb/in.}^2$.

$144 \times 20 + 62.4\,(2.75 + h) = 13.57 \times 62.4h$.

$\boldsymbol{h} = \mathbf{3.9\ ft}$.

Method B. Water column length method:

1 psi = 2.307 ft of water.

1 ft Hg = 13.57 ft of water.

$20 \times 2.307 + (2.75 + h) = 13.57h$.

$\boldsymbol{h} = \mathbf{3.9\ ft}$.

Solution 2 (SI System):

Method A. Pressure balance method:

$138.8\ \text{kN/m}^2 + (9.8\ \text{kN/m}^3)(0.84\ \text{m} + h) = (13.57 \times 9.8\ \text{kN/m}^3)h + P_A$.

$P_A = 0\ \text{kN/m}^3$ assumed.

$138.8 + 9.8 \times 0.84 + 9.8h = 13.57 \times 9.8h$.

$\boldsymbol{h = 1.19\ \text{m}}$.

Method B. Water column length method:

$1\ \text{psi} = 2.307\ \text{ft of water} = 0.703\ \text{m of water} = 6.94\ \text{kN/m}^2$.

$1\ \text{m Hg} = 13.57\ \text{m of water}$.

$1\ \text{kN/m}^2 = (0.703/6.94)\ \text{m}\ H_2O = 0.1013\ \text{m}\ H_2O$

$138.8 \times 0.1013\ \text{m} + 0.84\ \text{m} + h = 13.57h$.

$\boldsymbol{h = 1.1\ \text{m}}$.

EXAMPLE 5.7 DETERMINATION OF WATER FORCE ON RISER PIPE

Referring to Fig. 5.2, determine (a) the force exerted by water on the bottom plate AB of the 2-ft (0.61-m)-diameter riser pipe for water storage and (b) the total force on plane C. The dimensions given are $a = 2$ ft (0.61 m), $b = 12$ ft (3.66 m), $c = 6$ ft (1.83 m), and $d = 10$ ft (3.05 m).

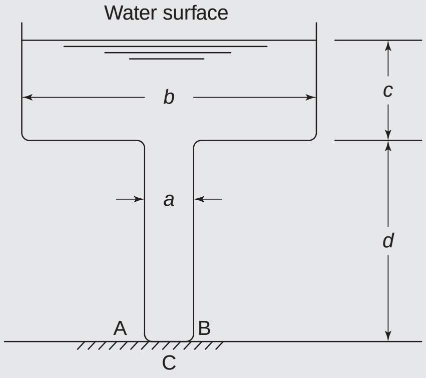


Figure 5.2 Riser pipe analysis.

Solution 1 (US Customary System):

a. $P_{fAB} = (16 \times 2^2 \times 0.785\ \text{ft}^3)(62.4\ \text{lb/ft}^3) = 3{,}135\ \text{lb}$.
$p_{AB} = P_{fAB}/A = 3{,}135\ \text{lb}/[(2^2 \times 0.785\ \text{ft}^2)(144\ \text{in.}^2/\text{ft}^2) = \mathbf{6.93\ psi}$.

b. $P_{fC} = [(12^2 \times 0.785 \times 6\ \text{ft}^3) + (2^2 \times 0.785 \times 10\ \text{ft}^3)]\ (62.4\ \text{lb/ft}^3) = 44{,}282\ \text{lb}$.
$p_C = 44{,}282\ \text{lb}/[(2^2 \times 0.785\ \text{ft}^2)(144\ \text{in.}^2/\text{ft}^2)] = \mathbf{97.93\ psi}$.

Solution 2 (SI System):

a. $P_{fAB} = (1.83 + 3.05) \times 0.61^2 \times 0.785\ \text{m}^3(1{,}000\ \text{kg/m}^3) = 1{,}425.4\ \text{kg}$.
$p_{AB} = P_{fAB}/A = 1{,}425.4\ \text{kg}/(0.61^2 \times 0.785\ \text{m}^2) = \mathbf{4{,}880\ kg/m^2}$.

b. $P_{fC} = [(3.66^2 \times 0.785 \times 1.83\ \text{m}^3) + (0.61^2 \times 0.785 \times 3.05\ \text{m}^3)](1{,}000\ \text{kg/m}^3) = 20{,}131\ \text{kg}$.
$p_C = 20{,}131\ \text{kg}/(0.61^2 \times 0.785\ \text{m}^2) = \mathbf{68{,}919\ kg/m^2}$.

EXAMPLE 5.8 HYDRAULIC FORCE EQUILIBRIUM

In Fig. 5.3, the areas of the hydraulic plunger A and cylinder B are 6.5 in.^{2} (42.52 cm^2) and 650 in.^{2} (4,193.15 cm^2), respectively. The weight of B is 9,500 lb (4,313 kg). The vessel and the connecting passages are filled with water of specific gravity 1.0. What force P_f is required for hydraulic force equilibrium, neglecting the weight of A? Distance $d = 16$ ft = 4.88 m.

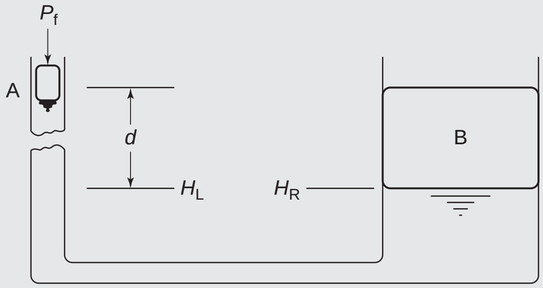


Figure 5.3 Force analysis for a vessel with a plunger and a cylinder.

Solution 1 (US Customary System):

Determine the unit pressure acting on plunger A first. Since H_L and H_R are at the same level in water, then

Pressure at H_L (psi) = pressure at H_R (psi).
Pressure at H_L = pressure under A + 16 ft water.
Pressure at H_R = 9,500 lb/650 in.^{2} = 14.65 psi.
$P_f/(6.5 \text{ in.}^2) + (62.4 \text{ lb/ft}^3)(16 \text{ ft})/(144 \text{ in.}^2/\text{ft}^2) = 9{,}500 \text{ lb}/(650 \text{ in.}^2)$.
$\boldsymbol{P_f}$ **= 50.18 lb**.

Solution 2 (SI System):

Determine the unit pressure acting on plunger A first. Since H_L and H_R are at the same level in water, then

Pressure at H_L (g/cm^2) = pressure at H_R (g/cm^2).
Pressure at H_L = pressure under A + 4.88 m water.
Pressure at H_R = 4,313,000 g/4,193.15 cm^2.
$\gamma = 1 \text{ g/cm}^3 = 1 \text{ kg/L} = 1 \text{ kg}/1{,}000 \text{ cm}^3$.
$P_f/(42.52 \text{ cm}^2) + (1 \text{ g/cm}^3)(488 \text{ cm}) = 4{,}313{,}000 \text{ g}/4{,}193.15 \text{ cm}^2$.
$\boldsymbol{P_f}$ **= 22,985 g = 22.99 kg**.

EXAMPLE 5.9 DETERMINATION OF HYDRAULIC FORCE EXERTED BY FLUID IN A TANK

Determine the hydraulic force developed on one of the vertical sides of a cubical tank with 4 ft (1.22 m) sides filled with water. Also determine the locations of the center of gravity C_g and the center of pressure C_p, as shown in Fig. 5.4.

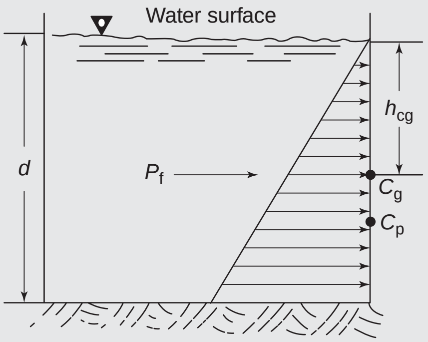


Figure 5.4 Cubical tank with four equal sides.

Solution 1 (US Customary System):

Average pressure exerted on one side is pressure existent at C_g of the side times area A of the side. Since pressure varies linearly with depth, as shown in Fig. 5.4, the C_p (center of pressure) is located below the C_g (center of gravity). The C_p is the location where a single concentrated force P_f can be used to replace the triangle-shape distributed pressure forces for moment purposes. The pressure existent at the area centroid times area equals force exerted normal to the area acting through the C_p.

$h_{cg} = 0.5 \times d = 0.5 \times 4$ ft $= 2$ ft.
$\mathbf{C_g}$ **= 2 ft down from water surface**.
$\mathbf{P_f} = (\gamma h_{cg})A = I_p A = (62.4\ \text{lb/ft}^3)(2\ \text{ft})(4\ \text{ft} \times 4\ \text{ft}) =$ **2,000 lb**.
$y_{cg} = h_{cg} = 2$ ft in this particular case.
$b = 4$ ft.
$h = 4$ ft.
$I_{cg} = bh^3/12 = 4 \times 4^3/12 = 21.33\ \text{ft}^4$.
$y_{cp} = [I_{cg}/y_{cg}A] + y_{cg} = [21.33\ \text{ft}^4/(2\ \text{ft} \times 4\ \text{ft} \times 4\ \text{ft})] + 2\ \text{ft} = 2.67$ ft.
It is noted that $2.67/4 = 2/3$.
$\mathbf{C_p}$ **= 2.67 ft down from water surface**.

Solution 2 (SI System):

Read the explanations in Solution 1.

$h_{cg} = 0.5 \times d = 0.5 \times 1.22\ \text{m} = 0.61$ m.
$\mathbf{C_g}$ **= 0.61 m downward from water surface**.
$\mathbf{P_f} = (\gamma h_{cg})A = I_p A = (9.8\ \text{kN/m}^3)(0.5 \times 1.22\ \text{m})(1.22\ \text{m} \times 1.22\ \text{m}) =$ **8.9 kN**.
$y_{cg} = h_{cg} = 0.5 \times 1.22\ \text{m} = 0.61$ m in this particular case.
$b = 1.22$ m.
$h = 1.22$ m.
$I_{cg} = bh^3/12 = 1.22 \times 1.22^3/12 = 0.185\ \text{m}^4$.
$y_{cp} = [I_{cg}/y_{cg}A] + y_{cg} = [0.185\ \text{m}^4/(0.61\ \text{m} \times 1.22\ \text{m} \times 1.22\ \text{m})] + 0.61\ \text{m} = 0.81$ m.
$\mathbf{C_p}$ **= 0.81 m downward from water surface**.

EXAMPLE 5.10 DETERMINATION OF DEPTH OF CENTER OF PRESSURE

Determine the depth to the center of pressure y_{cp} for a rectangular area ($b \times h$) vertically submerged with the long side ($h = 8.2$ ft $= 2.5$ m) at the water surface.

Solution 1 (US Customary System):

Using Eqs. (5.8) and (5.9), $y_{cp} = [I_{cg}/y_{cg}A] + y_{cg}$, where $y_{cg} = h/2$, $A = b \times h$, and $I_{cg} = bh^3/12$.

$y_{cp} = (bh^3/12)/[(h/2)(bh)] + (h/2) = 2h/3$.
$\mathbf{y_{cp}} = (2 \times 8.2\ \text{ft})/3 =$ **5.47 ft**.

Solution 2 (SI System):

Using Eqs. (5.8) and (5.9), $y_{cp} = [I_{cg}/y_{cg}A] + y_{cg}$, where $y_{cg} = h/2$, $A = b \times h$, and $I_{cg} = bh^3/12$.

$y_{cp} = (bh^3/12)/[(h/2)(bh)] + (h/2) = 2h/3$.
$\mathbf{y_{cp}} = (2 \times 2.5\ \text{m})/3 =$ **1.67 m**.

EXAMPLE 5.11 DETERMINATION OF HYDROSTATIC FORCE AND LOCATION OF CENTER OF PRESSURE

Determine the hydrostatic force and the location of the center of pressure on the 92-ft (28-m)-long dam shown in Fig. 5.5. The face of the dam is at an angle of 60°. The water temperature is 77°F (25°C) and the depth is 26.25 ft (8 m).

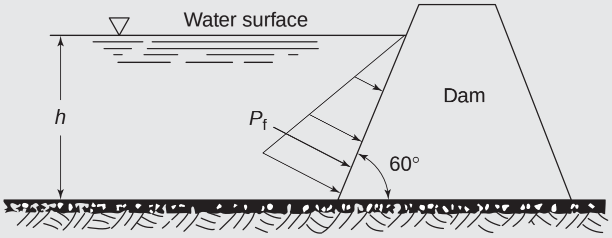


Figure 5.5 Hydrostatic force on a dam.

Solution 1 (US Customary System):

From Appendix 3, $\gamma = 62.28$ lb/ft^3 at 77°F.

$$\boldsymbol{P_f} = (\gamma h_{cg})A = (62.28 \text{ lb/ft}^3)(26.25 \text{ ft}/2)(92 \text{ ft})(26.25 \text{ ft}/\sin 60°) = \mathbf{2{,}279.54\ lb}.$$

The center of pressure is at two-thirds of the total water depth, (2/3)(26.25 ft) = **17.5 ft**.

Solution 2 (SI System):

From Appendix 3, $\gamma = 9.779$ kN/m^3 at 25°C.

$$\boldsymbol{P_f} = (\gamma h_{cg})A = (9.779 \text{ kN/m}^3)(8 \text{ m}/2)(28 \text{ m})(8\text{m}/\sin 60°) = 10{,}118 \text{ kN}.$$

The center of pressure is at two-thirds of the total water depth, (2/3)(8 m) = **5.33 m**.

EXAMPLE 5.12 DETERMINATION OF RESULTANT FORCE DUE TO WATER PRESSURE ACTING ON VERTICAL GATE

Determine the resultant force and its location due to the water acting on a 4 ft by 8 ft (1.22 m by 2.44 m) rectangular gate AB shown in Fig. 5.6. The top of the gate is 16 ft (4.88 m) below the water surface. The dimensions in Fig. 5.6 are $a = 16$ ft $= 4.88$ m; $h = 8$ ft $=$ 2.44 m; and $b = 4$ ft $= 1.22$ m.

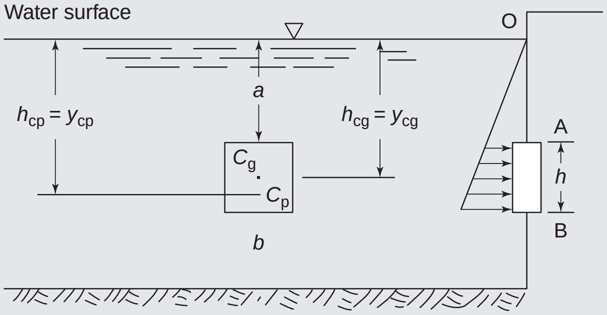


Figure 5.6 Vertical rectangular gate in a water tank.

Solution 1 (US Customary System):

$$\boldsymbol{P_f} = (\gamma h_{cg})A = (62.4 \text{ lb/ft}^3)(16 \text{ ft} + 8 \text{ ft} \times 0.5)(4 \text{ ft} \times 8 \text{ ft}) = \mathbf{39{,}936\ lb}.$$

The above resultant force acts at the center of pressure, which is at a distance y_{cp} from water surface O.

At vertical position, $h_{cg} = y_{cg}$.

$y_{cg} = 16 \text{ ft} + 8 \text{ ft} \times 0.5 = 20 \text{ ft}.$

$A = 4 \text{ ft} \times 8 \text{ ft} = 32 \text{ ft}^2.$

$y_{cp} = [I_{cg}/y_{cg}A] + y_{cg}$, where $A = b \times h$, and $I_{cg} = bh^3/12$.

$\boldsymbol{y}_{\mathbf{cp}} = [I_{cg}/y_{cg}A] + y_{cg} = [(4 \times 8^3 \text{ ft}^4/12)/(20 \text{ ft} \times 32 \text{ ft}^2)] + 20 \text{ ft}$

$=$ **20.27 ft from point O**.

Solution 2 (SI System):

$P_{\mathrm{f}} = (\gamma h_{cg})A = (9.8 \text{ kN/m}^3)(4.88 \text{ m} + 2.44 \text{ m} \times 0.5)(1.22 \text{ m} \times 2.44 \text{ m}) =$ **177.95 kN**.

The above resultant force acts at the center of pressure, which is at a distance y_{cp} from water surface O.

At vertical position, $h_{cg} = y_{cg}$.

$y_{cg} = 4.88 \text{ m} + 2.44 \text{ m} \times 0.5 = 6.1 \text{ m}$.

$A = 1.22 \text{ m} \times 2.44 \text{ m} = 2.98 \text{ m}^2$.

$y_{cp} = [I_{cg}/y_{cg}A] + y_{cg}$, where $A = b \times h$, and $I_{cg} = bh^3/12$.

$\boldsymbol{y}_{\mathbf{cp}} = [I_{cg}/y_{cg}A] + y_{cg} = [(1.22 \times 2.44^3 \text{ m}^4/12)/(6.1 \text{ m} \times 2.98 \text{ m}^2)] + 6.1 \text{ m}$

$=$ **6.18 m from point O**.

EXAMPLE 5.13 DETERMINATION OF RESULTANT FORCE DUE TO WATER ACTING ON AN INCLINED RECTANGULAR GATE

Determine the total resultant force and its location acting on the gate shown in Fig. 5.7. The gate is 4.5 ft × 6.5 ft (1.37 m × 1.98 m) in dimension. The apex of the triangle is at C. The gate top is 5.5 ft (1.68 m) below the water surface. The related dimensions in the figure are $a = 5.5$ ft $= 1.68$ m; $h = 4.5$ ft $= 1.37$ m; $b = 6.5$ ft $= 1.98$ m.

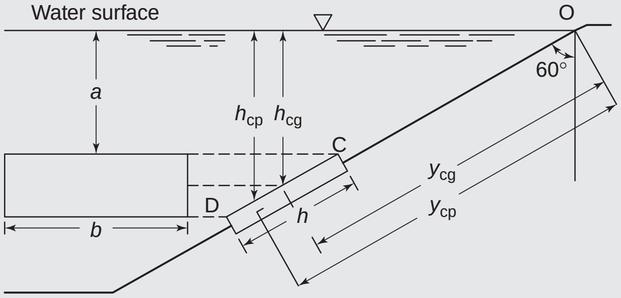


Figure 5.7 Inclined rectangular gate in a water tank.

Solution 1 (US Customary System):

$h_{cg} = 5.5 \text{ ft} + 0.5(4.5 \text{ Cos } 60°) = 6.625 \text{ ft}$.

$\boldsymbol{P}_{\mathbf{f}} = (\gamma h_{cg})A = (62.4 \text{ lb/ft}^3)(6.625 \text{ ft})(4.5 \text{ ft} \times 6.5 \text{ ft}) =$ **12,092 lb**.

$y_{cg} = h_{cg}/\text{Sin } 30° = 6.625 \text{ ft}/0.5 = 13.25 \text{ ft}$, or

$y_{cg} = (5.5 \text{ ft}/\text{Cos } 60°) + (4.5 \text{ ft}/2) = 13.25 \text{ ft}$.

$I_{cg} = bh^3/12 = (6.5 \times 4.5^3 \text{ ft}^4)/12 = 49.35 \text{ ft}^4$.

$y_{cp} = [I_{cg}/y_{cg}A] + y_{cg}$, where $A = b \times h$, and $I_{cg} = bh^3/12$.

$y_{cp} = [I_{cg}/y_{cg}A] + y_{cg} = [49.35 \text{ ft}^4/(13.25 \text{ ft} \times 4.5 \text{ ft} \times 6.5 \text{ ft})] + 13.25 \text{ ft} = 13.38 \text{ ft}$.

$\boldsymbol{h}_{\mathbf{cp}} = y_{cp} \text{ Sin } 30° = 13.38 \text{ ft} \times 0.5 =$ **6.69 ft**.

Solution 2 (SI System):

$h_{cg} = 1.68 \text{ m} + 0.5(1.37 \text{ m Cos } 60°) = 2.02 \text{ m}$.

$\boldsymbol{P}_{\mathbf{f}} = (\gamma h_{cg})A = (9.79 \text{ kN/m}^3)(2.02 \text{ m})(1.37 \text{ m} \times 1.98 \text{ m}) =$ **53.64 kN**.

$y_{cg} = h_{cg}/\text{Sin } 30° = 2.02 \text{ m}/0.5 = 4.04 \text{ m}$, or

$y_{cg} = (1.68 \text{ m}/\text{Cos } 60°) + (1.37 \text{ m}/2) = 4.04 \text{ m}$.

$I_{cg} = bh^3/12 = (1.98 \times 1.37^3 \text{ m}^4)/12 = 0.4243 \text{ m}^4$.

$y_{cp} = [I_{cg}/y_{cg}A] + y_{cg}$, where $A = b \times h$, and $I_{cg} = bh^3/12$.

$\mathbf{y_{cp} = [I_{cg}/y_{cg}A] + y_{cg} = [0.4243 \text{ m}^4/(4.04 \text{ m} \times 1.37 \text{ m} \times 1.98 \text{ m})] + 4.04 \text{ m} = 4.08 \text{ m}}$.

$\mathbf{h_{cp} = y_{cp} \text{ Sin } 30° = 4.08 \text{ m} \times 0.5 = 2.04 \text{ m}}$.

EXAMPLE 5.14 WATER, SPECIFIC WEIGHT, FORCE, AND MOMENT OF INERTIA

Water rises to level E in the pipe attached to water storage tank ABCD in Fig. 5.8. Neglecting the weight of the tank and riser pipe, determine (a) the resultant force acting on area AB, which is 8 ft wide (2.4 m); (b) the location of the resultant force acting on area AB; (c) the pressure on the bottom BC; and (d) the total weight of the water.

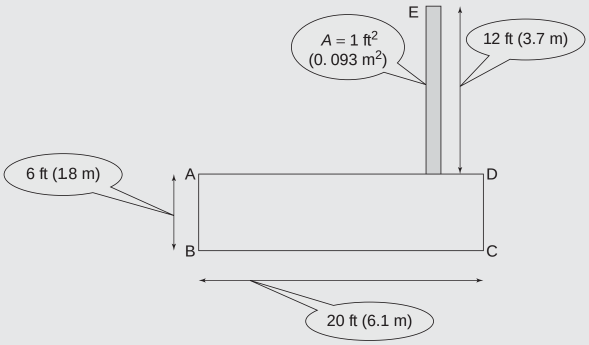


Figure 5.8 Water tank for Example 5.14.

Solution 1 (US Customary System):

a. The resultant force, P_f, acting on area AB:

$$\begin{aligned} \mathbf{P_f} &= (\gamma h_{cg})A \\ &= (62.4 \text{ lb/ft}^3)(12 \text{ ft} + 3 \text{ ft})(6 \text{ ft} \times 8 \text{ ft}) \\ &= \mathbf{45{,}000 \text{ lb}}. \end{aligned}$$

b. The location of the resultant force acting on area AB:

$$\begin{aligned} y_{cp} &= [I_{cg}/y_{cg}A] + y_{cg} \\ I_{cg} &= (1/12)bh^3 \\ y_{cp} &= [8(6)^3/12]/[15(6 \times 8)] + 15 \\ &= 0.20 + 15 \\ &= \mathbf{15.2 \text{ ft}}. \end{aligned}$$

c. The pressure, p, on the bottom BC:

$$\begin{aligned} p &= \gamma h \\ &= (62.4 \text{ lb/ft}^3)(6 \text{ ft} + 12 \text{ ft})/144 \text{ in.}^2/\text{ft}^2 \\ &= \mathbf{7.8 \text{ psi}}. \end{aligned}$$

d. The total weight, W, of the water:

$$\begin{aligned} \mathbf{W} &= \gamma V \\ &= 62.4 \text{ lb/ft}^3(20 \times 6 \times 8 + 12 \times 1)\text{ft}^3 \\ &= \mathbf{60{,}700 \text{ lb}}. \end{aligned}$$

Solution 2 (SI System):

a. Specific weight for water at 39°F (4°C) is 62.4 $\mathrm{lb/ft^3}$(9.81 $\mathrm{kN/m^3}$). Specific weight in SI unit can be calculated as

$$\begin{aligned}\gamma &= (\rho)(g) = (1{,}000\ \mathrm{kg/m^3})(9.81\ \mathrm{m/s^2})\\ &= 9{,}810\ \mathrm{N/m^3} = 9.81\ \mathrm{kN/m^3}.\end{aligned}$$

The resultant force, P_f, acting on area AB:

$$\begin{aligned}P_\mathrm{f} &= (\gamma)(hcg)(A)\\ &= (9.81\ \mathrm{kN/m^3})(3.7\ \mathrm{m} + 1.8\ \mathrm{m}/2)(1.8\ \mathrm{m} \times 2.44\ \mathrm{m})\\ &= \mathbf{198.2\ kN}.\end{aligned}$$

b. The location of the resultant force acting on area AB:

$$\begin{aligned}y_\mathrm{cp} &= [I_\mathrm{cg}/y_\mathrm{cg}A] + y_\mathrm{cg}\\ I_\mathrm{cg} &= (1/12)bh^3\\ y_\mathrm{cp} &= [(bh^3/12)/y_\mathrm{cg}A] + y_\mathrm{cg}\\ &= (2.44 \times 1.8^3/12)/[(4.56)(1.8 \times 2.44)] + 4.56\\ &= \mathbf{4.62\ m}.\end{aligned}$$

c. The pressure, p, on the bottom BC:

$$\begin{aligned}p &= \gamma h = (9.81\ \mathrm{kN/m^3})(1.8\ \mathrm{m} + 3.7\ \mathrm{m}) = 53.9\ \mathrm{kN/m^2}\\ &= \mathbf{53.9\ kPa}.\end{aligned}$$

d. The total weight, W, of the water:

$$\begin{aligned}W &= \rho V\\ &= (1{,}000\ \mathrm{kg/m^3})(6.1\ \mathrm{m} \times 1.8\ \mathrm{m} \times 2.44\ \mathrm{m} + 3.7\ \mathrm{m} \times 0.093\ \mathrm{m^2})\\ &= \mathbf{26{,}865\ kg}.\end{aligned}$$

5.1.2 Transmission Systems

Supply *conduits*, or *aqueducts*, transport water from the source of supply to the community and, hence, form the connecting link between collection works and distribution systems. The location of the source determines whether conduits are short or long, and whether transport is by *gravity* or *pumping*. Depending on topography and available materials, conduits are designed for *open channel* or *pressure flow*. They may follow the *hydraulic grade line* (HGL) as canals dug through the ground, flumes elevated above the ground, grade aqueducts laid in balanced cut and cover at the ground surface, and grade tunnels penetrating hills; or they may depart from the HGL as pressure aqueducts laid in balanced cut and cover at the ground surface, pressure tunnels dipping beneath valleys or hills, and pipelines of fabricated materials following the ground surface, if necessary over hill and through dale, sometimes even rising above the HGL. Illustrative examples of supply conduits and aqueducts include the following:

1. The *Central Arizona aqueduct* is a multipurpose water resource project that delivers Colorado River water into central and southern Arizona (see Fig. 1.6). The aqueduct diversion canal runs about 336 mi (541 km) from Lake Havasu to a terminus 14 mi (22.5 km) southwest of Tucson. The final extension to Tucson includes a tunnel through the mountains.
2. The *Colorado River aqueduct* of the Metropolitan Water District of Southern California is 242 mi (389 km) long and includes 92 mi (148 km) of grade tunnel, 63 mi (101 km) of canal, 54 mi (86.9 mi) of grade aqueduct, 29 mi (46.7 km) of inverted siphons, and 4 mi (6.4 km) of force main.
3. The *Delaware aqueduct* of New York City comprises 85 mi (137 km) of pressure tunnel in three sections.
4. *Pressure tunnels* 25 mi (40.2 km) long supply the metropolitan districts of Boston and San Francisco.
5. The *supply conduits* of Springfield, MA, are made of steel pipe and reinforced concrete pipe; those of Albany, NY, of cast iron pipe.

Figure 5.9 Water channel of the Nanzenji aqueduct, Kyoto, Japan (*Source:* http://en.wikipedia.org/wiki/Image:Nanzenji_aqueduct_channel.jpg).

6. The *Nanzenji aqueduct* water channel of Kyoto, Japan (Fig. 5.9).

The profile and typical cross-sections of a supply conduit are shown in Fig. 5.10. Static heads and HGLs are indicated for pressure conduits.

5.2 FLUID TRANSPORT

The hydraulic design of supply conduits is concerned chiefly with (a) resistance to flow in relation to available and needed heads or pressures and (b) required and allowable velocities of flow relative to cost, scour, and sediment transport. In long supply lines, *frictional* or *surface resistance* offered by the pipe interior is the dominant element. *Form resistance* responsible for losses in transitions and appurtenances is often negligible. In short transport systems, on the other hand, form resistance may be of controlling importance.

5.2.1 Rational Equation for Surface Resistance

The most nearly rational relationship between velocity of flow and head loss in a conduit is also one of the earliest. Generally referred to as the *Darcy–Weisbach formula*, it is actually written in the form suggested by Weisbach, rather than Darcy, namely,

$$h_\mathrm{f} = f(L/d)(v^2/2g) \qquad \textbf{(5.10a)}$$

$$h_\mathrm{f} = KQ^2 \qquad \textbf{(5.10b)}$$

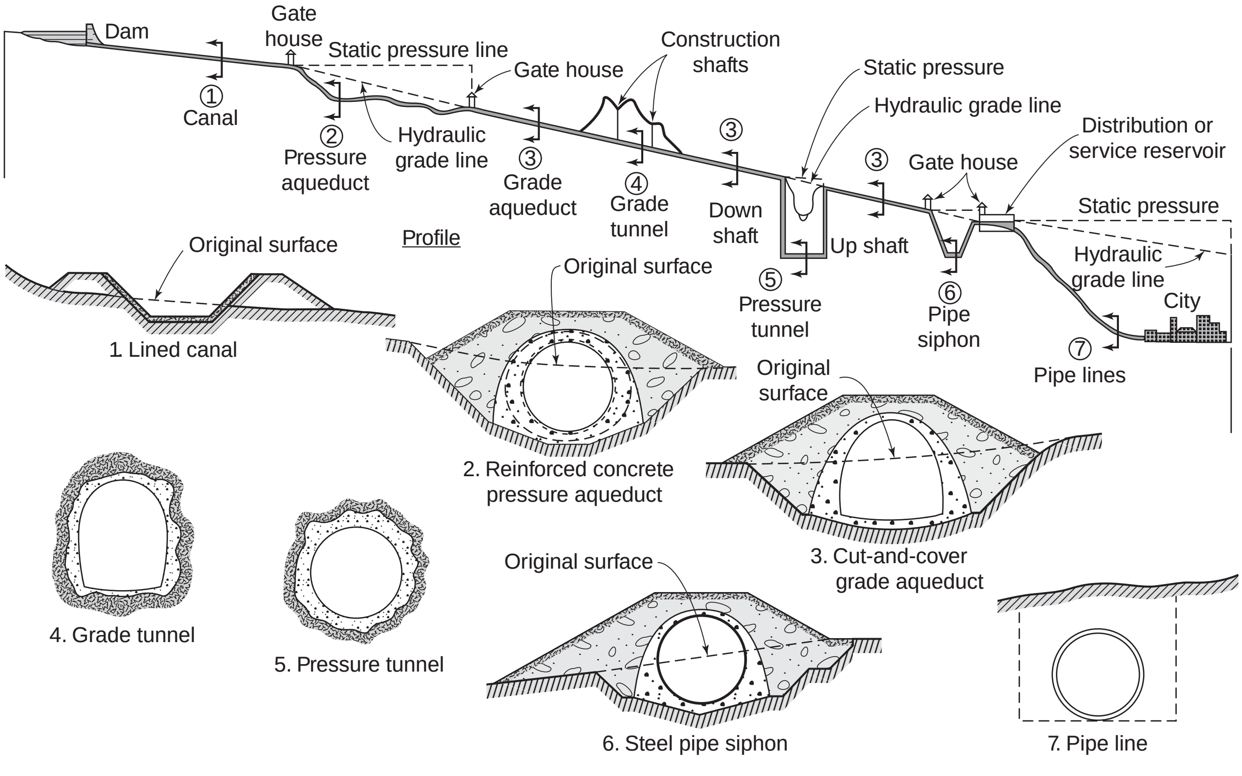


Figure 5.10 Profile and typical cross-sections of a water supply conduit.

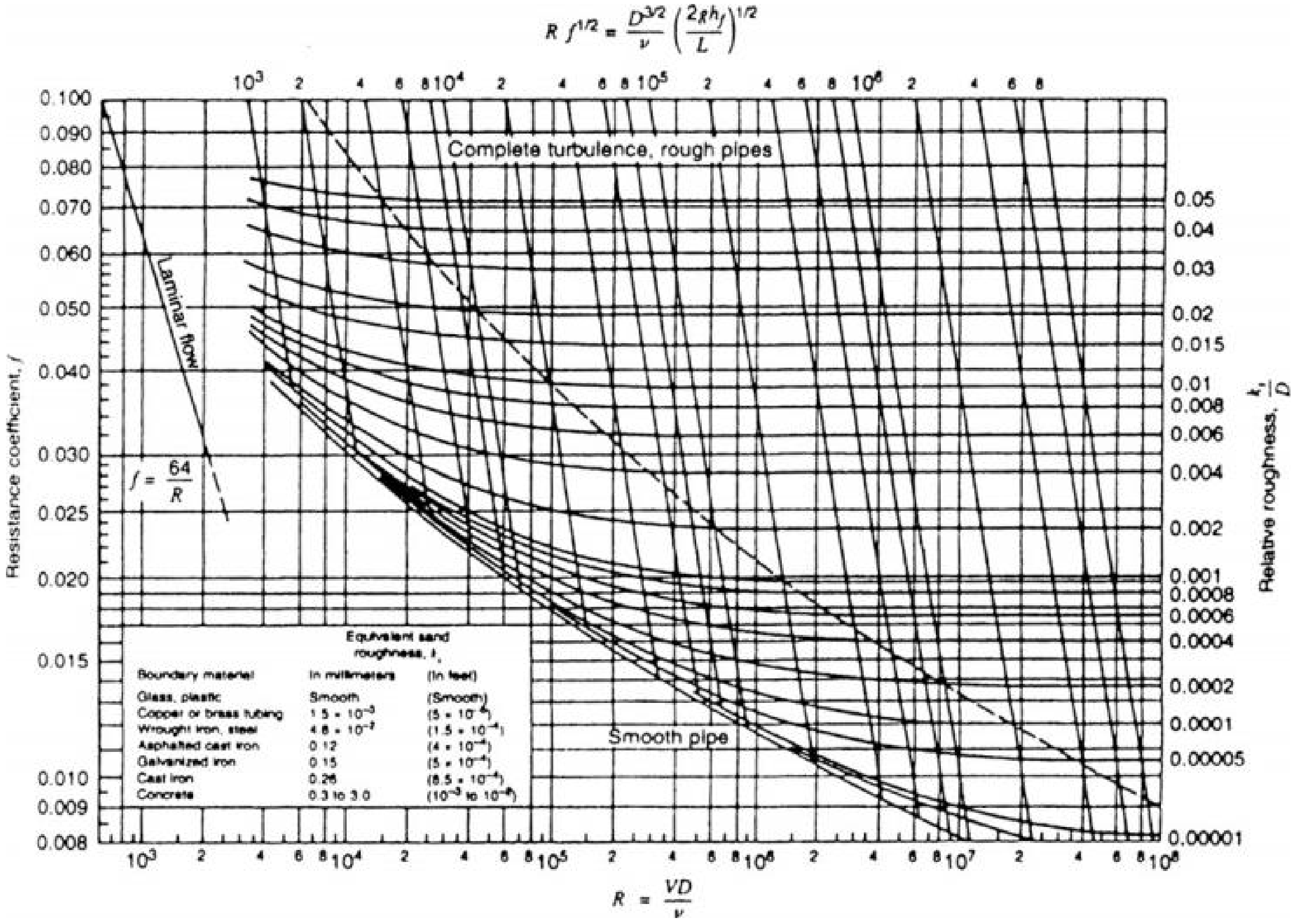


Figure 5.11 Moody diagram with the friction factor f, as a function of Reynolds number R, in Darcy–Weisbach formula for flow in conduits.

where h_f is the head loss in ft (m) (energy loss because of surface resistance) in a pipe of length L in ft (m) and diameter d in ft (m) through which a fluid is transported at a mean velocity v in ft/s (m/s) and flow rate Q in ft^3/s (m^3/s); g is the acceleration of gravity, 32.2 ft/s^2 (9.81 m/s^2); f is a dimensionless friction factor (see Fig. 5.11); and $K = 8fL/\pi^2 gd^5$. In the more than 100 years of its existence, use, and study, this formulation has been foremost in the minds of engineers concerned with the transmission of water as well as other fluids. That this has often been so in a conceptual rather than a practical sense does not detract from its importance.

Within Eq. (5.10a), the dimensionless friction factor f is both its strength and its weakness in applications—its strength as a function of the *Reynolds number R*,

$$\mathbf{R} = vd\rho/\mu = vd/\upsilon \qquad \textbf{(5.11)}$$

where μ is the absolute viscosity, $\upsilon = \mu/\rho$ is the kinematic viscosity of the fluid, and ρ is its density; its weakness as a function of relative roughness ε/r, where ε is a measure of *absolute roughness* and r is the inside radius of the pipe ($2\varepsilon/d = \varepsilon/r$). The f:R relationship is shown in Fig. 5.11, a general resistance diagram for flow in uniform conduits. This diagram evolves from a logarithmic plot of $1/\sqrt{f}$ against $\mathbf{R}/\sqrt{f}$ with scales for f and R added for convenience in finding f for use in Eq. (5.10a).

In reference to R and ε/r,:

1. Laminar flow persists until $R = 2{,}000$, and the f:R relationship is quite simply as follows:

$$f = 64/\mathbf{R} \qquad \textbf{(5.12a)}$$

   Reynolds number R and friction factor f are dimensionless.

2. Above $R = 4{,}000$, turbulent flow is fully established, and the single trace for laminar flow branches into a family of curves for increasing values of ε/r above a lower boundary that identifies the f:R relationship for smooth pipes as

$$1/\sqrt{f} = 2\log \mathbf{R}\sqrt{f} - 0.8 \qquad \textbf{(5.13)}$$

3. For rough pipes, the relative roughness ε/r takes command and

$$1/\sqrt{f} = 2\log r/\varepsilon + 1.74 \qquad \textbf{(5.14)}$$

   where r is the radius of the pipe (ft or m) and ε/r is the relative roughness, dimensionless.

4. In the critical zone between $\mathbf{R} = 2{,}000$ and $\mathbf{R} = 4{,}000$, both $\mathbf{R}$ and ε/r make their appearance in the semiempirical equation of Colebrook and White (1937–1938):

$$1/\sqrt{f} = 1.74 - 2\log(\varepsilon/r + 18.7/R\sqrt{f}) \quad \textbf{(5.15)}$$

The magnitudes of absolute roughness ε in the f:$\mathbf{R}$ and f:(ε/r) relationships depend on the angularity, height, and other geometrical properties of the roughness element and its distribution. Common magnitudes of ε/r have been evaluated for large pipes by Bradley and Thompson (1951).

Despite the logic and inherent conceptual simplicity of the combination of friction-factor diagram and Weisbach formulation, there are important reasons why water engineers do not make use of them for the routine solution of fluid transport problems encountered in water transmission lines and pipe networks. Among the reasons are the following:

1. Because the relative roughness ε/r is a key to f, it is not possible to find r (or d) directly when h_f, v or Q, ε, and water temperature (or υ) are given. A trial-and-error solution is required.
2. Because transmission lines may include noncircular as well as circular conduits, additional f:$\mathbf{R}$ diagrams are needed.
3. Because entry 2 also often applies to partially filled sections, additional diagrams are also necessary for them. For such sections, moreover, trial-and-error solutions must be performed whenever the depth of flow is unknown.

EXAMPLE 5.15 HEAD LOSS FOR LAMINAR FLOW

This example introduces Poiseuille's equation for laminar flow when Reynolds number R is smaller than or equal to 2,000.

$$h_f = (32\mu L v)/(\gamma d^2) = (32\upsilon L v)/(g d^2) \quad \textbf{(5.12b)}$$

where

h_f = head loss, ft, m
μ = absolute viscosity
d = pipe diameter, ft, m
γ = specific weight, 62.4 lb/ft^3, 9.8 kN/m^3 for water
L = pipe length, ft, m
v = average velocity, ft/s, m/s
$\upsilon = \mu/\rho$ = kinematic viscosity
g = acceleration due to gravity, 32.2 ft/s^2, 9.81 m/s^2

Develop Eqs. (5.10a) and (5.12a) using Poiseuille's equation (Eq. 5.12b) shown above.

Solution:

$h_f = (32\mu L v)/(\gamma d^2) = (32\upsilon L v)/(g d^2) = (32\upsilon L v)/(g d^2)(2v/2v)$.
$h_f = (64\upsilon/vd)(L/d)(v^2/2g) = (64/\mathbf{R})(L/d)(v^2/2g) = f(L/d)(v^2/2g)$.
$h_f = f(L/d)(v^2/2g)$, which is Eq. (5.10a).
$f = 64/R$, which is Eq. (5.12a).

f and R are the resistance coefficient and the Reynolds number, respectively.

EXAMPLE 5.16 VELOCITY, REYNOLDS NUMBER, AND HEAD LOSS UNDER LAMINAR FLOW CONDITIONS

The elevation of reservoir A's water surface is 210 m (688.98 ft), while the elevation of reservoir B's water surface is 200 m (656.17 ft). The two reservoirs are connected by a pipe which is 610 m (2,001.31 ft) long and 5 mm (0.0164 ft) in diameter. Assume (a) water flows from reservoir A to reservoir B under laminar flow conditions; (b) $\gamma = 9.8$ kN/m^3 = 62.4 lb/ft^3; and (c) $\mu = 1.003 \times 10^{-3}$ N-s/m^2 = 2.05×10^{-5} lb-s/ft^2 at 20°C (68°F). Determine the velocity, the Reynolds number, and head loss per unit length.

Solution 1 (US Customary System):

$h_f = 688.98$ ft − 656.17 ft = 32.81 ft.

$d = 0.0164$ ft.

$L = 2{,}001.31$ ft.

$\mu = 2.05 \times 10^{-5}$ lb-s/ft^2 at 68°F.

$\gamma = 62.4$ lb/ft^3

$h_f = (32\mu Lv)/(\gamma d^2)$.

32.81 ft = [32(2.05 × 10^{-5} lb-s/ft^2)(2,001.31 ft)(v ft/s)]/[(62.4 lb/ft^3)(0.0164 ft)^2].

***v* (velocity) = 0.41 ft/s**.

Check the Reynolds number.

$R = vd/\upsilon = vd\rho/\mu = vd\gamma/g\mu$.

R = [(0.41 ft/s)(0.0164 ft)(62.4 lb/ft^3)]/[(32.2 ft/s^2)(2.05 × 10^{-5} lb-s/ft^2)] = **630**.

Flow is laminar because R is below 2,000.

$Q = Av$ = [(0.0164 ft)^2 × 0.785](0.41 ft/s) = 8.66 × 10^{-5} ft^3/s.

$\boldsymbol{H_f}$ = h_f/L = 32.81 ft/2,001.31 ft = **0.0164 ft/ft**.

Solution 2 (SI System):

$h_f = 210$ m − 200 m = 10 m.

$d = 5$ mm = 0.005 m.

$L = 610$ m.

$\mu = 1.003 \times 10^{-3}$ N-s/m^2 at 20°C. (Note: 1 N-s/m^2 = 0.021 lb-s/ft^2)

$\gamma = 9.8$ kN/m^3 = 9.8 × 1,000 N/m^3.

$\rho = 1{,}000$ kg/m^3.

$h_f = (32\mu Lv)/(\gamma d^2)$.

10 m = [32(1.003 × 10^{-3} N-s/m^2)(610 m)(v m/s)]/[(9.8 × 1,000 N/m^3)(0.005 m)^2].

***v* (velocity) = 0.125 m/s**.

Check the Reynolds number.

$R = vd/\upsilon = vd\rho/\mu$.

R = [(0.125 m/s)(0.005 m)(1,000 kg/m^3)]/(1.003 × 10^{-3} N-s/m^2) = **630**.

Flow is laminar because R is below 2,000.

$Q = Av$ = [(0.005 m)^2 × 0.785](0.125 m/s) = 2.5 × 10^{-6} m^3/s.

$\boldsymbol{H_f}$ = h_f/L = 10 m/610 m = **0.0164 m/m**.

EXAMPLE 5.17 HEAD LOSS UNDER TURBULENT FLOW CONDITIONS

Compare Poiseuille's equation with Darcy–Weisbach formula, and discuss their applications for determining the head loss under turbulent and laminar flow conditions.

Solution:

From previous example, it can be seen that Poiseuille's equation (Eq. 5.12b) can be rearranged to have a form

$$h_f = f(L/d)(v^2/2g) \tag{5.10a}$$

in which $f = 64/R$, which is Eq. (5.12a).

Poiseuille's equation is used for hydraulic analyses under laminar flow conditions, $R < 2000$.

Darcy–Weisbach formula has the form

$$h_f = f(L/d)(v^2/2g) \tag{5.10a}$$

in which $f = a/R^b$, and a and b are numerical constants. Darcy–Weisbach formula with $f = a/R^b$ is mainly used for determining the head loss of turbulent flows, $R > 4{,}000$. The condition of the pipe wall causes complex degrees of turbulence and different values of f for the same R. Equations (5.13) and (5.14) show the complexity of f values.

In the critical zone between $R = 2{,}000$ and $R = 4{,}000$, Eq. (5.15) represents the complexity of f values.

A Moody diagram shown in Fig. 5.11 can be used for determination of f values, in turn, for calculation of h_f using Eq. (5.10a), when the pipe roughness, and Reynolds number, are known.

EXAMPLE 5.18 HEAD LOSS DETERMINATION USING MOODY DIAGRAM

A 3,200-ft (975.4-m)-long cast iron pipeline with 12 in. (304.8 mm) diameter carries a water flow at a velocity of 15 ft/s (4.572 m/s) and temperature of 68°F (20°C). Determine the Reynolds number, the resistance coefficient f, and the head loss using Moody diagram.

Solution 1 (US Customary System):

υ = kinematic viscosity of water at 68°F = 1.059×10^{-5} ft^2/s.

$R = vd/\upsilon$ = (15 ft/s)(12/12 ft)/(1.059×10^{-5} ft^2/s) = $\mathbf{1.4 \times 10^6 > 2{,}000}$ **turbulent flow**.

$K_s = 8.5 \times 10^{-4}$ ft for cast iron pipe (from Fig. 5.11)

$K_s/d = 8.5 \times 10^{-4}$ ft/(12/12) ft = 0.0008.

From Moody diagram (Fig. 5.11), $\boldsymbol{f} = \mathbf{0.02}$.

$\boldsymbol{h_f} = f(L/d)(v^2/2g)$ = 0.02(3,200 ft/1 ft)(15 ft/s)^2/[2(32.2 ft/s^2)] = **223.6 ft**.

Solution 2 (SI System):

υ = kinematic viscosity of water at 20°C = 1.007×10^{-6} m^2/s.

$\mathbf{R} = vd/\upsilon$ = (4.572 m/s)(0.3048 m)/(1.007×10^{-6} m^2/s) = $\mathbf{1.4 \times 10^6 > 2{,}000}$ **turbulent flow**.

K_s = 0.26 mm = 0.00026 m for cast iron pipe. (from Fig. 5.11)

K_s/d = 0.00026 m/0.3048 m = 0.00085.

From Moody diagram (Fig. 5.11), $\boldsymbol{f} = \mathbf{0.02}$.

$\boldsymbol{h_f} = f(L/d)(v^2/2g)$ = 0.02(975.4 m/0.3048 m)(4.572 m/s)^2/[2(9.81 m/s^2)] = **68.18 m**.

EXAMPLE 5.19 LAMINAR OR TURBULENT FLOW

Determine the type of flow occurring in a 16 in. (40.64 cm) pipe when (a) water at 60°F (15.55°C) flows at a velocity of 3.8 ft/s (1.1582 m/s); and (b) heavy fuel oil at 60°F (15.55°C) flows at the same velocity. At this temperature, kinematic viscosity is 1.217×10^{-5} ft^2/s (1.1306×10^{-6} m^2/s) for water, and 221×10^{-5} ft^2/s (20.53×10^{-5} m^2/s) for heavy oil.

Solution 1 (US Customary System):

$R = vd/\upsilon$ = (velocity)(diameter)/(kinematic viscosity).

R = (3.8 ft/s)(16/12) ft/(1.217×10^{-5} ft^2/s) = 416,324 > 2,000 for water.

So the water flow is under turbulent flow condition.

R = (3.8 ft/s)(16/12) ft/(221×10^{-5} ft^2/s) = 2,292 > 2,000 for heavy oil.

So the heavy oil is under turbulent flow condition, but very close to laminar flow condition.

Solution 2 (SI System):

$R = vd/\upsilon$ = (velocity)(diameter)/(kinematic viscosity).

$R = (1.1582\ \text{m/s})(0.4064\ \text{m})/(1.1306 \times 10^{-6}\ \text{m}^2\text{/s}) = 416{,}324 > 2{,}000$ for water.

So the water flow is under turbulent flow condition.

$R = (1.1582\ \text{m/s})(0.4064\ \text{m})/(20.53 \times 10^{-5}\ \text{m}^2\text{/s}) = 2{,}292 > 2{,}000$ for heavy oil.

So the heavy oil is under turbulent flow condition, but very close to laminar flow condition.

EXAMPLE 5.20 PIPE SIZING UNDER LAMINAR FLOW CONDITIONS

Determine the pipe diameter for delivering 96 gpm (363.3 L/min) of medium fuel oil at 41°F (5°C), assuming that the kinematic viscosity = $6.55 \times 10^{-5}\ \text{ft}^2\text{/s} = 6.085 \times 10^{-6}\ \text{m}^2\text{/s}$.

Solution 1 (US Customary System):

$Q = Av = (\text{area})(\text{velocity}) = (d^2 \times 0.785\ \text{ft}^2)(v\ \text{ft/s}) = 96\ \text{gpm} = 0.2139\ \text{ft}^3\text{/s}$.

$v = Q/A = (0.2139\ \text{ft}^3\text{/s})/(0.785d^2\ \text{ft}^2) = 0.272/d^2\ \text{ft/s}$.

$R = vd/\upsilon = (\text{velocity})(\text{diameter})/(\text{kinematic viscosity})$.

$R = (0.272/d^2\ \text{ft/s})(d\ \text{ft})/(6.55 \times 10^{-5}\ \text{ft}^2\text{/s}) = 2{,}000$.

Pipe diameter = 2.07 ft.

Solution 2 (SI System):

$Q = Av = (\text{area})(\text{velocity}) = (d^2 \times 0.785\ \text{m}^2)(v\ \text{m/s}) = 363.3\ \text{L/min} = 0.3633/60\ \text{m}^3\text{/s}$.

$v = Q/A = (0.0.3633/60)\ \text{m}^3\text{/s}/(0.785\ d^2\ \text{m}^2) = 0.0077/d^2\ \text{m/s}$.

$R = vd/\upsilon = (\text{velocity})(\text{diameter})/(\text{kinematic viscosity})$.

$R = (0.0077/d^2\ \text{m/s})(d\ \text{m})/(6.085 \times 10^{-6}\ \text{m}^2\text{/s}) = 2{,}000$.

Pipe diameter = 0.63 m.

EXAMPLE 5.21 HEAD LOSS USING DARCY–WEISBACH FORMULA

Oil of absolute viscosity 0.0021 lb-s/ft^2 (0.1 N-s/m^2) and specific gravity 0.851 flows through 10,000 ft (3,048 m) of 12 in. (300 mm) stainless steel pipe at a flow rate of 1.57 ft^3/s (0.0445 m^3/s). Determine the Reynolds number, the friction factor, and the head loss in the pipe.

Solution 1 (US Customary System):

$$\begin{aligned}\text{Velocity, } v &= Q/A \\ &= (1.57\text{ft}^3/\text{s})/[(1\ \text{ft})^2 \times 0.785] \\ &= \mathbf{2\ ft/s}.\end{aligned}$$

$$\begin{aligned}\text{Reynolds number, R} &= vd\rho/\mu = vd\gamma/\mu g \\ &= (2\ \text{ft/s})(1\ \text{ft})(0.851 \times 62.4\ \text{lb/ft}^3)/[(0.0021\ \text{lb-s/ft}^2) \times (32.2\ \text{ft/s}^2)] \\ &= \mathbf{1,570} < 2{,}000, \text{ which indicates a laminar flow.}\end{aligned}$$

$$\begin{aligned}\textbf{Friction factor}, f &= 64/\mathbf{R} \text{ under laminar flow conditions} \\ &= 6/1{,}570 \\ &= \mathbf{0.041}.\end{aligned}$$

$$\begin{aligned}\textbf{Head loss}, \boldsymbol{h}_\mathbf{f} &= f(L/d)(v^2/2g) \\ &= (0.041)(10{,}000\ \text{ft}/1\ \text{ft})(2\ \text{ft/s})^2/(2 \times 32.2\ \text{ft/s}^2) \\ &= \mathbf{25.5\ ft}.\end{aligned}$$

Solution 2 (SI System):

$$\begin{aligned}\text{Velocity, } v &= Q/A\\ &= (0.0445\ \mathrm{m^3/s})/[(0.300\ \mathrm{m})^2 \times 0.785]\\ &= \mathbf{0.62\ m/s}.\end{aligned}$$

$$\begin{aligned}\textbf{Reynolds number}, \mathbf{R} &= v d\rho/\mu\\ &= (0.62\ \mathrm{m/s})(0.300\ \mathrm{m})(0.851 \times 1{,}000\ \mathrm{kg/m^3})/(0.1\ \mathrm{N\text{-}s/m^2})\\ &= \mathbf{1,580} < 2{,}000, \text{ which indicates a laminar flow.}\end{aligned}$$

$$\begin{aligned}\textbf{Friction factor}, f &= 64/\mathbf{R} \text{ under laminar flow conditions}\\ &= 64/1{,}580\\ &= \mathbf{0.041}.\end{aligned}$$

$$\begin{aligned}\textbf{Head loss}, \boldsymbol{h}_\mathbf{f} &= f(L/d)(v^2/2g)\\ &= (0.041)(3{,}048\ \mathrm{m}/0.300\ \mathrm{m})(0.62\ \mathrm{m/s})^2/(2 \times 9.81\ \mathrm{m/s^2})\\ &= \mathbf{8.2\ m}.\end{aligned}$$

EXAMPLE 5.22 RATIO OF HEAD LOSS IN A PIPE TO THAT IN A PERFORATED PIPE

Show that the head loss h_f in a pipe is equal to three times the head loss h_f in a perforated pipe, having the same length, diameter, and friction factor.

Take the flow in the imperforated pipe as Q_0; assume a straight-line variation of the flow Q with distance in the perforated pipe, with $Q = Q_0$ at the inlet of the pipe and $Q = 0$ at the end of the line. Consider only losses to pipe friction, and assume no variation in the value of *f* with a changing *Q*.

Solution:

$$h_\mathrm{f} = f(L/d)(v^2/2g) \tag{5.10a}$$

Since

$$v = Q/A \tag{5.4}$$

then

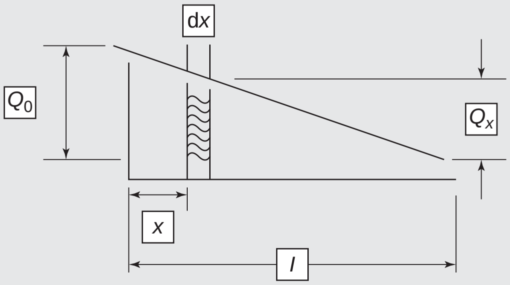
$$h_\mathrm{f} = f(L/D)(Q^2/2gA^2) \tag{5.16}$$

$$h_\mathrm{f} = (f/D)(1/2gA^2)Q^2L \tag{5.17}$$

$$h_\mathrm{f} = KQ^2L$$

where

$$K = (f/D)(1/2gA^2).$$



Taking a small length along the perforated pipe (see sketch) = d*x*, then

$$h_\mathrm{f} = K\int_0^L Q_x^2 \mathrm{d}x.$$

But

$$\frac{Q_x}{Q_0} = \frac{L - x}{L}.$$

$$Q_x = Q_0\frac{L - x}{L}.$$

So

$$h_\mathrm{f} = K\frac{Q_0^2}{L^2}\int_0^L (L - x)^2 \mathrm{d}x.$$

$$h_\mathrm{f} = K\frac{Q_0^2}{L^2}\int\limits_0^L [(L^2 - 2xL + x^2)\mathrm{d}x].$$

$$h_\mathrm{f} = K\frac{Q_0^2}{L^2}\left[L^2x - x^2L + \frac{x^3}{3}\right]_0^L.$$

$$h_\mathrm{f} = K\frac{Q_0^2}{L^2}\int\limits_0^L \left[L^3 - L^3 + \frac{L^3}{3}\right].$$

$$h_\mathrm{f} = KQ_0^2\frac{L}{3}.$$

Hence,

$$h_\mathrm{f} = 1/3(h_\mathrm{f})_0.$$

$$(h_\mathrm{f})_0 = 3h_\mathrm{f}.$$

That is, **the head loss h_f in a pipe is equal to three times the head loss h_f in a perforated pipe**.

EXAMPLE 5.23 HEAD LOSS BETWEEN TWO PRESSURE GAUGES

Water flows in a horizontal pipe of constant cross-sectional area. The pipe has a pressure gauge at location A with a pressure of 120 psig (832.8 kPa) and a pressure gauge at location B with a pressure of 90 psig (624.6 kPa). Determine the head loss between the two pressure gauges.

Solution 1 (US Customary System):

Use the energy equations (5.5a and 5.6b):

$$(P_\mathrm{A}/\gamma + v_\mathrm{A}^2/2g + Z_\mathrm{A}) + H_\mathrm{a} = (P_\mathrm{B}/\gamma + v_\mathrm{B}^2/2g + Z_\mathrm{B}) + h_\mathrm{f}.$$

Here $Z_\mathrm{A} = Z_\mathrm{B}$, and $H_\mathrm{a} = 0$.
Since the pipe size is constant, $v_\mathrm{A}{}^2/2g = v_\mathrm{B}{}^2/2g$.
The above energy equation reduces to

$$\boldsymbol{h_\mathrm{f}} = P_\mathrm{A}/\gamma - P_\mathrm{B}/\gamma = (120\ \mathrm{lb/in.}^2 - 90\ \mathrm{lb/in.}^2)(144\ \mathrm{in.}^2/\mathrm{ft}^2)/(62.4\ \mathrm{lb/ft}^3) = \mathbf{69.23\ ft}.$$

Solution 2 (SI System):

$$1\ \mathrm{kPa} = 1\ \mathrm{kN/m}^2.$$

Use the energy equations (5.5a and 5.6b),

$$(P_\mathrm{A}/\gamma + v_\mathrm{A}^2/2g + Z_\mathrm{A}) + H_\mathrm{a} = (P_\mathrm{B}/\gamma + v_\mathrm{B}^2/2g + Z_\mathrm{B}) + h_\mathrm{f}.$$

Here $Z_\mathrm{A} = Z_\mathrm{B}$, and $H_\mathrm{a} = 0$.
Since the pipe size is constant, $v_\mathrm{A}{}^2/2g = v_\mathrm{B}{}^2/2g$.
The above energy equation reduces to

$$\boldsymbol{h_\mathrm{f}} = P_\mathrm{A}/\gamma - P_\mathrm{B}/\gamma = (832.8\ \mathrm{kN/m}^2 - 624.6\ \mathrm{kN/m}^2)/(9.81\ \mathrm{kN/m}^3) = \mathbf{21.22\ m}.$$

EXAMPLE 5.24 DIFFERENTIAL GAUGE MEASUREMENT

A differential gauge shown in Fig. 5.12 is attached to two cross-sections D and E in a horizontal water pipe in which water is flowing. The deflection of the mercury h in the gauge is 2 ft (0.61 m), the level nearer D being the lower one. Determine the difference in lb/in.^{2} (kN/m^2) between sections D and E.

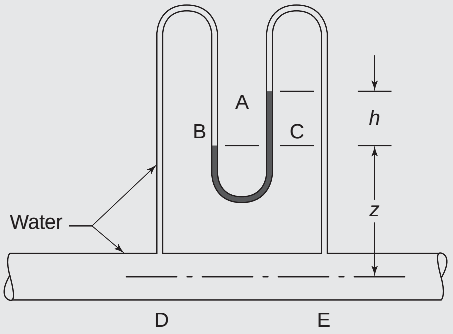


Figure 5.12 Differential gauge attached to a horizontal pipe.

Solution 1 (US Customary System):

Pressure head at B = pressure head at C.

$P_D/\gamma - Z = P_E/\gamma - Z - 2 \text{ ft} + 13.57 \times 2 \text{ ft}.$

$P_D/\gamma - P_E/\gamma =$ **25.4 ft H_2O**.

Or 25.4 × 0.433 = **11 psi**.

Solution 2 (SI System):

Pressure head at B = pressure head at C.

$P_D/\gamma - Z = P_E/\gamma - Z - 0.61 \text{ m} + 13.57 \times 0.61 \text{ m}.$

$P_D/\gamma - P_E/\gamma =$ **7.66 m H_2O**.

7.66 m H_2O = 7.66/0.1013 = **75.69 kN/m^2**.

Note: Since $(P_D/\gamma - P_E/\gamma)$ is positive, the pressure at D is larger than the pressure at E. For more accurate measurment, the diferential gauges should have the air extracted from all tubing before gauge readings are taken.

13.57 is the specific gravity of mercury.

EXAMPLE 5.25 DETERMINATION OF UNBALANCED MOMENT

Gate EBC shown in Fig. 5.13 is hinged at B and is 4.5 ft (1.37 m) wide. Neglecting the weight of the gate, determine the unbalanced moment due to water acting on the gate EBC. It is assumed that angle of the gate is 60°; d = 8.5 ft = 2.59 m; and a = 3.5 ft = 1.07 m.

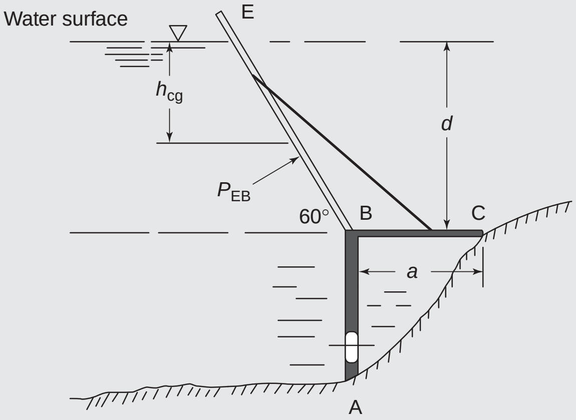


Figure 5.13 Analysis of moment for a hinged gate.

Solution 1 (US Customary System):

$\boldsymbol{P_\mathrm{f}} = (\gamma h_{\mathrm{cg}})A = P_{\mathrm{EB}}$.

EB = 8.5 ft/Sin (60°) = 9.81 ft.

$h_{\mathrm{cg}} = 8.5/2 = 4.25$ ft.

$A = 9.81 \text{ ft} \times 4.5 \text{ ft} = 44.2 \text{ ft}^2$.

$P_{\mathrm{EB}} = (62.4 \text{ lb/ft}^3)(4.25 \text{ ft})(44.2 \text{ ft}^2) = 11{,}722$ lb.

Force $P_{\mathrm{EB}} = 11{,}722$ lb acting at $(2/3) \times 9.81 \text{ ft} = 6.54$ ft from E, and perpendicular to gate EB.

$P_{\mathrm{BC}} = (62.4 \text{ lb/ft}^3)(8.5 \text{ ft})(3.5 \text{ ft} \times 4.5 \text{ ft}) = 8{,}354$ lb, acting at the center of gravity of BC since the pressure on BC is uniform. Taking moment about B (clockwise plus)

$$\begin{aligned}\textbf{Unbalanced moment } \boldsymbol{M} &= 11{,}722 \text{ lb} \times (1/3) \times 9.81 \text{ ft} - 8{,}354 \text{ lb} \times (1/2) \times 3.5 \text{ ft}\\ &= 38{,}331 \text{ lb-ft} - 14{,}620 \text{ lb-ft}\\ &= \textbf{23,711 lb-ft clockwise}.\end{aligned}$$

Solution 2 (SI System):

$\boldsymbol{P_\mathrm{f}} = (\gamma h_{\mathrm{cg}})A = P_{\mathrm{EB}}$.

EB = 2.59 m /Sin (60°) = 2.99 m.

$h_{\mathrm{cg}} = 2.59 \text{ m}/2 = 1.3$ m.

$A = 2.99 \text{ m} \times 1.37 \text{ m} = 4.1 \text{ m}^2$.

$P_{\mathrm{EB}} = (9.81 \text{ kN/m}^3)(1.3 \text{ m})(4.1 \text{ m}^2) = 52.23$ kN.

Force $P_{\mathrm{EB}} = 52.23$ kN acting at $(2/3) \times 2.99 \text{ m} = 1.99$ m from E, and perpendicular to gate EB.

$P_{\mathrm{BC}} = (9.81 \text{ kN/m}^3)(2.59 \text{ m})(1.07 \text{ m} \times 1.37 \text{ m}) = 37.25$ kN, acting at the center of gravity of BC since the pressure on BC is uniform. Taking moment about B (clockwise plus)

$$\begin{aligned}\textbf{Unbalanced moment } \boldsymbol{M} &= 52.23 \text{ kN} \times (1/3) \times 2.99 \text{ m} - 37.25 \text{ kN} \times (1/2) \times 1.07 \text{ m}\\ &= (52.05 - 19.93) \text{ kN-m}\\ &= \textbf{32.12 kN-m clockwise}.\end{aligned}$$

Note: 1 kN-m = 737.6 lb-ft.

EXAMPLE 5.26 HYDRAULIC GRADE LINE AND ENERGY GRADE LINE

In Fig. 5.14, water flows from A to B at a flow rate of 15 ft^3/s (0.4245 m^3/s) and the pressure head at A is 25 ft (7.62 m). Determine the pressure head at B assuming no loss of energy from A to B, d_A = 12 in. (30.48 cm), d_B = 24 in. (60.96 cm), Z_A = 10 ft (3.048 m), Z_B = 25 ft (7.62 m).

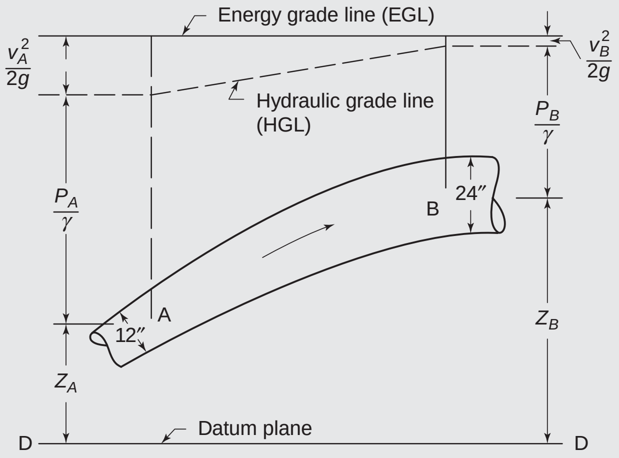


Figure 5.14 Hydraulic and energy grade lines. Conversion factor: 1″ = i in. = 2.54 cm.

Solution 1 (US Customary System):

$Q = (A_A)(v_A) = (A_B)(v_B)$.

15 $\text{ft}^3/\text{s} = [(12/12)^2 \times 0.785\ \text{ft}^2](v_A\ \text{ft/s}) = [(24/12)^2 \times 0.785\ \text{ft}^2](v_B\ \text{ft/s})$.

$v_A = 19.1$ ft/s.

$v_B = 4.78$ ft/s.

$(v_A)^2/2g = (19.1\ \text{ft/s})^2/2 \times 32.2\ \text{ft/s}^2 = 5.6$ ft.

$(v_B)^2/2g = (4.78\ \text{ft/s})^2/2 \times 32.2\ \text{ft/s}^2 = 0.35$ ft.

Apply the Bernoulli equation from A to B.
Use Eq. (5.5a), in which H_a, H_l, and H_e are all zero.

$[P_A/\gamma + (v_A)^2/2g + Z_A] + 0 - 0 - 0 = P_B/\gamma + (v_B)^2/2g + Z_B$.

$[25 + 5.6 + 10] = P_B/\gamma + 0.35 + 25$ assuming datum D–D.

$[25 + 5.6 + 0] = P_B/\gamma + 0.35 + 15$ assuming datum A.

For either datum, we obtain $P_B/\gamma = 15.25$ ft of water.
The total energy at any section can be plotted based on a chosen datum plane. Using D–D in this case,

Energy at A $= P_A/\gamma + (v_A)^2/2g + Z_A = 25 + 5.6 + 10 = 40.6$ ft.

Energy at B $= P_B/\gamma + (v_B)^2/2g + Z_B = 15.25 + 0.35 + 25 =$ **40.6 ft**.

It is noted that transformation from one form of energy to another occurs during flow. In this case a portion of both the pressure energy and the kinetic energy at A is transformed to potential energy at B.

Solution 2 (SI System):

$Q = (A_A)(v_A) = (A_B)(v_B)$.

$0.4245\ \text{m}^3/\text{s} = [(0.3048)^2 \times 0.785\ \text{m}^2](v_A\ \text{m/s}) = [(0.6096)^2 \times 0.785\ \text{m}^2](v_B\ \text{m/s})$.

$v_A = 5.82$ m/s.

$v_B = 1.46$ m/s.

$(v_A)^2/2g = (5.82)^2/2 \times 9.8 = 1.72$ m.

$(v_B)^2/2g = (1.46)^2/2 \times 9.8 = 0.109$ m.

Apply the Bernoulli equation from A to B.
Use Eq. (5.5a), in which H_a, H_l, and H_e are all zero.

$[P_A/\gamma + (v_A)^2/2g + Z_A] + 0 - 0 - 0 = P_B/\gamma + (v_B)^2/2g + Z_B$.

$[7.62 + 1.72 + 3.048] = P_B/\gamma + 0.109 + 7.62$ assuming datum D–D.

$[7.62 + 1.72 + 0] = P_B/\gamma + 0.109 + 4.57$ assuming datum A.

For either datum, we obtain $P_B/\gamma = 4.66$ m of water.
The total energy at any section can be plotted based on a chosen datum plane. Using D–D in this case,

Energy at A $= P_A/\gamma + (v_A)^2/2g + Z_A = 7.62 + 1.72 + 3.048 = 12.39$ m.

Energy at B $= P_B/\gamma + (v_B)^2/2g + Z_B = 4.66 + 0.109 + 7.62 =$ **12.39 m**.

It is noted that transformation from one form of energy to another occurs during flow. In this case a portion of both the pressure energy and the kinetic energy at A is transformed to potential energy at B.

EXAMPLE 5.27 VENTURI METER ANALYSIS

The deflection of the mercury in the differential gauge of a venturi meter is $y = 16.5$ in. (41.91 cm) shown in Fig. 5.15.

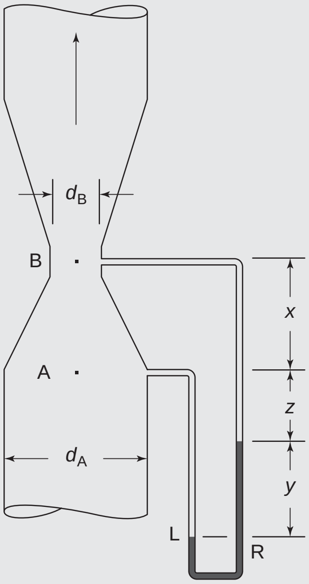


Figure 5.15 Venturi meter analysis for Example 5.27.

Determine the water flow through the venturi meter if no energy is lost between A and B, and the following are known: $d_A = 12$ in. (30.48 cm); $d_B = 6$ in. (15.24 cm); $x = 36$ in. (91.44 cm); $y = 16.5$ in. (41.91 cm); and z = unknown.

Solution 1 (US Customary System):

Apply the Bernoulli equation from A to B and use A as the datum:

$$\frac{P_A}{\gamma} + \frac{v_A^2}{2g} + 0 = \frac{P_B}{\gamma} + \frac{v_B^2}{2g} + \left(\frac{36}{12}\right).$$

$$\frac{P_A}{\gamma} - \frac{P_B}{\gamma} = \frac{v_B^2}{2g} - \frac{v_A^2}{2g} + 3.$$

From continuity equation,

$$Q = A_A v_A = A_B v_B.$$
$$(\pi/4)(12/12)^2 v_A = (\pi/4)(6/12)^2 v_B.$$
$$4v_A = v_B.$$
$$v_A = 0.25\, v_B.$$

Apply the Bernoulli equation for L to R (s.g. mercury, Hg = 13.6):

$$\frac{P_A}{\gamma} + z + \frac{16.5}{12} = \frac{P_B}{\gamma} + \frac{36}{12} + z + \frac{16.5}{12} \times 13.6.$$

$$\frac{P_A}{\gamma} - \frac{P_B}{\gamma} = 20.3 \text{ ft of water.}$$

$$20.3 = \frac{v_B^2}{2g} - \frac{(0.25\, v_B)^2}{2g} + 3.$$

Since $g = 32.2$ ft/s^2

$$v_B = 34.49 \text{ ft/s}.$$

$$\boldsymbol{Q} = A_B v_B = (6/12)^2(\pi/4)(34.49) = \mathbf{6.77\ ft^3/s}.$$

Solution 2 (SI System):

Apply the Bernoulli equation from A to B and use A as the datum:

$$\frac{P_A}{\gamma} + \frac{v_A^2}{2g} + 0 = \frac{P_B}{\gamma} + \frac{v_B^2}{2g} + 0.9144.$$

$$\frac{P_A}{\gamma} - \frac{P_B}{\gamma} = \frac{v_B^2}{2g} - \frac{v_A^2}{2g} + 0.9144.$$

From continuity equation

$$Q = A_A v_A = A_B v_B.$$
$$(\pi/4)(0.3048)^2 v_A = (\pi/4)(0.1524)^2 v_B.$$
$$4v_A = v_B.$$
$$v_A = 0.25\, v_B.$$

Apply the Bernoulli equation for L to R (s.g. mercury, Hg = 13.6):

$$\frac{P_A}{\gamma} + z + 0.4191 = \frac{P_B}{\gamma} + 0.9144 + z + 0.4191 \times 13.6.$$
$$\frac{P_A}{\gamma} - \frac{P_B}{\gamma} = 6.195 \text{ m of water.}$$
$$6.195 = \frac{v_B^2}{2g} - \frac{(0.25v_B)^2}{2g} + 0.9144.$$

Since $g = 9.81$ m/s^2

$$v_B = 10.51 \text{ m/s}.$$
$$\boldsymbol{Q} = A_B v_B = (0.1524)^2(\pi/4)(10.51) = \mathbf{0.192\ m^3/s}.$$

EXAMPLE 5.28 BUOYANCY AND FLOTATION

A body, floating or immersed in a liquid, is acted upon by a buoyant force equal to the weight of the liquid displaced. The buoyancy center is the point through which this buoyant force acts. It is located at the center of gravity of the displaced liquid. If the center of gravity of the body lies directly below the center of buoyancy (gravity) of the displaced liquid, the submerged body is stable. If the two points coincide, the submerged body is in neutral equilibrium position.

Determine the volume of a metal body and its specific gravity if the body weighs 100 lb (45.4 kg) in air and weighs 60 lb (27.2 kg) when immersed in water.

Solution 1 (US Customary System):

Buoyant force = (Weight in air) – (Weight in water)
= 100 lb – 60 lb = 40 lb.

Buoyant force = Weight of displaced liquid.

40 lb = γV = (62.4 lb/ft^3)V.

$\boldsymbol{V} = \mathbf{0.641\ ft^3}$.

Specific gravity = (Weight of metal body)/(Weight of an equal volume of water)
= 100 lb/40 lb
= **2.5**.

Solution 2 (SI System):

Buoyant force = (Weight in air) – (Weight in water)
= 45.4 kg – 27.2 kg = 18.2 kg.

Buoyant force = Weight of displaced liquid.

18.2 kg = γV = (1 kg/L)V.

$\boldsymbol{V} = \mathbf{18.2\ L}$.

Specific gravity = (Weight of metal body)/(Weight of an equal volume of water)
= 45.4 kg/18.2 kg
= 2.5.

EXAMPLE 5.29 EFFECT OF A FLOATING OBJECT ON WATER DEPTH

A floating cylinder 8 cm in diameter and weighing 960 g is placed in a cylindrical container 26 cm in diameter partially full of water. Determine the increase in the depth of water in the container due to placing the float in it.

Solution:

A 960 g cylinder will displace 960 g of water. Since 1 g of water occupies 1 cm^3 volume, the cylinder will displace 960 cm^3 of water.

The change in total volume beneath the water surface ΔV equals the area of the cylindrical container A, times the change in water level Δh, or

$$\begin{aligned}\Delta V &= A\Delta h\\ \Delta h &= \Delta V/A\\ &= 960\ \text{cm}^3/[\pi(26)^2\text{cm}^2]\\ &= \mathbf{1.81\ cm}.\end{aligned}$$

5.2.2 Exponential Equation for Surface Resistance

Because of practical shortcomings of the Weisbach formula, engineers have resorted to the so-called exponential equations in flow calculations. Among them the Chezy formula is the basic for all:

$$v = C\sqrt{rs} \qquad \textbf{(5.18a)}$$

where v = average velocity, ft/s; C = coefficient; r = hydraulic radius, which is defined as the cross-section area divided by the wetted perimeter, ft; and s = slope of water surface or energy gradient:

$$r = A/P_\text{w} \qquad \textbf{(5.19)}$$

$$s = h_\text{f}/L \qquad \textbf{(5.20)}$$

where r = hydraulic radius, ft or m; A = cross-section area, ft^2 or m^2; and P_w = wetted perimeter, ft or m; s = slope of water surface, dimensionless; and L = pipe length, ft or m.

The coefficient C can be obtained by using one of the following expressions:

$$\text{Chezy expression: } C = (8g/f)^{0.5} \qquad \textbf{(5.21)}$$

$$\text{Manning expression: } C = (1.486/n)(r)^{1/6} \qquad \textbf{(5.22)}$$

$$\text{Bazin expression: } C = 157.6/(1 + mr^{-0.5}) \qquad \textbf{(5.23)}$$

$$\text{Kutter expression: } C = (41.65 + 0.00281/s + 1.811/n)/[1 + (n/r^{0.5})(41.65 + 0.00281/s)] \qquad \textbf{(5.24)}$$

In the preceding expressions, f, n, and m are the friction or roughness factors determined by hydraulic experiments using water only. Of the above hydraulic equations, Robert Manning's expression is commonly used for both open channels and closed conduits. Combining Eqs. (5.18a) and (5.22) will give the following Manning formula using US customary units:

$$v = (1.486/n)(r)^{0.67}(s)^{0.5} \quad \text{(US customary units)} \qquad \textbf{(5.25)}$$

where v = velocity, ft/s; n =coefficient of roughness, dimensionless; r = hydraulic radius, ft; and s = slope of energy grade line, ft/ft.

Equation (5.26) is the equivalent Manning formula using SI units:

$$v = (1/n)(r)^{0.67}(s)^{0.5} \quad \text{(SI units)} \qquad \textbf{(5.26)}$$

where v = velocity, m/s; n = coefficient of roughness, dimensionless; r = hydraulic radius, m; and s = slope of energy grade line, m/m.

For a pipe flowing full, the hydraulic radius, Eq. (5.19), becomes

$$r = [(\pi/4)D^2]/(\pi D) = D/4 \qquad \textbf{(5.27)}$$

where r = hydraulic radius, ft or m; and D = pipe diameter, ft or m.

Substituting for r into Eqs. (5.25) and (5.26), the following Manning equations are obtained for practical engineering designs for circular pipes flowing full:

$$v = (0.59/n)(D)^{0.67}(s)^{0.5} \quad \text{(US customary units)} \qquad \textbf{(5.28)}$$

$$Q = (0.46/n)(D)^{2.67}(s)^{0.5} \quad \text{(US customary units)} \qquad \textbf{(5.29)}$$

where Q = flow rate, ft^3/s; v = velocity, ft/s; D = pipe diameter, ft; s = slope of energy grade line, ft/ft; and n = roughness coefficient, dimensionless. For SI measurements:

$$v = (0.40/n)(D)^{0.67}(s)^{0.5} \quad \text{(SI units)} \qquad \textbf{(5.30)}$$

$$Q = (0.31/n)(D)^{2.67}(s)^{0.5} \quad \text{(SI units)} \qquad \textbf{(5.31)}$$

where Q = flow rate, m^3/s; v = velocity, m/s; D = pipe diameter, m; s = slope of energy grade line, m/m; and n = roughness coefficient, dimensionless.

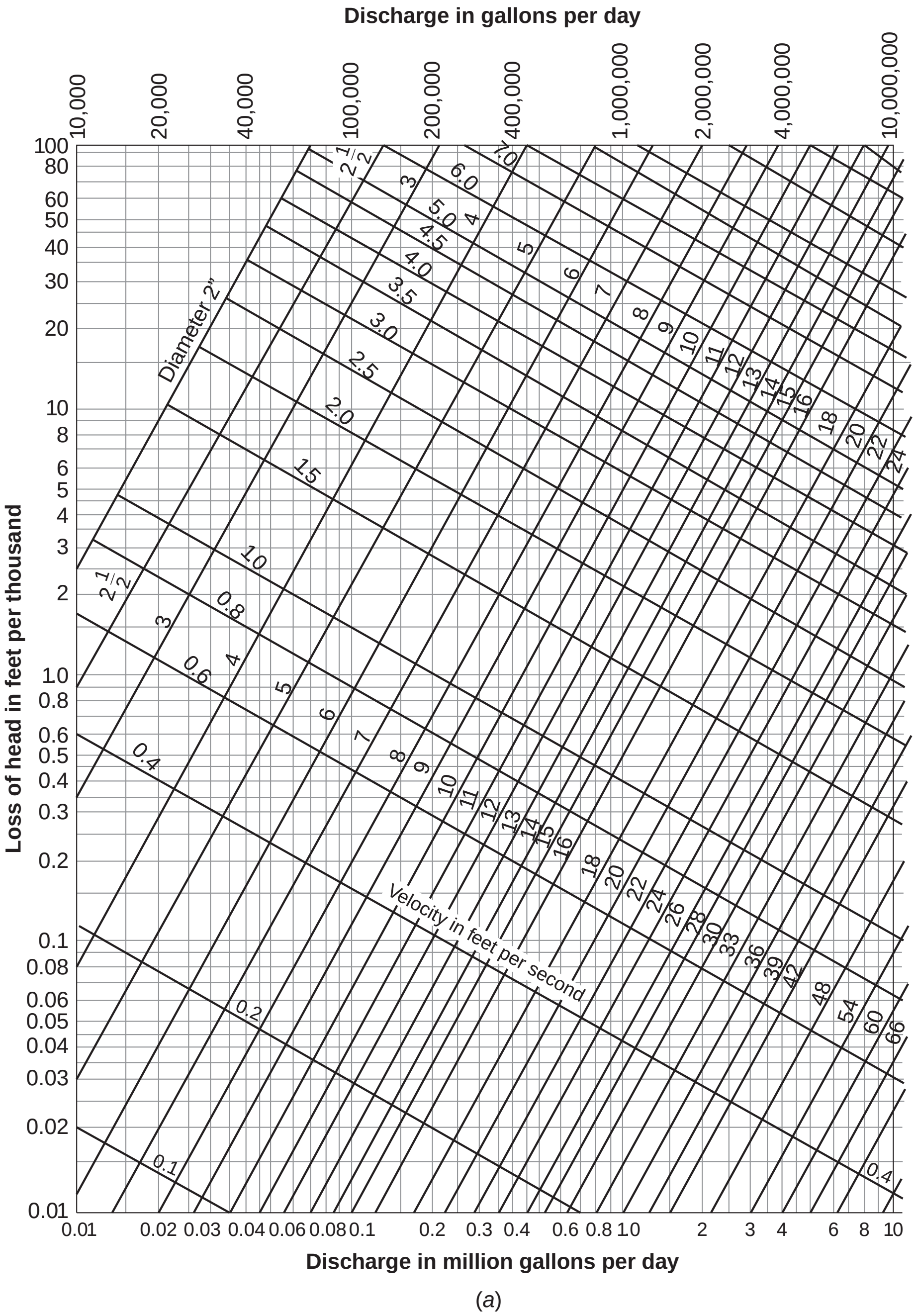
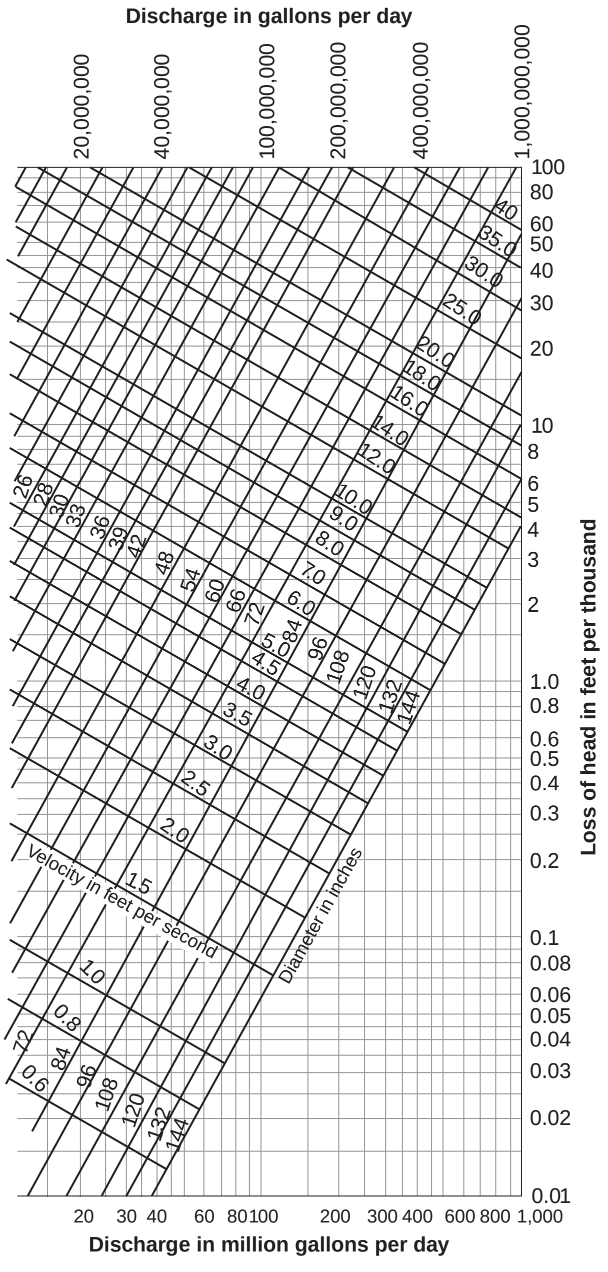


Figure 5.16 (a) Hazen–Williams pipe flow diagram for discharge of 0.01–10 MGD for $C = 100$. Conversion factors: 1 MGD = 3.785 MLD = 43.8 L/s; 1 gpd = 3.785 L/d; 1 ft/1,000 ft = 1 m/1,000 m = 1‰; 1 in. = 25.4 mm; 1 ft/s = 0.3048 m/s.

The *Hazen–Williams formula* is most widely used in the United States to express flow relations in pressure conduits or conduits flowing full, the *Manning formula* in free-flow conduits or conduits not flowing full. The Hazen–Williams formula, which was proposed in 1905, is discussed in another author's sister book entitled *Water Supply and Wastewater Removal*, 3rd ed., John Wiley & Sons (2011).

The following notation is used for the US customary units: Q = rate of discharge, gpm, gpd, MGD, or ft^3/s as needed; d = diameter of small circular conduits, in.; D = diameter of large circular conduits, ft; v = mean velocity, ft/s; $a = \pi D^2/4 = \pi d^2/576$ = cross-sectional area of conduit, ft^2; $r = a/\text{wetted perimeter} = D/4 = d/48$ = hydraulic radius, ft; h_f = loss of head, ft; L = conduit length, ft; and $s = h_\text{f}/L$ = hydraulic gradient, dimensionless.

There is another set of notation to be used for the SI units: Q = rate of discharge, m^3/s; d = diameter of small circular conduits, mm; D = diameter of large circular conduits, m; v = mean velocity, m/s; a = cross-sectional area of conduit, m^2; r = hydraulic radius, m; h_f = loss of head, m; L = conduit length, m; and $s = h_\text{f}/L$ = hydraulic gradient, dimensionless.



(*b*)

Figure 5.16 (b) Hazen–Williams pipe flow diagram for discharge of 10–1,000 MGD for $C = 100$. Conversion factors: 1 MGD = 3.785 MLD = 43.8 L/s; 1 gpd = 3.785 L/d; 1 ft/1,000 ft = 1 m/1,000 m = 1 ‰; 1 in. = 25.4 mm; 1 ft/s = 0.3048 m/s.

As written by the authors, the Hazen–Williams formula is

$$v = Cr^{0.63}s^{0.54}(0.001^{-0.04})$$

$$v = 1.318\ Cr^{0.63}s^{0.54} \quad \text{(US customary units)} \quad \textbf{(5.32)}$$

$$v = 0.849\ Cr^{0.63}s^{0.54} \quad \text{(SI units)} \quad \textbf{(5.33)}$$

where C is a coefficient known as the Hazen–Williams coefficient, and the factor $(0.001^{-0.04}) = 1.318$ makes C conform in general magnitude with established values of a similar coefficient in the more-than-a-century-older Chezy formula.

For circular conduits, the Hazen–Williams formulation can take one of the following forms:

$$v = 0.115\ Cd^{0.63}s^{0.54} \quad \text{(US customary units)} \quad \textbf{(5.34)}$$

$$v = 0.550\ CD^{0.63}s^{0.54} \quad \text{(US customary units)}$$

$$v = 0.3545\ CD^{0.63}s^{0.54} \quad \text{(SI units)}$$

$$h_f = 5.47\ (v/C)^{1.85}L/d^{1.17} \quad \text{(US customary units)} \quad \textbf{(5.35)}$$

$$h_f = 3.02\ (v/C)^{1.85}L/D^{1.17} \quad \text{(US customary units)}$$

$$h_f = 6.81\ (v/C)^{1.85}L/D^{1.17} \quad \text{(SI units)}$$

$$Q_{\text{gpd}} = 405\ Cd^{2.63}s^{0.54} \quad \text{(US customary units)} \quad \textbf{(5.36)}$$

$$Q_{\text{MGD}} = 0.279\ CD^{2.63}s^{0.54} \quad \text{(US customary units)}$$

$$Q_{\text{ft}^3/\text{s}} = 0.432\ CD^{2.63}s^{0.54} \quad \text{(US customary units)}$$

$$Q_{\text{m}^3/\text{s}} = 0.278\ CD^{2.63}s^{0.54} \quad \text{(SI units)}$$

$$h_f = 1.50 \times 10^{-5}(Q_{\text{gpd}}/C)^{1.85}L/d^{4.87} \quad \text{(US customary units)} \quad \textbf{(5.37)}$$

$$h_f = 10.6\ (Q_{\text{MGD}}/C)^{1.85}L/D^{4.87} \quad \text{(US customary units)}$$

$$h_f = 4.72\ (Q_{\text{ft}^3/\text{s}}/C)^{1.85}L/D^{4.87} \quad \text{(US customary units)}$$

$$h_f = 10.67\ (Q_{\text{m}^3/\text{s}}/C)^{1.85}L/D^{4.87} \quad \text{(SI units)}$$

$$h_f = KQ^{1.85} \quad \text{(US customary units or SI units)}$$

Note that in this Hazen–Williams relationship, the head loss is proportional to the flow raised to the power of 1.85, whereas in Darcy–Weisbach relationship (Eq. 5.10b) the head loss is proportional to the square of the flow.

Solutions of Eqs. (5.32) and (5.34)–(5.37) for Q, v, r, D, d, s, h_f, L, or C requires the use of logarithms, a log–log slide rule, tables, a diagram with logarithmic scales (see Figs. 5.16a and 5.16b), an alignment chart (Fig. 5.17 and Appendix 14) or a computer program. For C other than 100, one can multiply given Q or v in Fig. 5.17 by (100/C) to find s, or can multiply found value of Q or v in Fig. 5.17 by C/100 for given s. Use of the Hazen–Williams pipe flow diagram (Fig. 5.16) is explained in Fig. 5.16c. The weakest element

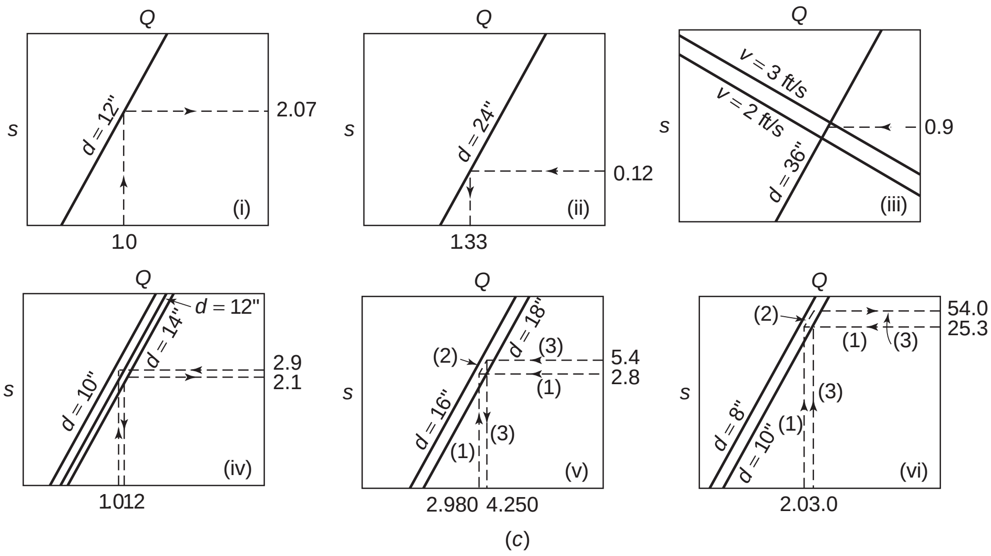


Figure 5.16 (c) Use of Hazen–Williams pipe flow diagram (a and b): (i) given *Q* and *d*; to find *s*; (ii) given *d* and *s*; to find *Q*; (iii) given *d* and *s*; to find *v*; (iv) given *Q* and *s*; to find *d*; (v) given *Q* and *h*; to find *Q* for different *h*; (vi) given *Q* and *h*; to find *h* for different *Q*. For *C* other than 100:

- Multiply given *Q* or *v* by (100/*C*) to find *s*.
- Multiply found value of *Q* or *v* by (*C*/100) for given *s*.

Conversion factors: 1″ = 1 in. = 25.4 mm; 1 ft/s = 0.3408 m/s.

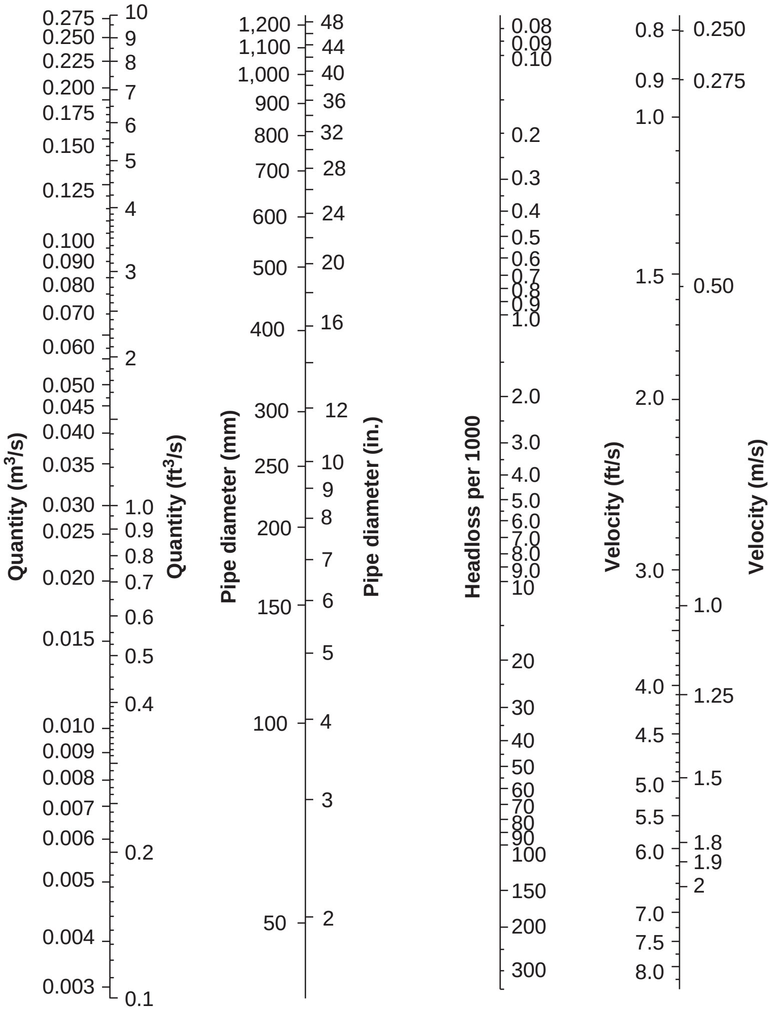


Figure 5.17 Nomogram for solving the Hazen–Williams equation ($C = 100$).

in the Hazen–Williams formula is the estimate of C in the absence of measurements of loss of head and discharge or velocity. Values of C vary for different conduit materials and their relative deterioration in service. They vary somewhat also with size and shape. The values listed in Table 5.1 reflect more or less general experience.

For purposes of comparison, the size of a noncircular conduit can be stated in terms of the diameter of a circular conduit of equal carrying capacity. For identical values of C and s, multiplication of Eq. (5.32) by the conduit area a and equating the resulting expression to Eq. (5.35), the diameter of the equivalent conduit becomes

$$D = 1.53\, a^{0.38} r^{0.24} \tag{5.38}$$

Equation (5.38) is applicable to both the US customary system and the SI system. The units for the above equation are D (ft or m), a (ft^2 or m^2), and r (ft or m).

Table 5.1 Values of the Hazen–Williams coefficient, C, for different conduit materials and age of conduit

	Age	
Conduit material	New	Uncertain
Cast iron pipe, coated (inside and outside)	130	100
Cast iron pipe, lined with cement or bituminous enamel	130[a]	130[a]
Steel, riveted joints, coated	110	90
Steel, welded joints, coated	140	100
Steel, welded joints, lined with cement or bituminous enamel	140[a]	130[a]
Concrete	140	130[a]
Wood stave	130	130
Cement-asbestos and plastic pipe	140	130

[a] For use with the nominal diameter, that is, the diameter of unlined pipe.

EXAMPLE 5.30 METRIC OPEN CHANNEL EQUATIONS

Compare the US customary system's open channel equations with the SI system's equations.

Solution:

Chezy formula, in terms of the US customary units, is presented below:

$$v = C\sqrt{rs} \tag{5.18a}$$

where v = average velocity, ft/s; C = coefficient, dimensionless; r = hydraulic radius, ft; and s = slope of water surface or energy gradient, dimensionless.

The equivalent Chezy formula using the SI system is

$$v = 0.552\, C\sqrt{rs} \tag{5.18b}$$

where v = average velocity, m/s; C = coefficient, dimensionless; r = hydraulic radius, m; and s = slope of water surface or energy gradient, dimensionless.

Substituting Eq. (5.22) into Eq. (5.18a), one obtains the Manning's US customary equation:

$$v = \frac{1.486}{n} (r)^{0.67} (s)^{0.5} \tag{5.25}$$

Its equivalent SI equation is

$$v = \frac{1}{n} (r)^{0.67} (s)^{0.5} \tag{5.26}$$

where v = average velocity, m/s; C = coefficient, dimensionless; r = hydraulic radius, m; and s = slope of water surface or energy gradient, dimensionless.

EXAMPLE 5.31 ROUGHNESS COEFFICIENT

Discuss the applications of the roughness coefficient. What is the roughness coefficient for an average cement-lined open channel?

Solution:

The following are the average values of the roughness coefficient n, for use in the application of the Kutter's and Manning's equations:

0.010	Smooth cement lining; best planed timber
0.012	New wood flumes; lined cast iron
0.013	Vitrified sewer pipe; concrete pipe; metal flumes
0.015	Average clay pipe; average cast iron pipe; average cement-lined pipe
0.023	Well-maintained earth canals
0.027	Dredged earth canals
0.040	Canals cut in rocks
0.030	Rivers in good condition

Therefore, selection of $n = 0.015$ for an average cement-lined open channel appears to be appropriate.

EXAMPLE 5.32 OPEN CHANNEL FLOW

Determine the expected flow in a 4.2-ft (1.28-m)-wide rectangular cement-lined open channel. The channel is laid on a slope of 4.8 ft in 10,000 ft (4.8 m in 10,000 m) and water flows 2.2 ft (0.67 m) deep. Use the Kutter's C assuming the roughness coefficient n equals 0.015.

Solution 1 (US Customary System)

$$v = C\sqrt{rs} \tag{5.18a}$$

$r = A/P_w = (4.2 \text{ ft} \times 2.2 \text{ ft})/(2.2 \text{ ft} \times 2 + 4.2 \text{ ft}) = 1.074$ ft.
$s = 4.8 \text{ ft}/10{,}000 \text{ ft} = 0.00048$.

$$C = (41.65 + 0.00281/s + 1.811/n)/[1 + (n/r^{0.5})(41.65 + 0.00281/s)] \tag{5.24}$$

$C = (41.65 + 0.00281/0.00048 + 1.811/0.015)/[1 + (0.015/1.074^{0.5})(41.65 + 0.00281/0.00048)]$.
$C = 100$.

$$\begin{aligned} Q &= Av = AC\sqrt{rs} \\ &= (4.2 \times 2.2)(100)\sqrt{1.074 \times 0.00048} \\ &= \mathbf{21.0\ ft^3/s}. \end{aligned}$$

Solution 2 (SI System):

$$v = 0.552C\sqrt{rs} \tag{5.18b}$$

$r = A/P_w = (1.28 \text{ m} \times 0.67 \text{ m})/(0.67 \text{ m} \times 2 + 1.28 \text{ m}) = 0.327$ m.
$s = 4.8 \text{ m}/10{,}000 \text{ m} = 0.00048$.
From Solution 1, $C = 100$.

$$\begin{aligned} Q &= Av = 0.552AC\sqrt{rs} \\ &= 0.552(1.28 \times 0.67)(100)\sqrt{0.327 \times 0.00048} \\ &= \mathbf{0.59\ m^3/s}. \end{aligned}$$

EXAMPLE 5.33 OPEN CHANNEL FIELD INVESTIGATION—KUTTER METHOD

A water flow of 21.0 ft^3/s (0.59 m^3/s) was measured in a rectangular open channel 4.2 ft (1.28 m) wide with water flowing 2.2 ft (0.67 m) deep. Determine the roughness factor n for the channel lining, if the channel slope is 0.00048. Use the Kutter's method.

Solution 1 (US Customary System):

$$v = C\sqrt{rs} \tag{5.18a}$$

$$r = A/P_w = (4.2 \text{ ft} \times 2.2 \text{ ft})/(2.2 \text{ ft} \times 2 + 4.2 \text{ ft}) = 1.074 \text{ ft}.$$

$$Q = Av = AC\sqrt{rs}.$$

$$21.0 = (4.2 \times 2.2)(C)\sqrt{1.074 \times 0.00048}.$$

$$C = (21.0)/(4.2 \times 2.2)\sqrt{1.074 \times 0.00048}$$
$$= 100.$$

Substitute $C = 100$, $r = 1.074$ ft, and $s = 0.00048$ into Eq. (5.24):

$$C = (41.65 + 0.00281/s + 1.811/n)/[1 + (n/r^{0.5})(41.65 + 0.00281/s)] \quad \textbf{(5.24)}$$

$$100 = (41.65 + 0.00281/0.00048 + 1.811/n)/[1 + (n/1.074^{0.5})(41.65 + 0.000281/0.00048)].$$

By the trial-and-error method, assuming $n = 0.011, 0.013, 0.015, 0.017$, and so on, ***n* is found to be 0.015**.

Solution 2 (SI System):

$$v = 0.552AC\sqrt{rs} \quad \textbf{(5.18b)}$$

$$r = A/P_w = (1.28 \text{ m} \times 0.67 \text{ m})/(0.67 \text{ m} \times 2 + 1.28 \text{ m}) = 0.327 \text{ m } (1.074 \text{ ft}).$$

$$Q = Av = 0.552AC\sqrt{rs}.$$

$$0.59 = 0.552(1.28 \times 0.67)(C)\sqrt{0.327 \times 0.00048}.$$

$$C = 100.$$

Substitute $C = 100$, $r = 1.074$, and $s = 0.00048$ into Eq. (5.24):

$$C = (41.65 + 0.00281/s + 1.811/n)/[1 + (n/r^{0.5})(41.65 + 0.00281/s)] \quad \textbf{(5.24)}$$

$$100 = (41.65 + 0.00281/0.00048 + 1.811/n)/[1 + (n/1.074^{0.5})(41.65 + 0.000281/0.00048)].$$

By the trial-and-error method, assuming $n = 0.011, 0.013, 0.015, 0.017$, and so on, ***n* is found to be 0.015**.

EXAMPLE 5.34 OPEN CHANNEL FIELD INVESTIGATION—MANNING METHOD

A water flow of 21.0 ft^3/s (0.59 m^3/s) was measured in a rectangular open channel 4.2 ft (1.28 m) wide with water flowing 2.2 ft (0.67 m) deep. Determine the roughness factor *n* for the channel lining, if the channel slope is 0.00048. Use the Manning's method.

Solution 1 (US Customary System):

$$v = \frac{1.486}{n}(r)^{0.67}(s)^{0.5} \quad \textbf{(5.25)}$$

$$A = 4.2 \times 2.2 = 9.24 \text{ ft}^2.$$

$$r = A/P_w = (9.24)/(2.2 \text{ ft} \times 2 + 4.2 \text{ ft}) = 1.074 \text{ ft}.$$

$v = (1.486/n)(1.074)^{0.67}(0.00048)^{0.5}$.

$Q = Av$.

$21.0 = 9.24[(1.486/n)(1.074)^{0.67}(0.00048)^{0.5}]$.

$n = 9.24[(1.486)(1.074)^{0.67}(0.00048)^{0.5}]/21.0$.

$\boldsymbol{n = 0.015}$.

Solution 2 (SI System):

$$v = \frac{1}{n}(r)^{0.67}(s)^{0.5} \quad \textbf{(5.26)}$$

$$A = 1.28 \times 0.67 = 0.858.$$

$$r = A/P_w = (0.858\ \text{m}^2)/(0.67\ \text{m} \times 2 + 1.28\ \text{m}) = 0.327\ \text{m}.$$

$v = (1/n)(r)^{0.67}(s)^{0.5}$

$Q = Av.$

$0.59 = (0.858)[(1/n)(0.327)^{0.67}(0.00048)^{0.5}].$

$n = (0.858)[(0.327)^{0.67}(0.00048)^{0.5}]/0.59.$

$\boldsymbol{n = 0.015}$.

EXAMPLE 5.35 APPLICATION OF MANNING FORMULA FOR A PIPE FLOWING FULL

Determine the velocity and flow of a circular pipe 24 in. (061 m) in diameter, which is flowing full. The pipe roughness coefficient is 0.013 and the slope of its energy line is $s = 0.0004$.

Solution 1 (US Customary System):

For a pipe flowing full $r = A/P_w = (\pi D^2/4)/(\pi D) = D/4$.

$$v = (0.59/n)(D)^{0.67}(s)^{0.5} \quad \textbf{(5.28)}$$

$$v = (0.59/0.013)(24/12)^{0.67}(0.0004)^{0.5}.$$

$$v = 1.44\ \text{ft}/s.$$

$$Q = (0.46/n)(D)^{2.67}(s)^{0.5} \quad \textbf{(5.29)}$$

$$Q = (0.46/0.013)(24/12)^{2.67}(0.0004)^{0.5}.$$

$$\boldsymbol{Q = 4.5\ \text{ft}^3/\text{s} = 2{,}020\ \text{gpm}}.$$

Using the nomogram (Appendix 15), for $n = 0.013$:

$$\boldsymbol{v = 1.45\ \text{ft}/\text{s}}.$$

$$\boldsymbol{Q = 2{,}000\ \text{gpm}}.$$

Solution 2 (SI System):

$$v = (0.40/n)(D)^{0.67}(s)^{0.5} \quad \textbf{(5.30)}$$

$$v = (0.40/0.013)(0.61)^{0.67}(0.0004)^{0.5}.$$

$$\boldsymbol{v = 0.44\ \text{m}/\text{s}}.$$

$$Q = (0.31/n)(D)^{2.67}(s)^{0.5} \quad \textbf{(5.31)}$$

$$Q = (0.31/0.013)(0.61)^{2.57}(0.0004)^{0.5}.$$

$$\boldsymbol{Q = 0.127\ \text{m}^3/\text{s}}.$$

Using the nomogram (Appendix 15), for $n = 0.013$:

$$\boldsymbol{v = 0.44\ \text{m}/\text{s}}.$$

$$\boldsymbol{Q = 0.126\ \text{m}^3/\text{s}}.$$

EXAMPLE 5.36 HEAD LOSS DETERMINATION USING HAZEN–WILLIAMS FORMULA

Determine the loss of head in ft/1,000 ft (m/1,000 m) for a 12 in. (0.3048 m) pipe with a flow of 1 MGD (3.785 MLD) and a C=100. Assume that the pipe is flowing full.

Solution 1 (US Customary System):

Method 1—use Fig. 5.16a.

When $Q = 1$ MGD, $d = 12$ in., $C = 100$

$\boldsymbol{h_f}$ **= 2.07 ft/1,000 ft**.

Method 2—use Fig. 5.17.

When $Q = 1$ MGD $= 1.547$ ft^3/s, $d = 12$ in., $C = 100$

$\boldsymbol{h_f}$ **= 2.07 ft/1,000 ft**.

Method 3—use Appendix 14.

When $Q = 1$ MGD $= 684.4$ gpm, $d = 12$ in., $C = 100$

$\boldsymbol{h_f}$ **= 2.07 ft/1,000 ft**.

Method 4—use Eq. (5.37).

$$h_f = 10.6\,(Q_{\text{MGD}}/C)^{1.85}L/D^{4.87} \tag{5.37}$$

$$h_f = 10.6(1/100)^{1.85}1{,}000/(12/12)^{4.87}.$$
s **= 2.07 ft/1,000 ft**.

Solution 2 (SI System):

Method 1—use Fig. 5.16a.

When $Q = 3.785$ MLD, $D = 0.3048$ m, $C = 100$, after all metric units are converted to the US customary units

$\boldsymbol{h_f}$ = 2.07 ft/1,000 ft = **2.07 m/1,000 m**.

Method 2—use Fig. 5.17.

Read Solution 1 after SI units are converted to the US customary units.

$\boldsymbol{h_f}$ **= 2.07 m/1,000 m**.

Method 3—use Appendix 14.

When $Q = 0.0438$ m^3/s = 43.8 L/s, $D = 0.3048$ m = 30.48 cm, and $C = 100$

$\boldsymbol{h_f}$ **= 2.07 m/1,000 m**.

Method 4—use Eq. (5.37).

where $Q = 0.0438$ m^3/s, $D = 0.3048$ m, $L = 1{,}000$ m and $C = 100$

$$h_f = 10.67\,(Q_{\text{m}^3/\text{s}}/C)^{1.85}L/D^{4.87} \tag{5.37}$$

$$h_f = 10.67(0.0438/100)^{1.85}1{,}000/(0.3048)^{4.87}.$$
s **= 2.17 m/1,000 m**.

EXAMPLE 5.37 MATHEMATICAL AND GRAPHICAL BASIS OF THE PIPE FLOW DIAGRAM

The pipe flow diagram (Fig. 5.16) establishes the numerical relationships between Q, v, d, and s for a value of $C = 100$. Conversion to other magnitudes of C is simple because both v and Q vary directly as C. Show the mathematical and graphical basis of this diagram.

Solution:

1. Written in logarithmic form, Eq. (5.36), $Q_{\text{gpd}} = 405Cd^{2.63}s^{0.54}$, is
   (a) $\log Q_{\text{gpd}} = 4.61 + 2.63\,\log d + 0.54\,\log s$, or
   (b) $\log s = -8.54 - 4.87\,\log d + 1.85\,\log Q_{\text{gpd}}$.

A family of straight lines of equal slope is obtained, therefore, when s is plotted against Q on log–log paper for specified diameters d. Two points define each line. Pairs of coordinates for a 12 in. pipe, for example, are as follows:

(c) $Q = 100{,}000$ gpd, $s = 0.028$ ft/1,000 ft.

(d) $Q = 1{,}000{,}000$ gpd, $s = 2.05$ ft/1,000 ft.

2. Written in logarithmic form, Eq. (5.34), $v = 0.115Cd^{0.63}s^{0.54}$, is as follows:

$\log v = 1.0607 + 0.63 \log d + 0.54 \log s$

If the diameter d is eliminated from the logarithmic transforms of Eqs. (5.36) and (5.34), then

(a) $\log Q = 0.180 + 4.17 \log v - 1.71 \log s$.

(b) $\log s = 0.105 + 2.43 \log v - 0.585 \log Q$.

3. A family of straight lines of equal slope is obtained when s is plotted against Q on log–log paper for specified velocities v. Two points define each line. Pairs of coordinates for a velocity of 1 ft/s, for example, are as follows:

(a) $Q = 100{,}000$ gpd, $s = 1.5$ ft/1,000 ft.

(b) $Q = 10{,}000{,}000$ gpd, $s = 0.10$ ft/1,000 ft.

EXAMPLE 5.38 CIRCULAR CONDUIT EQUIVALENCE TO A HORSESHOE CONDUIT

A tunnel having a horseshoe shape (Fig. 5.18) has a cross-sectional area of 27.9 ft^2 (2.59 m^2) and a hydraulic radius of 1.36 ft (0.41 m). Find the diameter, hydraulic radius, and area of the hydraulically equivalent circular conduit.

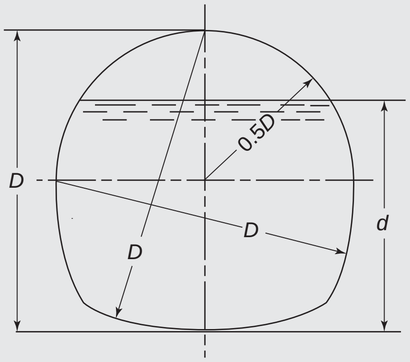


Figure 5.18 Horseshoe conduit section.

Solution 1 (US Customary System):

$$D = 1.53\, a^{0.38} r^{0.24} \tag{5.38}$$

$$D = 1.53 \times (27.9)^{0.38} \times (1.36)^{0.24} = \mathbf{5.85\ ft\ (1.78\ m)}.$$

$$\text{Hydraulic radius } r = D/4 = \mathbf{1.46\ ft\ (0.45\ m)}.$$

$$\text{Area } a = \pi D^2/4 = \mathbf{26.7\ ft^2\ (2.48\ m^2)}.$$

Solution 2 (SI System):

$$D = 1.53\, a^{0.38} r^{0.24}.$$

$$D = 1.53\,(2.59)^{0.38}\,(0.41)^{0.24} = \mathbf{1.78\ m\ (5.85\ ft)}.$$

$$\text{Hydraulic radius } r = D/4 = \mathbf{0.45\ m\ (1.46\ ft)}.$$

$$\text{Area } a = \pi D^2/4 = \mathbf{2.48\ m^2\ (26.7\ ft^2)}.$$

Note that neither the cross-sectional area nor the hydraulic radius of this equivalent circular conduit is the same as that of the horseshoe section proper. Equation (5.38) is applicable to both the US customary system and the SI system.

EXAMPLE 5.39 DETERMINATION OF PIPE DIAMETER USING A NOMOGRAM FOR SOLVING THE HAZEN–WILLIAMS EQUATION

Given the following:

1. If a flow of 210 L/s is to be carried from point A to point B by a 3,300-m ductile iron pipeline ($C = 100$) without exceeding a head loss of 43 m, what must be the pipe diameter?
2. If the elevation of point A is 580 m and the level of point B is 600 m and a minimum residual pressure of 3 bars (30 m of water) must be maintained at point B, what then must be the minimum actual pressure at point A?

Solution:

1. Pipe diameter:

$$C = 100.$$
$$s = h_f/L = 43/3{,}300 = 13\text{‰}.$$
$$Q = 210\ \text{L/s} = 0.210\ \text{m}^{3/s.}$$

From the nomogram of Fig. 5.17, $D = 380$ mm.
Therefore, use a nominal pipe diameter of 400 mm so as not to exceed a head loss of 43 m.

2. Actual pressure at point A:

$$P_A/\gamma + Z_A + (v_A)^2/2g = P_B/\gamma + Z_B + (v_B)^2/2g + h_f.$$

Knowing that

$$v_A = v_B \text{ since no change in D or Q}$$

the nomogram ($C = 100$) of Fig. 5.17 for $D = 400$ mm and $Q = 0.210\ \text{m}^3/\text{s}$ will give $s = 11\text{‰}$.

$$h_f = sL = 11\text{‰} \times 3{,}300\ \text{m} = 36\ \text{m}.$$
$$P_A/\gamma + 580 = 30 + 600 + 36.$$
$$\boldsymbol{P_A/\gamma} = \textbf{86 m or 8.6 bars is the actual pressure at point A.}$$

EXAMPLE 5.40 APPLICATIONS OF HAZEN–WILLIAMS FORMULA

Determine the expected velocity, full flow, and head loss for a 24 in. (609.6 mm) circular conduit with a Hazen–Williams coefficient $C = 100$, a length $L = 1{,}000$ ft (304.8 m), and a hydraulic gradient $s = 2.25\%$.

Solution 1 (US Customary System):

1. According to the nomogram shown in Fig. 5.17,

$$v = 3.3\ \text{ft/s and } Q = 10\ \text{ft}^3/\text{s}.$$
$$s = 0.0025 = h_f/\text{L} = h_f/1{,}000\ \text{ft}.$$
$$h_f = 1{,}000\ \text{ft} \times 0.0025 = 2.5\ \text{ft}.$$

2. Using Eqs. (5.34) and (5.36),

$$v = 0.115Cd^{0.63}s^{0.54} = 0.115 \times 100(24)^{0.63}(0.0025)^{0.54} = 3.3\ \text{ft/s}.$$
$$v = 0.55CD^{0.63}s^{0.54} = 0.55 \times 100(24/12)^{0.63}(0.0025)^{0.54} = \mathbf{3.3\ ft/s}.$$
$$Q_{\text{ft}^3/\text{s}} = 0.432CD^{2.63}s^{0.54} = 0.432 \times 100(24/12)^{2.63}(0.0025)^{0.54} = \mathbf{10.5\ ft^3/s}.$$
$$h_f = 1{,}000\ \text{ft} \times 0.0025 = \mathbf{2.5\ ft}.$$

Solution 2 (SI System):

1. According to the nomogram shown in Fig. 5.17,

$$v = 1\ \text{m/s and Q} = 0.28\ \text{m}^3/\text{s}.$$
$$h_f = 304.8\ \text{m} \times 0.0025 = 0.762\ \text{m}.$$

2. Using Eqs. (5.34) and (5.36),

$$v = 0.3545CD^{0.63}s^{0.54} = 0.3545 \times 100(0.6096)^{0.63}(0.0025)^{0.54} = \mathbf{1\ m/s}$$
$$Q_{\text{m}^3/\text{s}} = 0.278CD^{2.63}s^{0.54} = 0.278 \times 100(0.6096)^{2.63}(0.0025)^{0.54} = \mathbf{0.298\ m^3/s}.$$
$$h_f = 304.8\ \text{m} \times 0.0025 = \mathbf{0.762\ m}.$$

Table 5.2 Values of k for form losses

Fitting	k-value	Fitting	k-value
Pipe entrance		90° smooth bend	
Bellmouth	0.03−0.05	Bend radius / $D = 4$	0.16−0.18
Rounded	0.12−0.25	Bend radius / $D = 2$	0.19−0.25
Sharp edged	0.50	Bend radius / $D = 1$	0.35−0.40
Projecting	0.80	Mitered bend	
Contraction—sudden		$\theta = 15°$	0.05
$D_2/D_1 = 0.80$	0.18	$\theta = 30°$	0.10
$D_2/D_1 = 0.50$	0.37	$\theta = 45°$	0.20
$D_2/D_1 = 0.20$	0.49	$\theta = 60°$	0.35
Contraction—conical		$\theta = 90°$	0.80
$D_2/D_1 = 0.80$	0.05	Tee	
$D_2/D_1 = 0.50$	0.07	Line flow	0.30−0.40
$D_2/D_1 = 0.20$	0.08	Branch flow	0.75−1.80
Expansion—sudden		Cross	
$D_2/D_1 = 1.25$	0.16	Line flow	0.50
$D_2/D_1 = 2.00$	0.57	Branch flow	0.75
$D_2/D_1 = 5.00$	0.92	45° wye	
Expansion—conical		Line flow	0.30
$D_2/D_1 = 1.25$	0.03	Branch flow	0 50
$D_2/D_1 = 2.00$	0.08		
$D_2/D_1 = 5.00$	0.13		

Source: Courtesy of Haestad Methods Water Solutions, Bentley Institute Press.
Note: Subscript 1 = Upstream; Subscript 2 = Downstream.

5.2.3 Form Resistance

Pipeline transitions and appurtenances add *form* resistance to *surface* resistance. Head losses are stepped up by changes in cross-sectional geometry and changing directions of flow. Expansion and contraction exemplify geometric change; elbows and branches, directional change. Valves and meters as well as other appurtenances may create both geometrical and directional change. With rare exceptions, head losses are expressed either in terms of velocity heads, such as $Kv^2/2g$, or as equivalent lengths of straight pipe, $Le = h_f/s = Kv^2/2gs = KD/f$ (see Appendix 17). The outstanding exception is the loss on sudden expansion or enlargement $(v_1 - v_2)^2/2g$, where v_1 is the velocity in the original conduit and v_2 the velocity in the expanded conduit; even it is, however, sometimes converted, for convenience, into $Kv^2/2g$. Because continuity as $a_1v_1 = a_2v_2$ equates $k_1v_1^2/2g$ with $(v_1^2/2g)(1 - a_1/a_2)^2$, the loss at the point of discharge of a pipeline into a reservoir (making a_2 very large in comparison with a_1) equals approximately $v_1^2/2g$; consequently, there is no recovery of energy. In all but special cases like this, k must be determined experimentally. When there is no experimental information, the values of k in Table 5.2 give useful first approximations on likely losses.

EXAMPLE 5.41 FORM RESISTANCE

Summarize and discuss the methods for determination of form (minor) resistances of pipe connections, contractions, expansions (enlargement), and elbows, fittings, and valves,

Solution:

Hydraulic losses of head in pipe fittings are generally expressed as

$$h_f = K\frac{v^2}{2g}$$

or

$$h_f = K\frac{(v_1 - v_2)^2}{2g}$$

where subscript 1 refers to upstream and 2 refers to downstream.

Typical loss of head items can be listed as follows:

From tank to pipe (entrance loss) = $K\frac{v_2^2}{2g}$.

From pipe to tank (exit loss) = $K\frac{v_1^2}{2g}$.

Expansion (enlargement) = $K\frac{(v_1 - v_2)^2}{2g}$.

Contraction = $K\frac{(v_2)^2}{2g}$.

Fittings, elbows, and valves = $K\frac{v^2}{2g}$.

Ideally, K must be determined experimentally. If this is unavailable, the values of K can be obtained from Table 5.2 and Appendixes 17 and 25.

EXAMPLE 5.42 FORM AND MINOR HEAD LOSS DETERMINATION

Determine the form (minor) resistance caused by a sudden contraction from a 12-in. (0.3048 m)-diameter pipe to a 6-in. (0.1504-m)-diameter pipe when water flow is 0.5 ft^3/s (0.01416 m^3/s) and angle of contraction $\theta = 180°$.

Solution 1 (US Customary System):

$D_1 = 12$ in. = 1 ft.

$D_2 = 6$ in. = 0.5 ft.

$\theta = 180°$.

$D_2/D_1 = 0.5/1 = 0.5$.

From Appendix 25, $K_c = 0.37$.
From Table 5.2, $K = 0.37$.
From Appendix 17, equivalent length of 6 in. pipe = 5.5 ft.

$$Q_1 = A_1 v_1 = Q_2 = A_2 v_2.$$

$$0.5 = (\pi/4)(1)^2 v_1 = (\pi/4)(0.5)^2 v_2.$$

$$v_1 = 0.637 \text{ ft/s}.$$

$$v_2 = 2.548 \text{ ft/s}.$$

$$h_f = K\frac{(v_2)^2}{2g}.$$

$$\boldsymbol{h_f} = 0.37(2.548)^2/(2 \times 32.2) = \mathbf{0.037\ ft = 0.44\ in}.$$

Solution 2 (SI System):

$D_1 = 0.3048$ m.

$D_2 = 0.1504$ m.

$\theta = 180°$.

$D_2/D_1 = 0.1504/0.3048 = 0.5$.

From Appendix 25, $K_c = 0.37$.
From Table 5.2, $K = 0.37$.
From Appendix 17, equivalent length of 6 in. pipe = 5.5 ft = 1.676 m.

$$Q_1 = A_1 v_1 = Q_2 = A_2 v_2.$$

$$0.01416 = (\pi/4)(0.3048)^2 v_1 = (\pi/4)(0.1504)^2 v_2.$$

$$v_1 = 0.194\ \text{m/s}.$$

$$v_2 = 0.797\ \text{m/s}.$$

$$h_f = K\frac{(v_2)^2}{2g}.$$

$$\boldsymbol{h_f} = 0.37(0.797)^2/(2\times 9.80) = \mathbf{0.012\ m = 12\ mm}.$$

EXAMPLE 5.43 PLOTTING OF HYDRAULIC AND ENERGY GRADE LINES

A water transmission line consists of three pipes connected in series, pipes AB, CD, and EF, to carry the required water flow from A to F (see Fig. 5.19). There is a sudden reduction BC and a sudden enlargement DE.

a. Draw the HGLs by calculating/plotting H_1, H_2, H_3, H_4, H_5, *and* H_6.
b. Draw the energy grade lines by calculating/plotting E_1, E_2, E_3, E_4, E_5, *and* E_6.

The following data are given:

- Pipe AB is 200 ft long (61 m) and 12 in. (30.48 cm) in diameter, $f = 0.020$.
- Pipe CD is 100 ft long (30.5 m) and 6 in. (15.24 cm) in diameter, $f = 0.015$.
- Pipe EF is 100 ft long (30.5 m) and 12 in. (30.48 cm) in diameter, $f = 0.020$.
- Velocity in pipes AB and EF is 8.025 ft/s (2.446 m/s).
- Pressure head at point A is 300 ft (91.44 m).
- Sudden reduction $K = 0.37$.

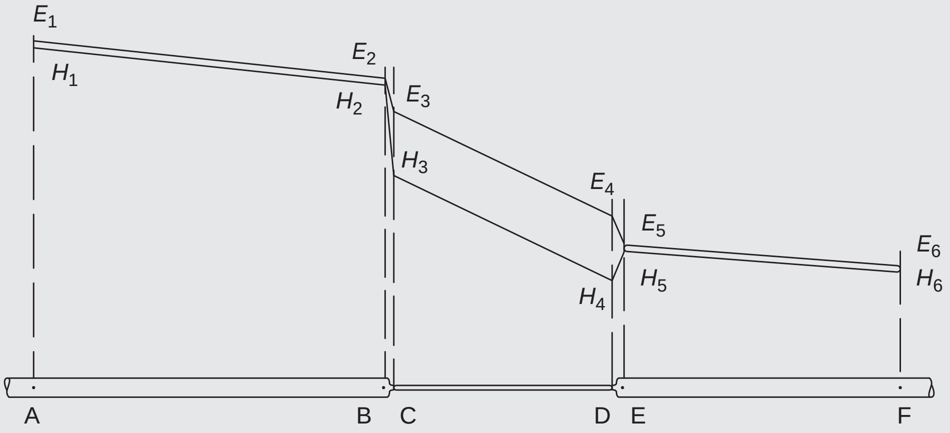


Figure 5.19 Plotting of hydraulic and energy grade lines.

Solution 1 (US Customary System):

$Q_{AB} = A_{AB}v_{AB} = (\pi/4)(12/12)^2(8.025) = 6.3\ \text{ft}^3/\text{s}.$

$Q_{CD} = Q_{AB} = 6.3 = A_{CD}v_{CD} = (\pi/4)(6/12)^2(v_{CD}).$

$v_{CD} = 32.1$ ft/s $\quad (v_{CD})^2/2\text{g} = (32.1)^2/(2 \times 32.2) = 16$ ft.

$v_{AB} = 8.025$ ft/s $\quad (v_{AB})^2/2\text{g} = (8.025)^2/(2 \times 32.2) = 1$ ft.

$v_{EF} = 8.025$ ft/s $\quad (v_{EF})^2/2\text{g} = (8.025)^2/(2 \times 32.2) = 1$ ft.

The energy grade line drops in the direction of water flow by the amount of the lost head. The HGL is below the energy line by the amount of the velocity head at any cross-section. It is important to note that the HGL can rise where a change (an enlargement in pipe size) occurs. This is so because the enlargement decreases the velocity, hence the velocity head decreases and the pressure head increases (conservation of total energy).

$(h_f)_{AB} = f(L/D)(v^2/2g) = 0.02(200/1)(1) = 4$ ft.

$(h_f)_{BC} = K(v^2/2g) = 0.37 \times 16 = 5.9$ ft. (Note: $K = 0.37$ from Table 5.2)

$(h_f)_{CD} = f(L/D)(v^2/2g) = 0.015(100/0.5)(16) = 48$ ft.

$(h_f)_{DE} = (v_D - v_E)^2/2g = (32.1 - 8.085)^2/(2 \times 32.2) = 9$ ft (see Section 5.2.3).

$(h_f)_{EF} = f(L/D)(v^2/2g) = 0.020(100/1)(1) = 2$ ft.

At	From	Head loss, h_f (ft)	EGL elevation (ft)		$v^2/2g$ (ft)	HGL elevation (ft)	
A	Level = 0		E_1	301.0	1.0	H_1	300.0
B	A to B	4	E_2	297.0	1.0	H_2	296.0
C	B to C	5.9	E_3	291.1	16	H_3	275.1
D	C to D	48	E_4	243.1	16	H_4	227.1
E	D to E	9	E_5	234.1	1.0	H_5	233.1
F	E to F	2	E_6	232.1	1.0	H_6	231.1

Plotting and connecting the dots of H_1 to H_6 will form the HGL. Plotting and connecting the dots of E_1 to E_6 will form the energy grade line as shown in Fig. 5.19.

Solution 2 (SI System):

$Q_{AB} = A_{AB}v_{AB} = (\pi/4)(0.3048)^2(2.446) = 0.1784\ \text{m}^3/\text{s}.$

$Q_{CD} = Q_{AB} = 0.1784 = A_{CD}v_{CD} = (\pi/4)(0.1524)^2(v_{CD}).$

$v_{CD} = 9.784$ m/s $\quad (v_{CD})^2/2g = (9.784)^2/(2 \times 9.80) = 4.884$ m.

$v_{AB} = 2.446$ m/s $\quad (v_{AB})^2/2g = (2.446)^2/(2 \times 9.80) = 0.305$ m.

$v_{EF} = 2.446$ m/s $\quad (v_{EF})^2/2g = (2.446)^2/(2 \times 9.80) = 0.305$ m.

Read the explanation in Solution 1.

$(h_f)_{AB} = f(L/D)(v^2/2g) = 0.02(61/0.3048)(0.305) = 1.22$ m.

$(h_f)_{BC} = K(v^2/2g) = 0.37 \times 4.884 = 1.807.$

$(h_f)_{CD} = f(L/D)(v^2/2g) = 0.015(30.5/0.1524)(4.884) = 14.66$ m.

$(h_f)_{DE} = (v_D - v_E)^2/2g = (9.784 - 2.446)^2/(2 \times 9.8) = 2.747$ m (see Section 5.2.3).

$(h_f)_{EF} = f(L/D)(v^2/2g) = 0.020(30.5/0.3048)(0.305) = 0.61$ m.

At	From	Head loss, h_f (m)	EGL elevation (m)		$v^2/2g$ (m)	HGL elevation (m)	
A	Level = 0		E_1	91.745	0.305	H_1	91.44
B	A to B	1.220	E_2	90.525	0.305	H_2	90.22
C	B to C	1.807	E_3	93.297	4.884	H_3	88.41
D	C to D	14.66	E_4	78.637	4.884	H_4	73.75
E	D to E	2.747	E_5	71.311	0.305	H_5	71.01
F	E to F	0.610	E_6	70.701	0.305	H_6	70.40

Note: 1 ft = 0.3048 m

Read the explanations in Solution 1 for plotting the EGL and HGL.

EXAMPLE 5.44 FRICTIONAL HEAD LOSS DETERMINATION

Given the following energy data at two sections, A and B, across a pipe transporting water in a steady-state flow, determine the head loss between the two sections.

a. Potential energy, Z

Section A = 66 ft = 20.12 m.
Section B = 136 ft = 41.45 m.

b. Kinetic energy, $v^2/2g$

Section A = 50 ft = 15.24 m.
Section B = 50 ft = 15.24 m.

c. Pressure energy, P/γ

Section A = 336 ft = 102.41 m.
Section B = 246 ft = 74.98 m.

d. Total energy, E

Section A = 452 ft = 137.77 m.
Section B = 432 ft = 131.67 m.

Solution 1 (US Customary System):

Total energy at B = total energy at A + energy input − energy loss.

$$432 = 452 + 0 - h_f.$$
$$\boldsymbol{h_f = 20 \text{ ft of water}}.$$

Solution 2 (SI System):

Total energy at B = total energy at A + energy input − energy loss.

$$131.67 \text{ m} = 137.77 \text{ m} + 0 - h_{f.}$$
$$\boldsymbol{h_f = 6.1 \text{ m of water}}.$$

EXAMPLE 5.45 MASS FLOW OF INCOMPRESSIBLE FLUID

Determine the mass-flow rate at section B of a pipe line if the static pressure at section A is 102 psig (707.9 kPa = 707.0 kN/m^2) and the 4-in. (101.6-mm)-diameter pipe is flowing full with turbulent flow at an average velocity of 32 ft/s (9.75 m/s).

Solution 1 (US Customary System):

$Q = Av.$

$\gamma Q = \gamma Av = \text{mass flow}$
$= (62.4\ \text{lb/ft}^3)[(\pi/4)(4/12)^2\text{ft}^2](32\ \text{ft/s}).$

Mass flow = 174 lb/s.

Solution 2 (SI System):

$Q = Av.$

$\gamma Q = \gamma Av = \text{mass flow}$
$= (9.8\ \text{kN/m}^3)[(\pi/4)(0.1016)^2\text{m}^2](9.75\ \text{m/s}).$

Mass flow = 0.77 kN/s.

Mass flow rate is the same at all sections. Mass flow at Section A equals mass flow at Section B

EXAMPLE 5.46 MASS FLOW OF COMPRESSIBLE FLUID

Determine the mass-flow rate of air traveling in a long length of 1-in. (25.4-mm)-diameter pipe. At section A, the pressure is 31 psia (215.14 kPa = 215.14 kN/m^2), the temperature is 300°F (148.88°C), and the air velocity is 32 ft/s (9.75 m/s). At the downflow section B, the pressure has been reduced by friction and heat loss to 21 psig (145.7 kPa = 145.7 kN/m^2) (R_g, the gas constant of air = 53.34 ft-lb/lb-°R (286.9 J/kg-K); 1 J = 0.101972 kg-m; and 1 N = 0.101972 kg.)

Solution 1 (US Customary System):

$R_g = PV/T.$

53.4 ft-lb/lb-°R = (31 × 144 lb/ft^2)(V)/(300 + 460°R).

$V = 9.081\ \text{ft}^3/\text{lb}.$

$\gamma = 1/V = 1/9.081\ \text{ft}^3/\text{lb} = 0.11\ \text{lb/ft}^3.$

Mass flow $= \gamma Q = \gamma Av$
$= (0.11\ \text{lb/ft}^3)[(\pi/4)(1/12)^2\text{ft}^2](32\ \text{ft/s})$
$= \mathbf{0.019\ lb/s}.$

Solution 2 (SI System):

$R_g = PV/T.$

286.9 J/kg-K × (0.101972 kg-m/J) = (215.14 kN/m^2)V/(148.88 +275)K.

$V = 57.36\ \text{m}^3/\text{kN}.$

$\gamma = 1/V = 1/57.46 = 0.0174\ \text{kN/m}^3.$

Mass flow $= \gamma Q = \gamma Av$
$= (0.0174\ \text{kN/m}^3)[(\pi/4)(0.0254)^2\text{m}^2](9.75\ \text{m/s})$
$= 0.0000859\ \text{kN/s}$
$= \mathbf{0.00875\ kg/s}$
$= \mathbf{8.75\ g/s}.$

EXAMPLE 5.47 TURBINE HEADLOSS AND HORSEPOWER CALCULATION

Water flows from pipe A (diameter $d_A = 12$ in. = 30.48 cm) to a turbine and then pipe B (diameter $d_B = 24$ in. = 60.96 cm) at the flow rate of 8 ft^3/s (0.2264 m^3/s). The pressures at A and B are 22 psi (152.68 kPa) and −5.2 psi (−36.09 kPa), respectively. The Distance (y) between A and B is 3.2 ft (0.975 m). Determine the headloss of turbine and the horsepower delivered to the turbine by the water.

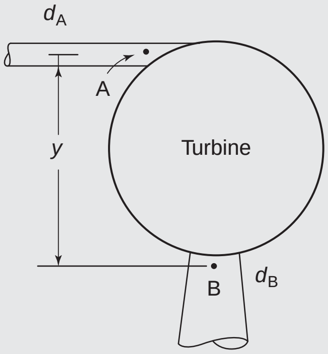


Figure 5.20 Water flow through a turbine.

Solution 1 (U.S. Customary System):

$$Q_A = A_A V_A = \left(\frac{12}{12}\right)^2 \frac{3.14}{4} \text{ ft}^2\, V_A = 8 \text{ ft}^3/\text{s}$$

$$V_A = 10.19 \text{ ft/s}$$

$$Q_A = Q_B = A_B V_B = \left(\frac{24}{12}\right)^2 \frac{3.14}{4} \text{ ft}^2\, V_B = 8 \text{ ft}^3/\text{s}$$

$$V_B = 2.55 \text{ ft/s}$$

$$\frac{P_A}{\gamma} + \frac{V_A^2}{2g} + Z_A - H_T = \frac{P_B}{\gamma} + \frac{V_B^2}{2g} + Z_B$$

$$Z_B = 0 \text{ (datum)};\ Z_A = 3.2 \text{ ft};\ g = 32.2 \text{ ft/s}^2$$

$$\frac{22 \times 144 \text{ lb/ft}^2}{62.4 \text{ lb/ft}^3} + \frac{10.19^2}{2g} + 3.2 - H_T = \frac{-5.2 \times 144 \text{ lb/ft}^2}{62.4 \text{ lb/ft}^3} + \frac{2.55^2}{2g} + 0$$

$$H_T = 67.58 \text{ ft}$$

$$\text{HP} = (Q \text{ gpm})(H \text{ ft})/3957$$

$$\text{HP} = (\gamma \text{ lb/ft}^3)(Q \text{ ft}^3/\text{s})(H \text{ ft}) \Big/ \left(550 \frac{\text{ft-lb/s}}{\text{HP}}\right)$$

$$\text{HP} = \frac{(62.4 \text{ lb/ft}^3)(8 \text{ ft}^3/\text{s})(67.58 \text{ ft})}{550 \text{ (ft-lb/s)/hp}} = 61.33 \text{ hp to turbine}$$

Solution 2 (SI System):

$$Q_A = A_A V_A = (0.3048)^2 \times 0.785 \text{ m}^2 V_A = 0.2264 \text{ m}^3/\text{s}$$

$$V_A = 3.1 \text{ m/s}$$

$$Q_A = Q_B = A_B V_B = (0.6096)^2 \times 0.785\, V_B = 0.2264 \text{ m}^3/\text{s}$$

$$V_B = 0.77 \text{ m/s}$$

$$\frac{P_A}{\gamma} + \frac{V_A^2}{2g} + Z_A - H_T = \frac{P_B}{\gamma} + \frac{V_B^2}{2g} + Z_B$$

$$Z_B = 0 \text{ (datum)};\ Z_A = 0.975 \text{ m};\ g = 9.81 \text{ m/s}^2$$

$$\gamma = 9.8 \text{ kN/m}^3;\ 1 \text{ kPa} = 1 \text{ kN/m}^2$$

$$\frac{152.68 \text{ kN/m}^2}{9.80 \text{ kN/m}^3} + \frac{3.1^2 \text{ m}^2/\text{s}^2}{2 \times 9.81 \text{ m/s}^2} + 0.975 \text{ m} - H_T = \frac{-36.09 \text{ kN/m}^2}{9.8 \text{ kN/m}^3} + \frac{0.77^2 \text{ m}^2/\text{s}^2}{2 \times 9.8 \text{ m/s}^2} + 0$$

$$H_T = 20.695 \text{ m}$$

$$\text{MP} = 9.8066\,(Q \text{ m}^3/\text{s})\,(H \text{ m}) = 9.8066(0.2264 \text{ m}^3)(20.695 \text{ m}) = 45.95 \text{ kW. Note: } 1 \text{ hp} = 0.7457 \text{ kW}$$

5.2.4 Hydraulic Transients

Transmission lines are subjected to transient pressures when valves are opened or closed or when pumps are started or stopped. Water hammer and surge are among such transient phenomena.

Water hammer is the pressure rise accompanying a sudden change in velocity. When velocity is decreased in this way, energy of motion must be stored by elastic deformation of the system. The sequence of phenomena that follows sudden closure of a gate, for example, is quite like what would ensue if a long, rigid spring, traveling at uniform speed, were suddenly stopped and held stationary at its forward end. A pressure wave would travel back along the spring as it compressed against the point of stoppage. *Kinetic energy* would change to *elastic energy*. Then the spring would vibrate back and forth. In a pipe, compression of the water and distention of the pipe wall replace the compression of the spring. The behavior of the pressure wave and the motion of the spring and the water are identically described by the differential equations for one-dimensional waves. Both systems would vibrate indefinitely, were it not for the dissipation of energy by internal friction.

Water hammer is held within bounds in small pipelines by operating them at moderate velocities, because the *pressure* rise in psi or kPa cannot exceed about 50 times the velocity expressed in ft/s or m/s. In larger lines the pressure is held down by arresting flows at a sufficiently slow rate to allow the relief wave to return to the point of control before pressures become excessive. If this is not practicable, pressure relief or surge valves are introduced.

Very large lines, 6 ft (1.8 m) or more in equivalent diameter, operate economically at relatively high velocities. However, the cost of making them strong enough to withstand water hammer would ordinarily be prohibitive if the energy could not be dissipated slowly in *surge tanks*. In its simplest form, a surge tank is a standpipe at the end of the line next to the point of velocity control. If this control is a gate, the tank accepts water and builds up back pressure when velocities are regulated downward. When demand on the line increases, the surge tank supplies immediately needed water and generates the excess hydraulic gradient for accelerating the flow through the conduit. Following a change in the discharge rate, the water level in a surge tank oscillates slowly up and down until excess energy is dissipated by hydraulic friction in the system.

5.3 CAPACITY AND SIZE OF CONDUITS

With rates of water consumption and fire demand known, the capacity of individual supply conduits depends on their position in the waterworks system and the choice of the designer for (a) a structure of full size or (b) duplicate lines staggered in time of construction.

Minimum workable size is one controlling factor in the design of tunnels. Otherwise, size is determined by hydraulic and economic considerations. For a gravity system, that is, where pumping is not required, controlling hydraulic factors are available heads and allowable velocities. Head requirements include proper allowances for drawdown of reservoirs and maintenance of pressure in the various parts of the community, under conditions of normal as well as peak demand. Reservoir heads greater than necessary to transport water at normal velocities may be turned into power when it is economical to do so.

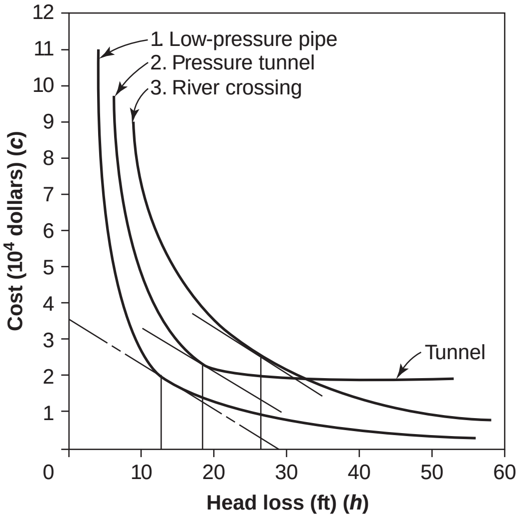


Figure 5.21 Lagrangian optimization of conduit sections by parallel tangents. Conversion factor: 1 ft = 0.3048 m.

Allowable velocities are governed by the characteristics of the water carried and the magnitude of the hydraulic transients. For silt-bearing waters, there are both lower and upper limits of velocity; for *clear* water, only an upper limit. The minimum velocity should prevent deposition of silt; it lies in the vicinity of 2–2.5 ft/s (0.60–0.75 m/s). The maximum velocity should not cause erosion or scour, nor should it endanger the conduit by excessive water hammer when gates are closed quickly. Velocities of 4–6 ft/s (1.2–1.8 m/s) are common, but the upper limit lies between 10 and 20 ft/s (3 and 6.1 m/s) for most materials of which supply conduits are built and for most types of water carried. Unlined canals impose greater restrictions. The size of force mains and of gravity mains that include power generation is fixed by the relative cost or value of the conduit and the cost of pumping or power.

When aqueducts include more than one kind of conduit, the most economical distribution of the available head among the component classes is effected when the change in cost Δc for a given change in head Δh is the same for each kind. The proof for this statement is provided by *Lagrange's method* of undetermined multipliers. As shown in Fig. 5.21 for three components of a conduit with an allowable, or constrained, head loss H, the Lagrangian requirement of $\Delta c_1/\Delta h_1 = \Delta c_2/\Delta h_2 = \Delta c_3/\Delta h_3$ is met when parallel tangents to the three $c:h$ curves identify, by trial, three heads h_1, h_2, and h_3 that satisfy the constraint $h_1 + h_2 + h_3 = H$.

EXAMPLE 5.48 DETERMINATION OF THE MOST ECONOMICAL DISTRIBUTION OF HEADS FOR THREE CONDUIT SECTIONS

Given the costs and losses of head shown in Fig. 5.21 for three sections of a conduit, find the most economical distribution of the available head $H = 60\,\text{ft}\,(18.3\,\text{m})$ between the three sections.

Solution:

By trial, the three heads h_1, h_2, and h_3 that satisfy the constraint $h_1 + h_2 + h_3 = H$ are as follows:

ft	m	
$h_1 = 13.5$	4.1	$c_1 = 2.0 \times 10^4$
$h_2 = 19.0$	5.8	$c_2 = 2.1 \times 10^4$
$h_3 = 27.5$	8.4	$c_3 = 2.2 \times 10^4$
$H = 60.0$	18.3	$C = 6.3 \times 10^4$

EXAMPLE 5.49 UPGRADING CAPACITY AND SIZE OF PIPES

A gravity water supply system consists of a water pipe ($C = 100$) 150 mm in diameter and 3,000 m long that joins two reservoirs that have a difference of water surface elevation of 13 m.

Expecting a higher water demand in the near future, the flow needs to be increased between the reservoirs to three times what would be produced by the present system. Two alternatives are being investigated for tripling the flow: either place a pump on the existing line or install an additional parallel line between the two reservoirs.

1. If a pump is placed, compute the required water head for the added pump.
2. If an additional line is installed, calculate the required size of the added pipe.

Solution:

First let us determine the existing flow in the 150 mm pipe:

$$P_A/\gamma + Z_A + (v_A)^2/2g = P_B/\gamma + Z_B + (v_B)^2/2g + h_f.$$
$$0 + (Z_A - Z_B) + 0 = 0 + h_f.$$
$$h_f = 13\,\text{m}.$$
$$s = h_f/L = 13/3{,}000 = 4.3\text{‰}.$$

The nomogram ($C = 100$) of Fig. 5.17 for $D = 150$ mm and $s = 4.3$‰ will give $\mathbf{Q = 0.010\,m^3}/\text{s}$.

1. The required water head for the added pump:

   Future flow $Q_2 = 3Q = 3 \times 0.010 = 0.030\,\text{m}^3/\text{s}$.

   The nomogram ($C = 100$) of Fig. 5.17 for $D = 150$ mm and $Q_2 = 0.030\,\text{m}^3/\text{s}$ will give $s = 30\%$.

   $$h_f = sL = 30\% \times 3{,}000 = 90\,\text{m}.$$
   $$P_A/\gamma + Z_A + (v_A)^2/2g + h_P = P_B/\gamma + Z_B + (v_B)^2/2g + h_f.$$
   $$0 + (Z_A - Z_B) + 0 + h_P = 0 + 0 + h_f.$$
   $$13 + h_P = 90.$$
   $$h_P = 90 - 13 = 77\,\text{m}.$$

   The required water head for the pump = 77 m.

2. The required size of the added pipe:

   $$Q_{\text{pipe 2}} = 0.030 - 0.010 = 0.020\,\text{m}^3/\text{s}.$$

   The nomogram ($C = 100$) of Fig. 5.17 for $Q_{\text{pipe 2}} = 0.020\,\text{m}^3/\text{s}$ and $s = 4.3\%$ will give $D = 200\,\text{mm}$.

   The required size of the added pipe is 200 mm.

EXAMPLE 5.50 CAPACITY AND SIZE OF PARALLEL PIPES

A town is receiving its water supply from a groundwater tank located at the treatment plant through a gravity main that terminates at an elevated water tank adjacent to the town. The difference between the levels of the tanks is 100 m, and the distance between them is 50 km. The tanks were originally connected by a single pipeline designed to carry 13,000 $\mathrm{m^3/day}$. It was later found necessary to increase the supply to 19,500 $\mathrm{m^3}$/day and consequently the decision was made to lay another pipeline of the same diameter alongside part of the original line and cross-connected to it.

Calculate (given C for all pipes = 100)

1. The diameter of the pipes
2. The length of the second pipe, which was necessary to install

Solution:

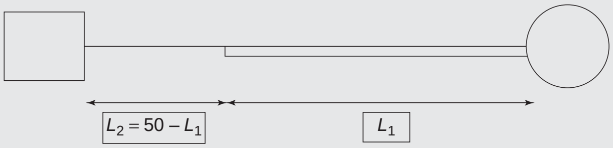
1. The diameter of the pipes:

At present: $Q = 13{,}000\,\mathrm{m^3/d} = (13{,}000\,\mathrm{m^3/d})/\,(24\,\mathrm{h/d})\,(60\,\mathrm{min/h})\,(60\,\mathrm{s/min}) = 0.150\,\mathrm{m^3/s}\ (150\,\mathrm{L/s})$.

$$S = h_\mathrm{f}/L = 100\,\mathrm{m}/50\,\mathrm{km} = 2\text{‰}.$$

The nomogram ($C = 100$) of Fig. 5.17 for $Q = 0.150\,\mathrm{m^3/s}$ and $s = 2\text{‰}$ will give a pipe diameter = 500 mm.

Diameter of existing pipe no. 1 = 500 mm.



$\boldsymbol{Q_2} = (19{,}500\,\mathrm{m^3/d})/(24\,\mathrm{h/d})\,(60\,\mathrm{min/h})\,(60\,\mathrm{s/min})$
$= \mathbf{0.226\ m^3/s} = (226\,\mathrm{L/s})$.

$\boldsymbol{Q_1} = 0.226/2 = \mathbf{0.113\ m^3/s = 113\ L/s}$.

Lines are in parallel, their flows are equal.

2. The length of the second pipe, which had to be installed:

$h_\mathrm{f} = (h_\mathrm{f})_1 + (h_\mathrm{f})_2$.

Lines are in series, their head losses are additive.

$100 = L_1 s_1 + L_2 s_2$.
$100 = L_1 s_1 + (50 - L_1)\, s_2$.

The nomogram ($C = 100$) of Fig. 5.17 for $Q_1 = 0.113\,\mathrm{m^3/s}$ and $D = 500\,\mathrm{mm}$ will give $s_1 = 1.0\text{‰}$.

The nomogram ($C = 100$) of Fig. 5.17 for $Q_2 = 0.226\,\mathrm{m^3/s}$ and $D = 500\,\mathrm{mm}$ will give $s_2 = 4.3\text{‰}$.

$$\begin{aligned} 100 &= L_1\,\mathrm{km} \times 1.0\text{‰} + (50 - L_1) \times 4.3\text{‰}.\\ 100 &= L_1 + 215 - 4.3\,L_1.\\ 3.3\,L_1 &= 115.\\ L_1 &= 115/3.3 = 35\,\mathrm{km}. \end{aligned}$$

The length of the second pipe, which had to be installed, is 35 km.

5.4 MULTIPLE LINES

Although masonry aqueducts and tunnels of all kinds are best designed to the full projected capacity of the system, this is not necessarily so for pipe lines. Parallel lines (Fig. 5.22) built a number of years apart may prove to be more economical. Cost, furthermore, is not the only consideration. It may be expedient to lay more than one line (a) when the maximum pipe size of manufacture is exceeded; 36 in. (900 mm) in the case of centrifugal cast iron pipe, for example; (b) when possible failure would put the line out of commission for a long time; and (c) when pipe location presents special hazards—floods, ice, and ships' anchors endangering river crossings or submarine pipes and cave-ins rupturing pipe lines in mining areas, for example.

Twin lines generally cost 30–50% more than a single line of equal capacity. If they are close enough to be interconnected at frequent intervals, gates should be installed in the bridging pipes to keep most of the system in operation

Figure 5.22 The dual 1,600-mm desalinated water transmission line in Abu Dhabi, UAE. The pipes are 250 km long. The pipes are ductile iron laid above ground level with bitumen/zinc coating (*Source:* http://www.water-technology.net/projects/shuweihat/shuweihat4.html).

during repairs to affected parts. However, if failure of one line will endanger the other, twin lines should not be laid in the same trench. Thus, cast iron pipe can fail so suddenly that a number of pipe lengths will be undermined and pulled apart before the water can be turned off. Another reason for having dual lines traverse different routes is to have them feed water into opposite ends of the distribution system.

5.5 CROSS-SECTIONS

Both hydraulic performance and structural behavior enter into the choice of cross-section. Because hydraulic capacity is a direct function of the hydraulic radius, and the circle and half circle possess the largest hydraulic radius or smallest (frictional) surface for a given volume of water, the circle is the cross-section of choice for closed conduits and the semicircle for open conduits whenever structural conditions permit. Next best are cross-sections in which circles or semicircles can be inscribed. Examples are (a) trapezoids approaching half a hexagon as nearly as maintainable slopes of canals in earth permit; (b) rectangles twice as wide as they are deep for canals and flumes of masonry or wood; (c) semicircles for flumes of wood staves or steel; (d) circles for pressure aqueducts, pressure tunnels, and pipelines; and (e) horseshoe sections for grade aqueducts and grade tunnels.

Internal pressures are best resisted by cylindrical tubes and materials strong in tension; external earth and rock pressures (not counterbalanced by internal pressures) by horseshoe sections and materials strong in compression. By design, the hydraulic properties of horseshoe sections are only slightly poorer than are those of circles. Moreover, their relatively flat *invert* makes for easy transport of excavation and construction materials in and out of the aqueduct. As shown in Fig. 5.18, four circular arcs are struck to form the section: a circular arc rising from the *springing line* of the arch at half depth, two lateral arcs struck by radii equaling the height of the crown above the invert, and a circular arc of like radius establishing the bottom.

5.6 STRUCTURAL REQUIREMENTS

Structurally, closed conduits must resist a number of different forces singly or in combination:

1. Internal pressure equal to the full head of water to which the conduit can be subjected
2. Unbalanced pressures at bends, contractions, and closures
3. Water hammer or increased internal pressure caused by sudden reduction in the velocity of the water—by the rapid closing of a gate or shutdown of a pump, for example
4. External loads in the form of backfill and traffic
5. Their own weight between external supports (piers or hangers)
6. Temperature-induced expansion and contraction

Internal pressure, including water hammer, creates transverse stress or *hoop tension*. Bends and closures at dead ends or gates produce unbalanced pressures and *longitudinal stress*. When conduits are not permitted to change length, variations in temperature likewise create longitudinal stress. External loads and foundation reactions (manner of support), including the weight of the full conduit, and atmospheric pressure (when the conduit is under a vacuum) produce *flexural stress*.

In jointed pipes, such as bell-and-spigot cast iron pipes, the longitudinal stresses must either be resisted by the joints or be relieved by motion. Mechanical joints offer such resistance. The resistance of joints in bell-and-spigot cast iron pipe to being pulled apart can be estimated from Prior's (1935) observational equations:

$$p = \frac{3{,}800}{d+6} - 40 \qquad \text{(US customary units)} \tag{5.39a}$$

$$P_\mathrm{f} = \left(\frac{3{,}000}{d+6} - 31\right) d^2 \qquad \text{(US customary units)} \tag{5.40a}$$

where d is the diameter, in.; p is the intensity of pressure, psig; and P_f is the total force, lb.

The equivalent equations using the SI units are the following:

$$p = \frac{670{,}000}{d+152} - 278 \qquad \text{(SI units)} \tag{5.39b}$$

$$P_\mathrm{f} = \left(\frac{525}{d+152} - 0.2\right) d^2 \qquad \text{(SI units)} \tag{5.40b}$$

where d is the diameter, mm; p is the intensity of pressure, kPa gauge; and P_f is the total force, N.

EXAMPLE 5.51 PRESSURE INTENSITY AND FORCE OF PIPE'S MECHANICAL JOINTS

In jointed cast iron pipes, the longitudinal stresses can be resisted by the mechanical joints. Determine the pressure intensity and total force resisted by the mechanical joints of a 24 in. (609.6 mm) bell-and-spigot cast iron pipe.

Solution 1 (US Customary System):

$$\begin{aligned} p &= [3{,}800/(d+6)] - 40 \\ &= [3{,}800/(24+6)] - 40 \\ &= 86.66 \text{ psig.} \end{aligned}$$

$$\begin{aligned} P_\mathrm{f} &= [3{,}000/(d+6) - 31]d^2 \\ &= [3{,}000/(24+6) - 31](24)^2 \\ &= 39{,}744 \text{ lb.} \end{aligned}$$

Solution 2 (SI System):

$$\begin{aligned} p &= [670{,}000/(d+152)] - 278 \\ &= [670{,}000/(609.6+152)] - 278 \\ &= 601.73 \text{ kPa gauge.} \end{aligned}$$

$$\begin{aligned} P_\mathrm{f} &= [525/(d+152) - 0.2]d^2 \\ &= [525/(609.6+152) - 0.2](609.6)^2 \\ &= 181{,}844 \text{ N.} \end{aligned}$$

Tables of standard dimensions and laying lengths are found in professional manuals, specifications of the American Water Works Association (AWWA), and publications of manufacturers and trade associations.

5.7 LOCATION

Supply conduits are located in much the same way as railroads and highways.

5.7.1 Line and Grade

The invert of a grade aqueduct or grade tunnel is placed on the same slope as the HGL. Cut and fill, as well as cut and cover, are balanced to maintain a uniform gradient and reduce haul. Valleys and rivers that would be bridged by railroads and highways may be bridged also by aqueducts. Such indeed was the practice of ancient Rome, but modern aqueducts no longer rise above valley, stream, and hamlet except where a bridge is needed primarily to carry road or railway traffic. Pressure conduits have taken their place. Sometimes they are laid in trenches as sag pipes to traverse valleys and pass beneath streams; sometimes they strike deep below Earth's surface in pressure tunnels for which geologic exploration fixes both line and grade.

Pressure aqueducts and pipelines move freely up and down slopes. For economy they should hug the HGL in profile and a straight line in plan (Fig. 5.23). The size and thickness of conduit and difficulty of construction must be kept in balance with length. The shortest route is not necessarily the cheapest.

Figure 5.23 Concrete water pipe (*Source*: http://upload.wikimedia.org/wikipedia/commons/2/2e/Concrete_water_pipe.jpg).

Air released from the water and trapped at high points reduces the waterway, increases friction, and may interrupt flow unless an air relief valve or vacuum pump is installed. True siphons should be avoided if possible. However, if the height of rise above the hydraulic grade is confined to less than 20 ft (6 m) and the velocity of flow is kept above 2 ft/s (0.60 m/s), operating troubles will be few. For best results, the line should leave the summit at a slope less than that of the hydraulic gradient.

In practice, possible locations of supply conduits are examined on available maps of the region; the topographic and geologic sheets of the US Geological Survey are useful examples. Route surveys are then carried into the field. Topography and geology are confirmed and developed in needed detail, possibly by aerial surveys, borings, and seismic exploration. Rights of way, accessibility of proposed routes, and the nature of obstructions are also identified. The use of joint rights-of-way with other utilities may generate economies.

5.7.2 Vertical and Horizontal Curves

In long supply lines, changes in direction and grade are effected gradually in order to conserve head and avoid unbalanced pressures. Masonry conduits built in place can be brought to any desired degree of curvature by proper form work. Cast iron and other sectional pipelines are limited in curvature by the maximum angular deflection of standard lengths of pipe at which joints will remain tight. The desired curve is built up by the necessary number of offsets from the tangent. Sharper curves can be formed by shorter or shortened pipes. The smaller the pipe, the sharper can be the deflection. Welded pipelines less than 15 in. (400 mm) in diameter are sufficiently flexible to be bent in the field. The ends of larger steel pipe must be cut at an angle that depends on the type of transverse joint, the thickness of the steel plate, and the size of the pipe.

For sharp curves, transitions, and branches, special fittings are often built up or manufactured of the same materials as the main conduit.

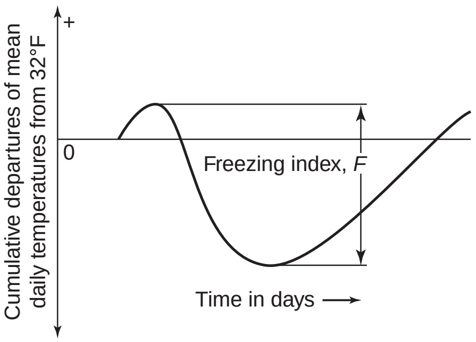


Figure 5.24 Determination of the freezing index of soils as the cumulative departure of the mean daily temperature from 323°F.

5.7.3 Depth of Cover

Conduits that follow the surface of the ground are generally laid below the *frost line*, although the thermal capacity and latent heat of water are so great that there is little danger of freezing as long as the water remains in motion. To reduce the external load on large conduits, only the lower half may be laid below frost. Along the 42nd parallel of latitude, which describes the southern boundaries of Massachusetts, upper New York, and Michigan in the United States, frost seldom penetrates more than 5 ft (1.5 m) beneath the surface; along the 45th parallel the depth increases to 7 ft (2 m). The following equation approximates Shannon's (1945) observations of frost depth:

$$d = 1.65F^{0.468} \qquad \text{(US customary units)} \tag{5.41a}$$

where d is the depth of frozen soil, in.; and F, the freezing index, is the algebraic difference between the maximum positive and maximum negative cumulative departures, $\Sigma(T_d - 32)$, of the daily mean temperatures (T_d) from 32°F. Accumulation, as shown in Fig. 5.24, begins with the first day on which a freezing temperature is recorded. In concept, the *freezing index* is analogous to the *degree day*, which describes the heat requirements of buildings during the heating season. The authors of this book have developed the following frost depth equation using the SI units:

$$d = 55.18F^{0.468} \qquad \text{(SI units)} \tag{5.41b}$$

where d is the depth of frozen soil, mm; and F is the freezing index, which is the algebraic difference between the maximum positive and maximum negative cumulative departures, $\Sigma(T_d - 0)$, of the daily mean temperatures (T_d) from 0°C.

In the absence of daily readings, the value of F may be approximated, in North America, from the mean monthly temperatures as follows:

$$F = (32n - \Sigma T_m)\ 30.2 \qquad \text{(US customary units)} \tag{5.42a}$$

$$F = (0 - \Sigma T_m)\ 30.2 \qquad \text{(SI units)} \tag{5.42b}$$

Here n is the number of months during which the temperature is less than 32°F (0°C); ΣT_m is the sum of the mean temperatures °F or °C during each of these months; and 30.2 is the mean number of days in December, January, February, and March.

Pipes laid at depths of 2–3 ft (0.60–0.90 m) are safe from extremes of heat and ordinary mechanical damage, but it is wise to go to 5 ft (1.5 m) in streets or roads open to heavy vehicles. Otherwise, structural characteristics of conduits determine the allowable depth of cover or weight of backfill. Some conduits may have to be laid in open cut to keep the depth of backfill below the maximum allowable value.

EXAMPLE 5.52 FREEZING INDEX AND FROST DEPTH DETERMINATION

The following are the average consecutive 2-week temperature records of a typical coldest month.

Day	1	2	3	4	5	6	7	8	9	10	11	12	13	14
T_d (°F)	35	37	37	30	28	25	29	30	24	36	40	40	42	45
T_d (°C)	1.67	2.78	2.78	−1.11	−2.22	−3.88	−1.66	−1.11	−4.44	2.22	4.44	4.44	5.55	7.22

Determine the freezing index and the required minimum frost depth.

Solution 1 (US Customary System):

Day	T_d (°F)	$T_d - 32$	$\Sigma\left(T_d - 32\right)$	
1	35	3	3	
2	37	5	8	
3	37	5	13	First maximum positive
4	30	−2	11	
5	28	−4	7	
6	25	−7	0	
7	29	−3	−3	
8	30	−2	−5	
9	24	−8	−13	First maximum negative
10	36	4	−9	
11	40	8	−1	
12	40	8	7	
13	42	10	17	
14	45	13	30	

Figure 5.24 can be plotted for determining F.

The freezing index = 13 + 13 = **26** (**US customary unit**).

The frost depth $d = 1.65F^{0.468}$.

$d = 1.65\,(26)^{0.468} =$ **7.58 in**.

There shall be a minimum of 1 ft (12 in.) of fill material on the top of a sewer or water main for freezing protection. Certain US states near Canada require a minimum of 3 ft of fill material.

Solution 2 (SI system):

Day	T_d (°C)	$T_d - 0$	$\Sigma\left(T_d - 0\right)$	
1	1.67	1.67	1.67	
2	2.78	2.78	4.45	
3	2.78	2.78	7.22	First maximum positive
4	−1.11	−1.11	6.11	
5	−2.22	−2.22	3.9	
6	−3.88	−3.88	0.02	
7	−1.66	−1.66	−1.64	
8	−1.11	−1.11	−2.75	
9	−4.44	−4.44	−7.19	First maximum negative
10	2.22	2.22	−4.97	
11	4.44	4.44	−0.53	
12	4.44	4.44	3.91	
13	5.55	5.55	9.46	
14	7.22	7.22	26.68	

The freezing index = 7.22 + 7.19 = **14.41** (**SI unit**).

The frost depth $d = 55.18F^{0.468}$.

$d = 55.18\,(14.41)^{0.468} =$ **192.33 mm**.

There shall be a minimum of 305 mm of fill material on the top of a sewer or water main for freezing protection. Certain US states near Canada require a minimum of 1 m (3 ft) of fill material.

EXAMPLE 5.53 FREEZING INDEX AND FROST DEPTH REQUIREMENT IN NORTH AMERICA

In the absence of daily readings, the value of freezing index F may be approximated by Eqs. (5.42a) and (5.42b), using the US customary units and the SI units, respectively. Determine the freezing index of a northern state in the United States assuming the following coldest monthly temperatures recorded:

	T_m(°F)	T_m(°C)
December	23	−5
January	26	−3.33
February	24	−4.44
March	31	−0.55

Solution 1 (US Customary System):

$$\begin{aligned}\Sigma T_m &= 23 + 26 + 24 + 31 = 104.\\ F &= (32n - \Sigma T_m) \times 30.2\\ &= (32 \times 4 - 104) \times 30.2\\ &= \mathbf{724.8}.\\ d &= 1.65F^{0.468}\\ &= 1.65(724.8)^{0.468}\\ &= \mathbf{35.98\ in}. > 12 \text{ in. Select 36 in. of fill material.}\end{aligned}$$

Solution 2 (SI System):

$$\begin{aligned}\Sigma T_m &= -5 - 3.33 - 4.44 - 0.55 = -13.32.\\ F &= [0 - \Sigma T_m] \times 30.2\\ &= [0 - (-13.32)] \times 30.2\\ &= \mathbf{402.26}.\\ d &= 55.18F^{0.468}\\ &= 55.18\,(402.26)^{0.468}\\ &= \mathbf{913.46\ mm} > 305 \text{ mm. Select 1 m of fill material.}\end{aligned}$$

5.8 MATERIALS OF CONSTRUCTION

Selection of pipeline materials is based on carrying capacity, strength, life or durability, ease of transportation, handling, and laying, safety, availability, cost in place, and cost of maintenance. Various types of iron, steel, reinforced concrete, and fiberglass are most used for water transmission pipes, but plastic pipes are now being made in the smaller sizes. Other materials may come into use in the future.

Loss of capacity with age or, more strictly, with service depends on the properties of the water carried and the characteristics of the pipe. Modern methods for controlling aggressive water promise that the corrosion of metallic pipes and the disintegration of cement linings and of reinforced concrete pipe will be held in check very largely, if not fully.

Cement and bituminous-enamel linings and reinforced concrete and fiberglass pipes do not, as a rule, deteriorate significantly with service.

5.8.1 Carrying Capacity

The initial value of the Hazen–Williams coefficient C hovers around 140 for all types of well-laid pipelines but tends to be somewhat higher for reinforced concrete and fiberglass lines and to drop to a normal value of about 130 for unlined cast iron pipe. Cast iron and steel pipes lined with cement or with bituminous enamel possess coefficients of 130 and over on the basis of their nominal diameter; improved smoothness offsets the reduction in cross-section.

5.8.2 Strength

Steel pipes can resist high internal pressures, but large lines cannot withstand heavy external loads or partial vacuums unless special measures are taken to resist these forces. Ductile iron and fiberglass pipes are good for moderately high water pressures and appreciable external loads, provided that they are properly bedded. Prestressed reinforced concrete pipe is satisfactory for high water pressures. All types of concrete pipe can be designed to support high external loads.

5.8.3 Durability

Experience with all but coated ductile iron pipe (length of life 100 years) has been too short and changes in water treatment have been too many to give us reliable values on the length of life of different pipe materials. The corrosiveness of the water, the quality of the material, and the type and thickness of protective coating all influence the useful life of the various types of water pipes. External corrosion (soil corrosion) is important, along with internal corrosion. Pipes laid in acid soils, seawater, and cinder fills may need special protection.

5.8.4 Transportation

When pipelines must be built in rugged and inaccessible locations, their size and weight become important. Ductile iron pipe is heavy in the larger sizes; steel pipe relatively much lighter. The normal laying length of cast iron pipe is 12 ft (3.7 m). Lengths of 16.4, 18, and 20 ft (5, 5.5, and 6 m) are also available in different types of bell-and-spigot pipe. The length of steel pipe is 20–30 ft (6–9 m). Both prestressed and cast reinforced concrete pipe are generally fabricated in the vicinity of the pipeline. The sections are 12 and 16 ft (3.6 and 4.9 m) long and very heavy in the larger sizes. A diameter smaller than 24 in. (600 mm) is unusual.

5.8.5 Safety

Breaks in cast iron pipes can occur suddenly and are often quite destructive. By contrast, steel and reinforced concrete pipes fail slowly, chiefly by corrosion. However, steel pipelines may collapse under vacuum while they are being drained. With proper operating procedures, this is a rare occurrence. Fiberglass pipe fails suddenly, much like cast iron pipe.

5.8.6 Maintenance

Pipelines of all sizes and kinds must be watched for leakage or loss of pressure—outward signs of failure. There is little choice between materials in this respect. Repairs to precast concrete pipe are perhaps the most difficult, but they are rarely required. Cast iron and small welded-steel pipes can be cleaned by scraping machines and lined in place with cement to restore their capacity. New lines and repaired lines should be disinfected before they are put into service.

5.8.7 Leakage

All pipelines should be tested for tightness as they are constructed. Observed *leakage* is often expressed in gal/day/in. diameter (nominal)/mile of pipe using the US customary units, or in L/day/mm diameter (nominal)/km of pipe using the SI units. The test pressure must naturally be stated. To conduct a leakage test, the line is isolated by closing gates and placing a temporary header or plug at the end of the section to be tested. The pipe is then filled with water and placed under pressure, the water needed to maintain the pressure being measured by an ordinary household meter. Where there is no water, air may be substituted. Losses are assumed to vary with the square root of the pressure, as in orifices.

The allowable leakage of bell-and-spigot cast iron pipe that has been carefully laid and well tested during construction is often set at

$$Q = \frac{nd\sqrt{p}}{1{,}850} \qquad \text{(US customary units)} \tag{5.43a}$$

where Q is the leakage, gal/h; n is the number of joints in the length of line tested; d is the nominal pipe diameter, in.; and p is the average pressure during test, psig. A mile (1.6 km) of 24 in. (600 mm) cast iron pipe laid in 12 ft (3.6 m) lengths and tested under a pressure of 64 psig (444 kPa), for example, can be expected to show a leakage of

$$Q = (5{,}280/12) \times 24 \times \sqrt{64}/1{,}850 = 46 \text{ gal/h } (174 \text{ L/h}).$$

Considering that the pipe has a carrying capacity of 250,000 gal/h (946,000 L/h) at a velocity of 3 ft/s (0.90 m/s), the expected leakage from joints $(46 \times 24 \text{ gal/day})/24$ in./mi = 46 gal/day/in./mi, which is relatively small.

The equivalent metric leakage equation using the SI units is as follows:

$$Q = \frac{nd\sqrt{p}}{32{,}500} \qquad \text{(SI units)} \tag{5.43b}$$

where Q is the leakage, L/h; n is the number of joints in the length of line tested; d is the nominal pipe diameter, mm; and p is the average pressure during test, kPa. For the same example introduced above, the expected leakage calculated using SI units is

$$Q = \left[(1.6 \times 1{,}000/3.6) \times 600\sqrt{444}\right]/32{,}500 = 173 \text{ L/h}.$$

The expected leakage from joints is equal to $(173 \times 24 \text{ L/day})/600 \text{ mm}/1.6 \text{ km} = 4.32$ L/day/mm/km.

5.9 APPURTENANCES

To isolate and drain pipeline sections for test, inspection, cleaning, and repairs, a number of appurtenances, or auxiliaries, are generally installed in the line (Figs. 5.25 and 5.26).

5.9.1 Gate Valves

Gate valves are usually placed at major summits of pressure conduits because (a) summits identify the sections of line that can be drained by gravity and (b) pressures are least at these points, making for cheaper valves and easier operation. For the sake of economy, valves smaller in diameter than the conduit itself are generally installed together with necessary

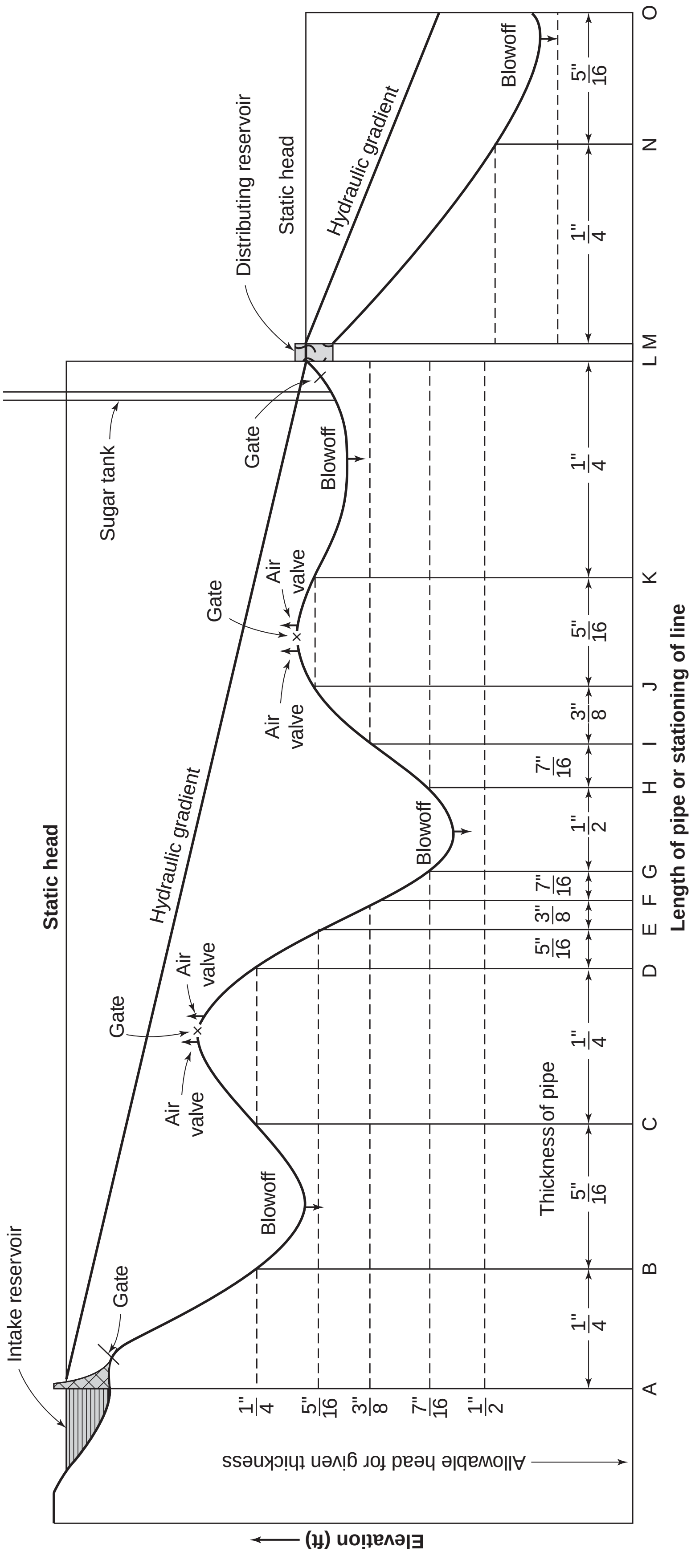


Figure 5.25 Profile of pipeline showing pipe thickness and location of gates, blowoffs, and air valves (not to scale, vertical scale magnified). Conversion factors: 1″ = 1 in. = 25.4 mm; 1′ = 1 ft = 0.3048 m.

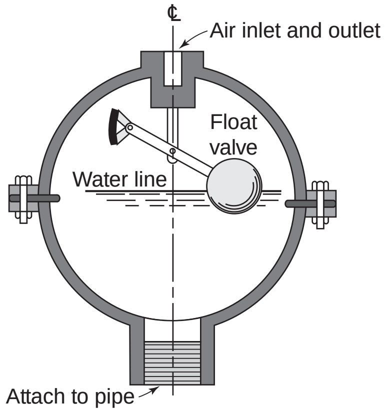


Figure 5.26 Air inlet and release valve.

reducers and increasers. Gates 8 in. (200 mm) in diameter or larger commonly include a 4 or 6 in. (100 or 150 mm) gated bypass. When the larger gate is seated under pressure, water admitted through the bypass can equalize the pressure on both sides and make it easier to lift the main gate.

Gravity conduits are commonly provided with gate chambers (a) at points strategic for the operation of the supply conduit, (b) at the two ends of sag pipes and pressure tunnels, and (c) wherever it is convenient to drain given sections. Sluice gates are normally installed in grade conduits, particularly in large ones. In special situations, needle valves are preferred for fine control of flow, butterfly valves for ease of operation, and cone valves for regulating time of closure and controlling water hammer.

5.9.2 Blowoffs

In pressure conduits, small, gated takeoffs, known as *blowoff* or *scour valves*, are provided at low points in the line. They discharge into natural drainage channels or empty into a sump from which the water can be pumped to waste. There should be no direct connection to sewers or polluted water courses. For safety, two blowoff valves are placed in series. The chance of both failing to close is thus reduced greatly. Their size depends on local circumstances, especially on the time in which a given section of line is to be emptied and on the resulting velocities of flow. Calculations are based on orifice discharge under a falling head, equal to the difference in elevation of the water surface in the conduit and the blowoff, minus the friction head. Frequency of operation depends on the quality of the water carried, especially on silt loads. The drainage gates of gravity conduits are placed in gate chambers.

5.9.3 Air Valves

Rigid pipes and pressure conduits are equipped with *air valves* at all high points. The valves automatically remove (a) air displaced while the line is being filled and (b) air released from the flowing water when the pressure decreases appreciably or summits lie close to the HGL. A manually operated cock or gate can be substituted if the pressure at the summit is high. Little, if any, air will then accumulate, and air needs to escape only while the line is being filled.

Steel and other flexible conduits are equipped with automatic air valves that will also admit air to the line and prevent its collapse under negative pressure (see Fig. 5.26). Pressure differences are generated when a line is being drained on purpose or when water escapes accidentally through a break at a low point. Locations of choice are both sides of gates at summits, the downstream side of other gates, and changes in grade to steeper slopes in sections of line not otherwise protected by air valves.

The required valve size is related to the size of the conduit, and to the velocities at which the line is emptied. The following ratios of air valve size to conduit diameter provide common but rough estimates of needed sizes:

For release of air only: 1:12 or 1 in./ft (83 mm/m)

For admission as well as release of air: 1:8 or 1.5 in./ft (125 mm/m).

An approximate calculation will show that under a vacuum of 48 in. (1,220 mm) of water, an automatic air valve, acting as an injection orifice with a coefficient of discharge of 0.5 under a head of $4/\left(1.3\times10^{-3}\right) =$ 3,080 ft (939 m) of air of specific gravity 1.3×10, is expected to admit about $0.5\sqrt{2g\times3{,}080} = 220\,\text{ft}^3/\text{s}$ of air/ft^2 $\left(67.2\,\text{m}^3/\text{s of air}/\text{m}^2\right)$ of valve. If the diameter ratio is 1:8, the displacement velocity in the conduit can be as high as $220/64 = 3.5$ ft/s (1 m/s) without exceeding a vacuum of 48 in. (1,220 mm) of water. A similar calculation will show the rate of release of air. The amounts of air that can be dissolved by water at atmospheric pressure are about 2.9% by volume at 32°F (0°C) and 1.9% at 77°F (25°C), changing in direct proportion to the pressure. Accordingly, they are doubled at 2 atm or 14.7 psig (102 kPa gauge).

5.9.4 Check Valves

Check valves are used to maintain flow in one direction only by closing when the flow begins to reverse. They are placed on force mains to prevent *backflow* when pumps shut down. When the flow is in the same direction as the specified direction of the check valve, the valve is considered to be fully open.

5.9.5 Pressure-Reducing Valves

Pressure-reducing valves are used to keep pressures at safe levels in low-lying areas. These valves are often used to separate pressure zones in water distribution networks. These valves prevent the pressure downstream from exceeding a specified level, in order to avoid pressures and flows that could otherwise have undesirable effects on the system. A pressure or a hydraulic grade is used to control the operation of the valve.

5.9.6 Pressure-Sustaining Valves

Pressure-sustaining valves maintain a specified pressure upstream of the valve. Similar to the other regulating valves, they are often used to ensure that pressures in the system (upstream, in this case) will not drop to unacceptable levels. A pressure or a hydraulic grade is used to control the operation of a pressure-sustaining valve.

5.9.7 Pressure Breaker Valves

Pressure breaker valves create a specified head loss across the valve and are often used to model components that cannot be easily modeled using standard minor loss elements.

5.9.8 Flow Control Valves

A *flow control valve* limits the flow rate through the valve to a specified value in a specified direction. A flow rate is used to control the operation of a flow control valve. These valves are commonly found in areas where a water district has contracted with another district or a private developer to limit the maximum demand to a value that will not adversely affect the provider's system.

5.9.9 Throttle Control Valves

Throttle control valves simulate minor loss elements whose head loss characteristics change over time. With a throttle control valve, the minor loss K is adjusted based on some other system flow or head.

5.9.10 Manholes

Access *manholes* are spaced 1,000–2,000 ft (300–600 m) apart on large conduits. They are helpful during construction and serve later for inspection and repairs. They are less common on cast iron lines than on steel and concrete lines.

5.9.11 Insulation Joints

Insulation joints control electrolysis by introducing resistance to the flow of stray electric currents along pipelines. Modern insulation joints make use of rubber gaskets or rings and of rubber-covered sections of pipe sufficiently long to introduce appreciable resistance.

5.9.12 Expansion Joints

The effect of temperature changes is small if pipe joints permit adequate movement. Steel pipe laid with rigid transverse joints must either be allowed to expand at definite points or be rigidly restrained by anchoring the line.

5.9.13 Anchorages

Anchorages are employed (a) to resist the tendency of pipes to pull apart at bends and other points of unbalanced pressure when the resistance of their joints to longitudinal stresses is exceeded, (b) to resist the tendency of pipes laid on steep gradients to pull apart when the resistance of their joints to longitudinal stresses is inadequate, and (c) to restrain or direct the expansion and contraction of rigidly joined pipes under the influence of temperature changes.

Anchorages take many forms as follows:

1. For bends—both horizontal and vertical—concrete buttresses or *kick blocks* resisting the unbalanced pressure by their weight, much as a gravity dam resists the pressure of the water behind it, taking into consideration the resistance offered by the pipe joints themselves, by the friction of the pipe exterior, and by the bearing value of the soil in which the block is buried
2. Steel straps attached to heavy boulders or to bedrock
3. Lugs cast on pipes and fittings to hold tie rods that prevent movement of the pipeline
4. Anchorages of mass concrete on steel pipe to keep it from moving, or to force motion to take place at expansion joints inserted for that purpose—the pipe being well bonded to the anchors, for example, by angle irons welded onto the pipe
5. Gate chambers so designed of steel and concrete that they hold the two ends of steel lines rigidly in place.

In the absence of expansion joints, steel pipe must be anchored at each side of gates and meters in order to prevent their destruction by pipe movement. In the absence of anchors, flanged gates are sometimes bolted on one side to the pipe—usually the upstream side—and on the other side to a cast iron nipple connected to the pipe by means of a sleeve or expansion joint.

5.9.14 Other Appurtenances

Other appurtenances that may be necessary include the following:

1. *Surge tanks* at the end of the line to reduce water hammer created by operation of a valve at the end of the line

2. *Pressure relief valves* or overflow towers on one or more summits to keep the pressure in the line below a given value by letting water discharge to waste when the pressure builds up beyond the design value
3. *Self-acting shutoff valves* triggered to close when the pipe velocity exceeds a predetermined value as a result of an accident
4. *Altitude-control valves* that shut off the inlet to service reservoirs, elevated tanks, and standpipes before overflow levels are reached
5. *Venturi* or other meters and recorders to measure the flows

5.10 ADDITIONAL HYDRAULICS TOPICS

The following additional hydraulic topics are important to those who plan to take professional engineering (PE) examinations in North America. Several of the included solved examples and homework problems were given previously in PE examinations.

5.10.1 Measurement of Fluid Flow and Hydraulic Coefficients

Various coefficients of hydraulics have been developed and used in civil and environmental engineering practice mainly for measuring fluids flow. The common fluid measurement devices include pitot tubes, nozzles, orifices, venturi meters, and flumes. Application of Bernoulli equation and hydraulic coefficients is important for calibration of fluid measurement devices.

The coefficient of discharge, C_d, is the ratio of actual discharge through a hydraulic device to the ideal discharge. This coefficient is expressed as

$$C_d = \text{actual flow/ideal flow}$$

$$C_d = Q/A\,(2gh)^{0.5} \tag{5.44}$$

$$Q = C_d A\,(2gh)^{0.5} = Av \tag{5.45}$$

$$v = C_d\,(2gh)^{0.5} \tag{5.46}$$

where

C_d = coefficient of discharge, dimensionless
Q = actual flow, ft^3/s (m^3/s)
A = cross-sectional area of flow through the hydraulic device, ft^2 (m^2)
g = 32.2 ft/s^2 (9.81 m/s^2)
h = total head, ft (m)
v = ideal velocity, ft/s (m/s)

The coefficient of velocity C_v is the ratio of the actual mean velocity in the cross-section of the flow stream (jet) to the ideal mean velocity, which would occur without friction. Thus

$$C_v = \text{actual mean velocity/ideal mean velocity}$$

$$C_v = v/v_i = v/(2gh)^{0.5} \tag{5.47a}$$

$$v = C_v(2gh)^{0.5} \tag{5.47b}$$

where

C_v = coefficient of velocity, dimensionless
v = actual mean velocity, ft/s (m/s)
v_i = ideal or theoretical velocity, ft/s (m/s)

The coefficient of contraction C_c is the ratio of the area of the contracted section of a fluid stream (jet) to the area of the opening through which the fluid flows.

$$C_c = \text{area of stream (jet)/area of opening}$$

$$C_c = A_{jet}/A_o = C_d/C_v \tag{5.48a}$$

$$C_d = C_v\,C_c \tag{5.48b}$$

where

C_c = coefficient of contraction, dimensionless
A_{jet} = area of jet stream, ft^2 (m^2)
A_o = area of opening, ft^2 (m^2)

The area of jet, A_{jet}, is measured at the "vena contracta" (v.c.), which is located at ½ orifice diameter downstream from the orifice. Hence, C_c and C_v can be redefined according to v.c.:

$$C_c = \text{area at v.c./area of orifice} \tag{5.48c}$$

$$C_v = \text{actual velocity at v.c./theoretical velocity at v.c.} \tag{5.47c}$$

$$v_i = (2gh)^{0.5} \tag{5.47d}$$

The head loss, h_f, in hydraulic measuring devices can be expressed as

$$h_f = \{[(1/(C_v)^2] - 1\}(v_{jet})^2/2g \tag{5.49}$$

Where

v_{jet} = jet stream velocity, ft/s or m/s

The coordinates (x, y) of a hydraulic jet can be determined according to the following two kinematic equations:

$$x = vt \tag{5.50}$$

$$y = {}^{1}\!/_{2}gt^2 \tag{5.51}$$

EXAMPLE 5.54 DETERMINATION OF DISCHARGE COEFFICIENT

Determine the discharge confident, C_d, for a hydraulic system when water flows through a standard 1-in. (25.4-mm)-diameter orifice under an 18 ft (5.4864 m) head at a rate of 0.111 ft^3/s (0.0031435 m^3/s).

Solution 1 (US Customary System):

$$Q = C_\mathrm{d} A\,(2gh)^{0.5} \tag{5.45}$$

$$0.111 = C_\mathrm{d}(\pi/4)(1/12)^2\,(2 \times 32.2 \times 18)^{0.5}.$$

$$\boldsymbol{C_\mathrm{d} = 0.60}.$$

Solution 2 (SI System):

$$Q = C_\mathrm{d} A\,(2gh)^{0.5} \tag{5.45}$$

$$0.0031435 = C_\mathrm{d}\,(\pi/4)(0.0254)^2\,(2 \times 9.81 \times 5.4864)^{0.5}.$$

$$\boldsymbol{C_\mathrm{d} = 0.60}.$$

EXAMPLE 5.55 DETERMINATION OF CONTRACTION COEFFICIENT

Water flows through 1-in. (25.4-mm)-diameter orifice under an 18 ft (5.4864 m) head. The jet strikes a wall 5 ft (1.524 m) away and 0.4 ft (0.1219 m) vertically below the center line of the contracted section of the water jet. Determine the coefficient of contraction, C_c, if the coefficient of discharge, C_d, is known to be 0.60 from Example 5.54.

Solution 1 (US Customary System):

$$x = vt \tag{5.50}$$

$$y = {}^1\!/_2 g t^2 \tag{5.51}$$

Eliminating t from the two equations one gets

$$x^2 = (2\,v^2/g)y.$$

$$(5)^2 = (2 \times v^2/32.2)(0.4).$$

$$v = 32.1\ \mathrm{ft/s}.$$

$$v = C_v\,(2gh)^{0.5}. \tag{5.47b}$$

$$32.1 = C_v\,(2 \times 32.2 \times 18)^{0.5}.$$

$$C_v = 0.95.$$

$$C_\mathrm{c} = C_\mathrm{d}/C_v. \tag{5.48a}$$

$$\boldsymbol{C_\mathrm{c}} = 0.60/0.95 = \mathbf{0.63}.$$

Solution 2 (SI System):

$$x = vt \tag{5.50}$$

$$y = {}^1\!/_2\, g t^2 \tag{5.51}$$

Eliminating t from the two equations one gets

$$x^2 = (2\,v^2/g)y.$$

$$(1.524)^2 = (2v^2/9.81)\,(0.1219).$$

$$v = 9.67\ \mathrm{m/s}.$$

$$v = C_v (2gh)^{0.5} \tag{5.47b}$$

$$9.67 = C_v (2 \times 9.81 \times 5.4864)^{0.5}.$$
$$C_v = 0.93.$$

$$C_c = C_d / C_v \tag{5.48a}$$

$$\boldsymbol{C_c} = 0.60/0.93 = \mathbf{0.64}.$$

EXAMPLE 5.56 DETERMINATION OF DISCHARGE COEFFICIENT

Determine the coefficient of discharge for an orifice 2 in. (50.8 mm) in diameter, which discharges from a tank with a head of 16 ft (4.876 m). The discharge rate Q is measured at 0.55 ft^3/s (0.0156 m^3/s). The actual velocity at the vena contracta, v.c., is 29.0 ft/s (8.839 m/s).

Solution 1 (US Customary System):

$$v_i = (2gh)^{0.5} \tag{5.47d}$$

$$v_i = (2 \times 32.2 \text{ ft/s}^2 \times 16 \text{ ft})^{0.5} = 32.1 \text{ ft/s}.$$

$$C_v = v/v_i \tag{5.47a}$$

$$C_v = (29 \text{ ft/s})/(32.1 \text{ ft/s}) = 0.90.$$

$$\text{Area of orifice:} \quad A_o = (\pi/4)(2/12)^2 = 0.0218 \text{ ft}^2.$$

$$\text{Area at v.c.:} \quad A_{jet} = Q/v = (0.55 \text{ ft}^3/\text{s})/(29 \text{ ft/s}) = 0.019 \text{ ft}^2.$$

$$C_c = A_{jet}/A_o = (0.019 \text{ ft}^2)/(0.0218 \text{ ft}^2) = 0.87 \tag{5.48a}$$

$$\boldsymbol{C_d} = C_v C_c = 0.90 \times 0.87 = \mathbf{0.78} \tag{5.48b}$$

Solution 2 (SI System):

$$v_i = (2gh)^{0.5} \tag{5.47d}$$

$$v_i = (2 \times 9.81 \text{ m/s}^2 \times 4.876 \text{ m})^{0.5} = 9.78 \text{ m/s}.$$

$$C_v = v/v_i \tag{5.47a}$$

$$C_v = (8.839 \text{ m/s})/(9.78 \text{ m/s}) = 0.90.$$

$$\text{Area of orifice:} \quad A_o = (\pi/4)(0.0508)^2 = 0.002 \text{ m}^2.$$

$$\text{Area at v.c.:} \quad A_{jet} = Q/v = (0.0156 \text{ m}^3/\text{s})/(8.839 \text{ m/s}) = 0.00176 \text{ m}^2.$$

$$C_c = A_{jet}/A_o = (0.00176 \text{ m}^2)/(0.002 \text{ m}^2) = 0.88 \tag{5.48a}$$

$$\boldsymbol{C_d} = C_v C_c = 0.90 \times 0.88 = \mathbf{0.79}. \tag{5.48b}$$

5.10.2 Forces Developed by Moving Fluids

Water hammer is caused by the sudden decrease in liquid motion. In a water pipeline, the time of pressure wave to travel upstream and back (round-trip) is given by

$$t = 2L/c \tag{5.52}$$

where

t = round-trip travel time, s
L = length of pipe, ft (m)
c = celerity (speed of propagation) of pressure wave, ft/s (m/s)

Increase in water hammer pressure is caused by the sudden closing of a valve or a power outage. The change in pressure is calculated by

$$(\Delta P) = \rho c\,(\Delta v) \qquad \textbf{(5.53)}$$

where
ΔP = change in pressure, lb/ft^2 (N/m^2)
ρ = fluid density, slug/ft^3 (kg/m^3)
c = celerity, ft/s (m/s)
Δv = change in fluid velocity, ft/s (m/s)

For rigid pipes, such as cast iron pipes, the celerity of the pressure wave is obtained by

$$c = (E_B/\rho)^{0.5} \qquad \textbf{(5.54)}$$

where
c = celerity, ft/s (m/s)
E_B = bulk modulus of the fluid, lb/ft^2 (N/m^2)
ρ = density, slug/ft^3 (kg/m^3)

For non-rigid pipes, such as PVC pipes, the expression for celerity is

$$c = \{E_B/\rho[1 + (E_B/E)(D/t_w)]\}^{0.5} \qquad \textbf{(5.55)}$$

where
E = modulus of elasticity of pipe wall, lb/ft^2 (N/m^2)
D = inner diameter of pipe, ft (m)
t_w = pipe wall thickness, ft (m)

The tensile stress, S_t, developed in a moving fluid can be calculated by the following equation:

$$S_t = (\Delta P)\,r/t_k \qquad \textbf{(5.56)}$$

where
S_t = tensile stress of pipe, lb/ft^2 (N/m^2)
ΔP = change in pressure, lb/ft^2 (N/m^2)
r = pipe radius, ft (m)
t_k = pipe wall thickness, ft (m)

Equation (5.56) can be used for moving fluid as well as non-moving fluid in a pipe or cylindrical liquid storage tank. When the equation is used for a tank, ΔP is the internal–external pressure difference, or the measured gauge pressure (actual pressure = −14.7 psi).

A spherical liquid storage tank is twice as strong as a cylindrical tank with hemispherical ends. In such a case:

$$S_t = {}^1\!/_2(\Delta P)\,r/t_k \qquad \textbf{(5.57)}$$

where
S_t = tensile stress in tank wall, lb/ft^2 (N/m^2)
ΔP = internal–external pressure difference, that is, gauge pressure, lb/ft^2 (N/m^2)
r = tank radius, in. (mm)
t_k = tank wall thickness, in. (mm)

EXAMPLE 5.57 VELOCITY OF PRESSURE WAVES

Determine the velocity of pressure waves (celerity) traveling along a rigid pipe containing water at 60°F (15.6°C).

Solution 1 (US Customary System):
From Appendix 3 at 60°F,

$$E_B = 311 \times 10^3\ \text{lb/in.}^2 = 4.478 \times 10^7\ \text{lb/ft}^2.$$
$$\rho = 1.938\ \text{slug/ft}^3\ (1\ \text{slug} = 1\ \text{lb-s}^2/\text{ft}).$$

$$c = (E_B/\rho)^{0.5} \qquad \textbf{(5.54)}$$
$$c = (4.478 \times 10^7/1.938)^{0.5}.$$
$$\mathbf{c = 4{,}807\ ft/s}.$$

Solution 2 (SI System):
From Appendix 3 at 15.6°C,

$$E_B = 214 \times 10^7\ \text{N/m}^2.$$
$$\rho = 999.1\ \text{kg/m}^3.$$

$$c = (E_B/\rho)^{0.5} \qquad \textbf{(5.54)}$$
$$c = (214 \times 10^7/999.1)^{0.5}.$$
$$\mathbf{c = 1{,}463\ m/s}.$$

EXAMPLE 5.58 PRESSURE INCREASE DUE TO SUDDEN STOP OF FLOW

Determine the expected increase in pressure if the liquid flow is suddenly stopped assuming the following:

1. 12 in. (0.3048 m) rigid pipe.
2. Liquid in pipe is oil with a specific gravity of 0.8.
3. Flow velocity is 4 ft/s (1.22 m/s).
4. Bulk modulus of oil is 200,000 lb/in.^{2} = 200,000 × 144 lb/ft^2 (138.8 × 10^7 N/m^2).

Solution 1 (US Customary System):

$$c = (E_B/\rho)^{0.5} \tag{5.54}$$

and

$$\rho = \gamma/g.$$

$$c = (200{,}000 \times 144/0.8 \times 62.4/32.2)^{0.5}.$$
$$c = 4{,}310 \text{ ft/s}.$$

$$(\Delta P) = \rho c\,(\Delta v) \tag{5.53}$$

$$\begin{aligned}(\boldsymbol{\Delta P}) &= (\gamma/g)\,c\,(\Delta v)\\ &= (0.8 \times 62.4/32.2)\,(4{,}310)(4 - 0)\\ &= 26{,}700 \text{ lb/ft}^2\\ &= \mathbf{185\ lb/in.^2}.\end{aligned}$$

Solution 2 (SI System):

$$c = (E_B/\rho)^{0.5} \tag{5.54}$$

$$\begin{aligned}c &= [(138.8 \times 10^7/(0.8 \times 1{,}000)]^{0.5}.\\ &= 1{,}317 \text{ m/s}.\end{aligned}$$

$$(\Delta P) = \rho c(\Delta v) \tag{5.53}$$

$$\begin{aligned}&= (0.8 \times 1{,}000)\,(1{,}317)\,(1.22 - 0)\\ &= 1{,}285{,}400 \text{ N/m}^2\\ &= \mathbf{1{,}285.4\ kN/m^2}.\end{aligned}$$

EXAMPLE 5.59 PIPE TENSILE STRESS DUE TO SUDDEN VALVE CLOSURE

A 48 in. (1,219 mm) steel pipe 3/8 in. (9.525 mm) thick carries water at 60°F (15.6°C) and a velocity of 6.5 ft/s (1.98 m/s). The pipe is 11,000 ft (3,353 m) long. Assume the steel pipe is elastic and non-rigid and its celerity was determined to be 3,155 ft/s (961.6 m/s). Determine

1. The time of travel of the pressure wave upstream and back (round-trip).
2. The increase in water pressure within the pipe.
3. The tensile stress in the pipe, if the valve is closed in 2 s.
4. Discuss how the water hammer can be avoided.

Solution 1 (US Customary System):

1. The time of travel of the pressure wave upstream and back (round-trip):

$$t = 2\,L/c \tag{5.52}$$

$$\boldsymbol{t} = 2 \times 11{,}000 \text{ ft}/(3{,}155 \text{ ft/s})$$
$$t = \mathbf{7.0\ s}.$$

2. The increase in water pressure within the pipe:

   Since the valve was closed in 2 s, but the time for the wave round-trip is 7 s, this is equivalent to sudden closure.

$$(\Delta P) = \rho c(\Delta v) \qquad \textbf{(5.53)}$$

$$\begin{aligned}(\boldsymbol{\Delta P}) &= (\gamma/g)\,c\,(\Delta v)\\ &= (64.4/32.2)(3{,}155)(6.5\ -\ 0)\\ &= \mathbf{39{,}800\ lb/ft^2}.\end{aligned}$$

3. The tensile stress in the pipe, if the valve is closed in 2 s:

$$\begin{aligned}\boldsymbol{S_t} &= (\Delta P)\,r/t_k \qquad \textbf{(5.56)}\\ &= (39{,}800\ \mathrm{lb/ft^2})\,(24/12\ \mathrm{ft})/[(3/8)/12\ \mathrm{ft}]\\ &= 25.46\ \times 10^5\ \mathrm{lb/ft^2}\\ &= \mathbf{17{,}680\ lb/in.^2\ (psi\ increase\ in\ pressure)}.\end{aligned}$$

4. Discuss how the water hammer can be avoided.

   The closure time should be at least 7.0 s, preferably10 times larger than 2 s, that is, 20 s.

Solution 2 (SI System):

1. The time of travel of the pressure wave upstream and back (round-trip):

$$t = 2\ \mathrm{L/c} \qquad \textbf{(5.52)}$$

$$t = 2\ \times 3{,}353\ \mathrm{m}/(961.6\ \mathrm{m/s}).$$
$$\boldsymbol{t} = \mathbf{7.0\ s}.$$

2. The increase in water pressure within the pipe:

   Since the valve was closed in 2 s, but the time for the wave round-trip is 7 s, this is equivalent to sudden closure.

$$\begin{aligned}(\boldsymbol{\Delta P}) &= \rho c\,(\Delta v) \qquad \textbf{(5.53)}\\ &= (1{,}000)(961.6)(1.98 - 0)\\ &= 1{,}905{,}000\ \mathrm{N/m^2}\\ &= \mathbf{1{,}905\ kN/m^2}.\end{aligned}$$

3. The tensile stress in the pipe, if the valve is closed in 2 s:

$$\begin{aligned}\boldsymbol{S_t} &= (\Delta P)\,r/t_k \qquad \textbf{(5.56)}\\ &= (1{,}905{,}000\ \mathrm{N/m^2})\,(1.2192\ \mathrm{m}/2)/(9.525/1{,}000\ \mathrm{m})\\ &= 1.22\ \times\ 10^8\ \mathrm{N/m^2}\\ &= \mathbf{1.22\ \times\ 10^5\ kN/m^2\ increase\ in\ pressure}.\end{aligned}$$

4. Discuss how the water hammer can be avoided.

   Same as Solution 1.

5.10.3 Impulse–Momentum Principles

Change in direction or magnitude of flow velocity of a liquid leads to change in fluid's momentum. From kinetic mechanics, it is known that

linear impulse = change in linear momentum

$$\begin{aligned}\left(\sum F\right)t &= M(\Delta v)\\ &= M(v_2 - v_1)\\ \left(\sum F\right) &= \rho\,Q\,(v_2\ -\ v_1)\\ &= (\gamma/g)\,Q\,(v_2\ -\ v_1)\end{aligned} \qquad \textbf{(5.58)}$$

where

M = mass whose momentum changed in time t, slug (kg)
t = time, s
$\sum F$ = sum of linear impulse forces, lb (N)
v_2 = exit velocity, ft/s (m/s)
v_1 = entrance velocity, ft/s (m/s)
Q = flow, ft^3/s (m^3/s)
g = acceleration due to gravity, 32.2 ft/s^2 (9.81 m/s^2)
γ = specific weight of fluid, 62.4 lb/ft^3 (9.8 kN/m^3)
ρ = density, slug/ft^3 (kg/m^3)

In the horizontal x-direction:

$$(Mv_x)_1 \pm \left(\sum F_x\right) t = (Mv_x)_2 \tag{5.59}$$

In the horizontal x-direction:

$$(Mv_y)_1 \pm \left(\sum F_y\right) t = (Mv_y)_2 \tag{5.60}$$

where

$(Mv_x)_1$ = initial linear momentum in x-direction, slug-ft/s (kg-m/s)
$(Mv_y)_1$ = initial linear momentum in y-direction, slug-ft/s (kg-m/s)
$(Mv_x)_2$ = final linear momentum in x-direction, slug-ft/s (kg-m/s)
$(Mv_y)_2$ = final linear momentum in y-direction, slug-ft/s (kg-m/s)
$\sum F_x$ = sum of linear impulse forces in x-direction, lb (N)
$\sum F_y$ = sum of linear impulse forces in y-direction, lb (N)

EXAMPLE 5.60 IMPULSE–MOMENTUM ANALYSIS

A 24 in. (610 mm) pipe is connected to a 12 in. (305 mm) pipe by a standard pipe reducer fitting. The water flow rate is 31.4 ft^3/s (0.90 m^3/s) and the water pressure in the 24 in. pipe is 40 lb/in.^{2} (278 kN/m^2). Determine the force that is exerted by water on the reducer neglecting the minor head loss.

Solution 1 (US Customary System):

$Q = A_1 v_1 = A_2 v_2$.
31.4 ft^3/s = $[(\pi/4)(24/12)^2$ ft$^2]v_1 = [(\pi/4)(12/12)^2$ ft$^2]v_2$.
v_1 = 10 ft/s.
v_2 = 40 ft/s.
$P_1/\gamma + (v_1)^2/2g + z_1 = P_2/\gamma + (v_2)^2/2g + z_2 + h_f$.
$40 \times 144/62.4 + (10)^2/(2 \times 32.2) + 0 = P_2/\gamma + (40)^2/(2 \times 32.2) + 0 + 0$.
$P_2/\gamma = 40 \times 144/62.4 + (10)^2/(2 \times 32.2) - (40)^2/(2 \times 32.2) = 69$ ft of water.
P_2 = 69 ft × 62.4 lb/ft^3 = 4,306 lb/ft^2 = 30 psi.
$F_1 = P_1 A_1$ = (40 × 144 lb/ft^2)$(\pi/4)(24/12)^2$ = 18,086 lb toward the right.
$F_2 = P_2 A_2$ = (4,306 lb/ft^2)$(\pi/4)(12/12)^2$ = 3,380 lb toward the left.

$$\left(\sum F\right) = (\gamma/g)\,Q\,(v_2 - v_1) \tag{5.58}$$

(18,086 lb – 3,380 lb – F_x) = (62.4/32.2)(31.4)(40 – 10).
F_x = 14,706 – 1,825 = 12,881 lb to the left acting on the water.

The force that is exerted by water on the reducer = 12,881 lb to the right.
The vertical component F_y is 0.

Solution 2 (SI System):

$Q = A_1\, v_1 = A_2\, v_2$.
0.90 m^3/s = $[(\pi/4)(0.610)^2$ m$^2]v_1 = [(\pi/4)(0.305)^2$ m$^2]v_2$.
v_1 = 3.05 m/s.

$v_2 = 12.2$ m/s.

$P_1/\gamma + (v_1)^2/2g + z_1 = P_2/\gamma + (v_2)^2/2g + z_2 + h_f$.

$278/9.8 + (3.05)^2/(2 \times 9.8) + 0 = P_2/\gamma + (12.2)^2/(2 \times 9.8) + 0 + 0$.

Here $\gamma = 9.8$ kN/m^3, and g = 9.8 m/s^2

$P_2/\gamma = 278/9.8 + (3.05)^2/(2 \times 9.8) - (12.2)^2/(2 \times 9.8) = 21$ m of water.

$P_2 = 21 \text{ m} \times 9.8 \text{ kN/m}^3 = 195 \text{ kN/m}^2$.

$F_1 = P_1A_1 = (278 \text{ kN/m}^2)(\pi/4)(0.61)^2 \text{ m}^2 = 81$ kN toward the right

$F_2 = P_2A_2 = (195 \text{ kN/m}^2)(\pi/4)(0.305)^2 \text{ m}^2 = 14$ kN toward the left.

$$\left(\sum F\right) = (\gamma/g)\,Q\,(v_2 - v_1) \tag{5.58}$$

$(81 - 14 - F_x) = (9.8/9.8)(0.90)(12.2 - 3.05)$.

$F_x = 67 - 8 = 59$ kN to the left acting on the water.

The force that is exerted by water on the reducer = 59 kN to the right.

The vertical component F_y is 0.

5.10.4 Drag and Lift Forces

Drag is the component of the resultant hydraulic force exerted by a fluid on a subject parallel to the relative motion of the fluid. Lift is the component of the resultant hydraulic force exerted by the fluid on a subject perpendicular to the relative motion of the fluid.

The drag and lift forces are given by the following equations:

$$F_D = C_D \gamma A\,(v^2/2\,g) \tag{5.61}$$

$$F_L = C_L\,\gamma A(v^2/2g) \tag{5.62}$$

where

F_D = drag force, lb (N)
F_L = lift force, lb (N)
C_D = drag coefficient, dimensionless
C_L = lift coefficient, dimensionless
A = area projected on a plane perpendicular to the relative motion of the fluid, ft^2 (m^2)
v = relative velocity of the fluid with respect to the subject, ft/s (m/s)
g = acceleration due to gravity, 32.2 ft/s^2 (9.81 m/s^2)
γ = specific weight of fluid, 62.4 lb/ft^3 (9.8 kN/m^3)

EXAMPLE 5.61 DETERMINATION OF DRAG FORCE

A flat plate 4 ft by 4 ft (1.22 m by 1.22 m) moves at 23 ft/s (7 m/s) normal to its plane. Determine the resistance to the plate moving through water at 60°F (15.6°C). Assume a drag coefficient = 1.16 for length/width = 1.

Solution 1 (US Customary System):

At 60°F, $\gamma = 62.37$ lb/ft^3 (Appendix 3)

$$\begin{aligned} F_D &= C_D\,\gamma\,A\,(v^2/2\,g) \qquad (5.61)\\ &= 1.16\,(62.37 \text{ lb/ft}^3)(4 \times 4 \text{ ft}^2)\,(23 \text{ ft/s})^2/(2 \times 32.2 \text{ ft/s}^2)\\ &= \mathbf{9{,}513\ lb}. \end{aligned}$$

Solution 2 (SI System):

At 15.6°C, $\gamma = 9.798$ kN/m^3 (Appendix 3)

$$\begin{aligned} F_D &= C_D\,\gamma\,A\,(v^2/2g) \qquad (5.61)\\ &= 1.16\,(9.798 \text{ kN/m}^3)(1.22 \times 1.22 \text{ m}^2)\,(7 \text{ m/s})^2/(2 \times 9.8 \text{ m/s}^2)\\ &= \mathbf{42.4\ kN}. \end{aligned}$$

PROBLEMS/QUESTIONS

5.1 A water supply system is shown in Fig. 5.27. The values of K (in $h_f = KQ^2$) are indicated in the figure. The total discharge rate from the system is 9 ft^3/s (255 L/s). The residual pressure at point C is to be maintained at 20 psi (139 kPa). Find

(a) The flow rate in each pipe

(b) The elevation of water surface in the reservoir

(c) The pressures at A, B, and E

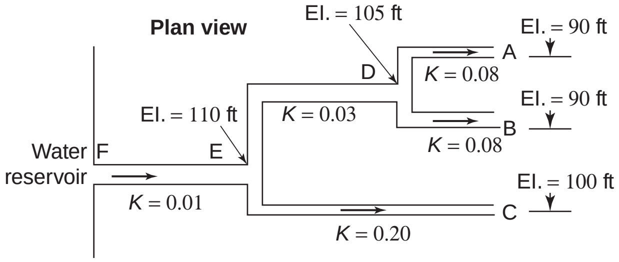


Figure 5.27 Water system for Problem 5.1. Conversion factor: 1 ft = 0.3048 m.

5.2 A village is investigating a new water supply plan. The supply is to consist of a dam impounding water at A, from which it must be pumped over a summit at B. From B it runs by gravity to the village at D (see Fig. 5.28). Since the head available at the village is more than would be used, putting in a hydroelectric plant at C has been suggested. The electric power will be transmitted back to the pumping station and used for pumping water up to B.

Assume the following efficiencies: turbine 80%, generator 90%, transmission line 95%, pump motor 80%, pump 75%. If the rate of water use is to be 6.28 ft^3/s (178 L/s), determine the excess water horsepower still available at C.

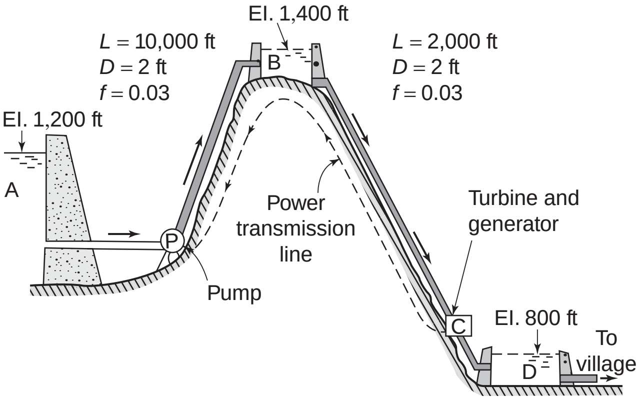


Figure 5.28 Water system for Problem 5.2. Conversion factor: 1 ft = 0.3048 m.

5.3 In the sketch shown in Fig. 5.29, C and D represent points on a university campus that are connected to a common junction B through 100 and 200 m lengths of pipe, respectively. Points B and C are on the lower level of campus at elevation of 12 and 6 m, respectively, above mean sea level (MSL), while point D is on the upper level of the campus at an elevation of 66 m above MSL. The source of supply is at 300 m from the junction at point A. Pipe diameters are 5, 4, and 2.5 cm as indicated in the figure ($g = 9.8\ \text{m/s}^2$; $\rho = 1\ \text{gm/cm}^3$; $f = 0.0196$).

(a) With C closed off and D freely flowing, determine the total static head required at A for a flow of 2.64 L/s at D.

(b) With both points C and D open to the atmosphere, determine the maximum quantity of water that can be provided at point C before any water starts to flow out at D.

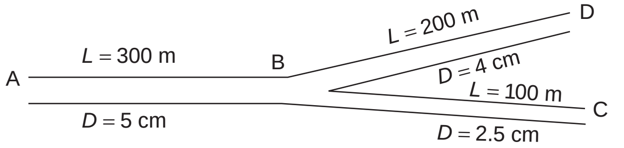


Figure 5.29 Water system for Problem 5.3.

5.4 The elevation of a city distribution reservoir is 400 ft (122 m) and that of the city is 250 ft (76 m). When city water consumption is at the rate of 16 ft^3/s (453 L/s), the pressure in the city is 60 psi (416 kPa).

Assuming that during a fire the minimum required pressure is 20 psi (139 kPa), determine the fire flow available if the coincidental city demand is 14 ft^3/s (396 L/s). (Neglect velocity heads and the variation of f with Reynolds number.)

5.5 Given the system shown in Fig. 5.30 and neglecting minor losses, velocity heads, and the variation of f with Reynolds number:

(a) What will be the flow rate from A to D if the pressure at D is to be maintained at 20 psi (139 kPa)?

(b) What will the rate of flow be if the water level in reservoir A drops 10 ft (3 m), that is, to El. 170 ft (51.8 m)?

(c) What size of ductile iron pipe ($f = 0.020$) is required to carry an additional 22 ft^3/s (623 L/s) from A to D with the reservoir at 180 ft (55 m) and residual pressure at D to be maintained at 20 psi (139 kPa)?

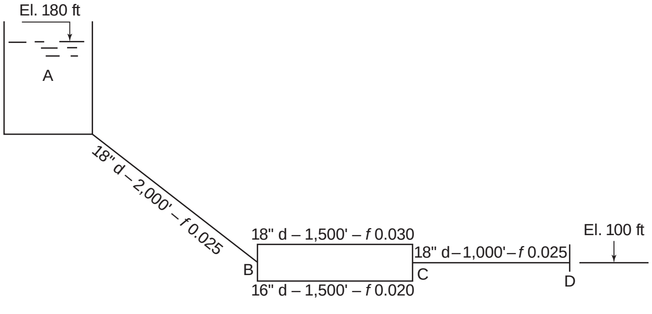


Figure 5.30 Water system for Problem 5.5. Conversion factors: 1′ = 1 ft = 0.3048 m; 1″ = 1 in. = 25.4 mm.

5.6 A pipe 2 ft (600 mm) in diameter and 3 mi (4.83 km) long connects two reservoirs. Water flows through it at a velocity of 5 ft/s (1.5 m/s). To increase the flow, a pipe 1.5 ft (450 mm) in diameter is laid parallel alongside the first pipe for the last mile.

Find the increase in flow between the reservoirs assuming the same value of f for the two pipes. Also assume that the water levels in the reservoirs remain the same. (1 mi = 5,280 ft or 1,610 m.)

5.7 In the water supply system shown in Fig. 5.31, the pumping station is operating at a flow capacity of 125 L/s against a total dynamic head of 50 m. Determine the maximum depth of water that can be stored in the elevated reservoir.

Elevation of water level in underground tank = 500 m.

Elevation of bottom of elevated reservoir = 520 m.

C for all pipes = 100.

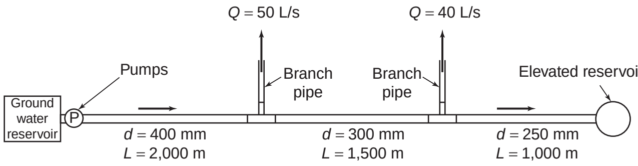


Figure 5.31 Water system for Problem 5.7.

5.8 In the water supply system shown in Fig. 5.32, water flows by gravity from reservoir A to points C and D as well as to reservoir E. The elevations of known points are as follows:

Water level A = 650.00 m.

Elevation B = 590.00 m.

Elevation C = 550.00 m.

Elevation D = 570.00 m.

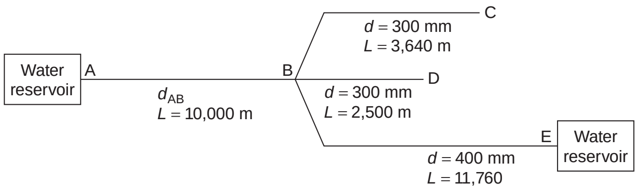


Figure 5.32 Water system for Problem 5.8.

The measured residual pressures are as follows:

Point B = 300 kPa.

Point C = 500 kPa.

The known flows are as follows:

Pipe BD = 30 L/s.

Pipe BE = 75 L/s.

C for all pipes = 100.

(a) Calculate the flow in line BC.

(b) Find the residual pressure at point D.

(c) Compute the maximum water level in reservoir E.

(d) Determine the required diameter for pipe AB.

5.9 A water supply system consists of a ground reservoir with lift pumps (A), elevated storage (C), a withdrawal point (B), and equivalent pipelines as given in the plan view in Fig. 5.33. Assume the following elevations:

Water level in ground reservoir (A) = 600.00 m.

Ground level at withdrawal point (B) = 609.10 m.

Ground level at elevated water tank (C) = 612.20 m.

Water level in tank (C) = 30 m above ground.

The lift pumps provide flow at a discharge pressure of 550 kPa. The value of *C* for all pipes is 100.

Compute the discharge *Q* in L/s at withdrawal point B for the following conditions:

(a) If the residual pressure at point B is 200 kPa

(b) If water is neither flowing to, nor from, the elevated storage tank

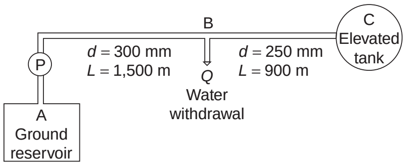


Figure 5.33 Water system for Problem 5.9.

5.10 The water supply system shown in Fig. 5.34 is designed to deliver water from the pumping station to city ABC. The water level in the wet well at the pumping station is at 1,000 ft.

The head delivered by the pumps to the water is 100 ft. The value of *C* for all pipes is 100. Assume the following elevations:

A. 900.00 ft.

B. 915.00 ft.

C. 910.00 ft.

(a) Determine the required diameter of main PA, if the pressure at point A is not allowed to drop below 52 psi.

(b) Determine the required diameter of pipe BC, if the maximum allowed head loss in the network ABC is 3 ft/1,000 ft.

(c) Calculate the actual residual pressures at points A, B, and C.

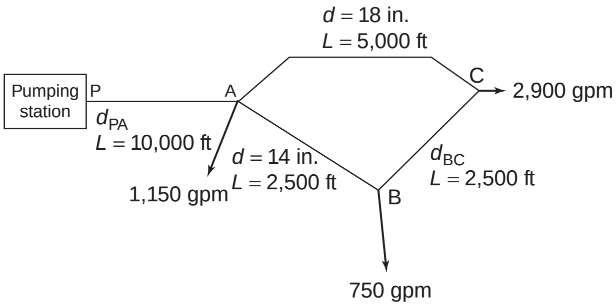


Figure 5.34 Water system for Problem 5.10. Conversion factors: 1 gpm = 3.785 L/min; 1 ft = 0.3048 m;1 in. = 25.4 mm.

5.11 Water is pumped from ground reservoir A to the elevated reservoir G through the network of pipes shown in Fig. 5.35.

The following data are given:
All pipes have the same diameter = 300 mm.
C for all pipes = 100.
Water level in ground reservoir = 500 m.
Water flow rate = 80 L/s.
Pump operating head = 70 m.

(a) Calculate the maximum water level in the elevated reservoir when valves E and F are closed.

(b) How many meters will the water level rise in the elevated reservoir if valves E and F were opened?

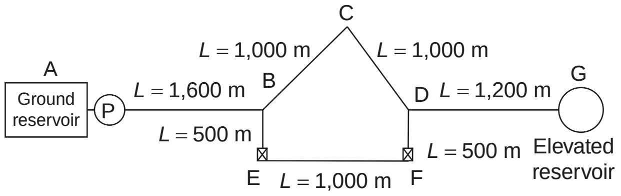


Figure 5.35 Water system for Problem 5.11.

5.12 In the water supply system shown in Fig. 5.36, the pumping station (P) delivers the water from the water treatment plant (WTP) to city ABCDE. The water head delivered by the pumps is 28 m. The water level in the pumping station wet well is at 600.00 m. The value of C for all pipes is 100.

The following elevations are known:

A. 570.00 m.

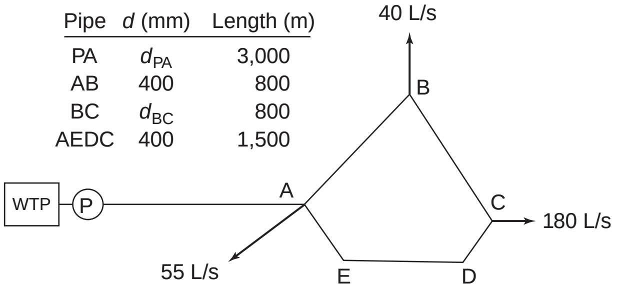
B. 575.00 m.

C. 573.00 m.

(a) Find the required diameter of the force main PA (d_{PA}), if the pressure at point A is not allowed to drop below 370 kPa.

(b) Determine the required diameter of pipe BC (d_{BC}), if the maximum allowed head loss in the network ABC is 3‰.

(c) Calculate the actual residual pressures at points A and C.



Pipe	d (mm)	Length (m)
PA	d_{PA}	3,000
AB	400	800
BC	d_{BC}	800
AEDC	400	1,500

Figure 5.36 Water system for Problem 5.12.

5.13 In the water supply system shown in Fig. 5.37, water is pumped from the ground reservoir to the village ABCD (elevation of 120.00 m) through the water main PA. The value of C for all pipes is 100. A water meter located on pipe DC indicates a flow of 20 L/s. The following network data are provided:

Pipe	d (mm)	L (m)
PA	500	10,000
AB	250	500
AC	200	700
AD	200	500
BC	250	1,000
CD	150	1,000

(a) Determine the flow in each pipe as well as the flow delivered to point C (Q_c).

(b) Calculate the pumping head, if the residual pressure at point C is not to be less than 150 kPa.

(c) Find the residual pressure at point C if a different pump is used, one that can deliver half the flow into the system at the same pumping head above.

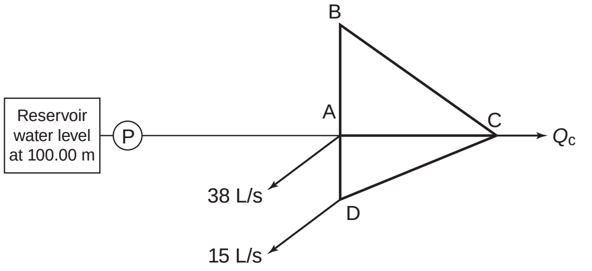


Figure 5.37 Water system for Problem 5.13.

5.14 Consider a vertical rectangular gate ($b \times h$ = 13.12 ft × 6.56 ft = 4 m × 2 m) that is vertically submerged in water so that the top of the gate is 13.12 ft (4 m) below the water surface. Determine the total resultant force on the gate and the location of the center of pressure (see Fig. 5.6).

5.15 In Fig. 5.3 the areas of the plunger A and cylinder B are 6 in.^{2} (38.71 cm^2) and 600 in.^{2} (3,871 cm^2), respectively, and the weight of B is 9,000 lb (4,086 kg). The vessel and the connecting passages are filled with oil of specific gravity 0.75. What force is required for equilibrium, neglecting the weight of A and assuming d = 16 ft (4.88 m).

5.16 Gate EBC shown in Fig. 5.13 is hinged at B and is 4 ft (1.22 m) wide. Neglecting the weight of the gate, determine the unbalanced moment due to water acting on the gate EBC. It is assumed that $\theta = 60°$, d = 8 ft (2.44 m), and a = 3 ft (0.91 m).

5.17 Oil of specific gravity 0.75 flows through the nozzle shown in Fig. 5.1 and deflects the mercury (specific gravity 13.57) in the U-tube gauge. Determine the value of h if the pressure at D is 20 psi (138.8 kN/m^2) and m is 2.75 ft (0.84 m).

5.18 The funnel shown in Fig. 5.38 is full with water. The volume of the upper part is 5.9 ft^3 (0.1671 m^3) and of the lower part is 2.6 ft^3

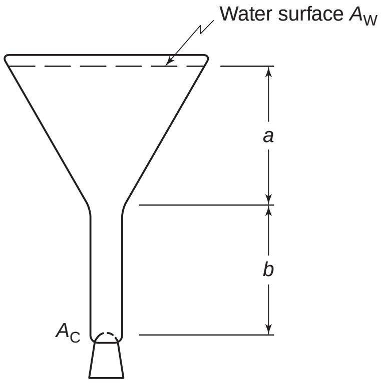


Figure 5.38 Funnel for Problem 5.18.

(0.0736 m^3). The cross-sectional area A_C of the lower part is 0.5 ft^2 (0.0464 m^2). The height of the upper part $a = 5$ ft (1.524 m), the height of the lower part b = 5 ft (1.524 m) and the water surface area A_W of the upper part is 20 ft^2 (1.8581 m^2). Determine the force tending to push the plug out.

5.19 Determine the components of the forces P_x and P_y due to the water acting on the curved area AB in Fig. 5.39 per unit length ft (m). Also locate the components of these forces P_x and P_y. The radius r is 6.5 ft (1.98 m). The center of gravity of a quadrant of a circle is located at a distance $4r/3\pi$ from either mutually perpendicular radius.

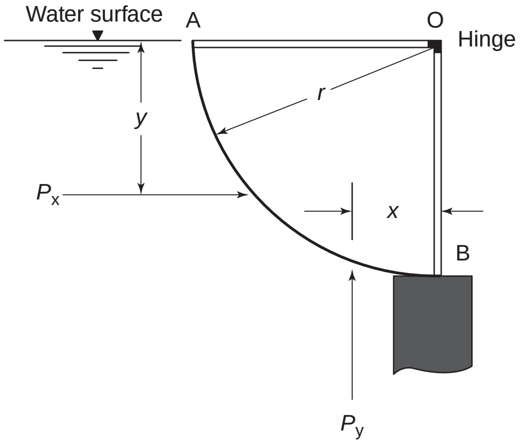


Figure 5.39 Hydrostatic force analysis.

5.20 Determine (1) the total resultant force acting on an inclined rectangular gate shown in Fig. 5.7, and (2) the location of the pressure center. Assume $a = 5$ ft (1.52 m), $b = 6$ ft (1.83 m), $h = 4$ ft (1.22 m), and $\theta = 60°$.

5.21 An object is 1.5 ft (0.4572 m) wide by 2.5 ft (0.762 m) long and 1.5 ft (0.4572 m) high. It weighs 150 lb (68.1 kg) at a water depth of 10 ft (3.048 m). What is the weight of the object in air and what is its specific gravity?

5.22 A prismatic object is 8 in. (20.32 cm) thick, 8 in. (20.32 cm) wide, and 16 in. (40.64 cm) long. It weighs 12 lb (5.45 kg) at a water depth of 20 in. (50.8 cm). What is the weight of the object in air and what is its specific gravity?

5.23 A horizontal pipe carrying oil of specific gravity 0.8 is 6 in. (15.24 cm) in diameter. Near the end E, the pressure is measured to be 13.2 psi (91.6 kPa). After E, the 6 in. (15.24 cm) pipe turns 90° upwards for 12 ft (3.66 m) reaching another 90° bend, which connects to an upper horizontal pipe having a diameter of 18 in. (45.72 cm) at point R where the pressure is also measured to be 8.75 psi (60.73 kPa) (Fig. 5.40). Determine

1. The oil velocities in the two horizontal pipes
2. The direction of oil flow
3. The head loss between E and R, if the oil discharge is 5 ft^3/s (0.1415 m^3/s)

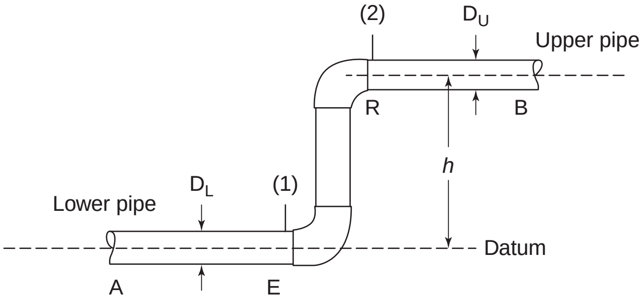


Figure 5.40 Pipe line analysis.

5.24 In the sketch shown in Fig. 5.41 a barrel whose empty weight is 60 lb (27.2 kg) contains an unknown quantity of water in which a container holding 200 lb (90.8 kg) of oil (specific gravity = 0.95) floats, and a beaker containing 10 lb (4.54 kg) of mercury (specific gravity = 13.00) floats in the oil. The beaker, the oil container, and the barrel are 6, 24, and 36 in. (0.1524, 0.6096, and 0.9144 m) in diameter, respectively. The total weight indicated by the scale is 860 lb (390.44 kg). Determine the height h of the water in the barrel.

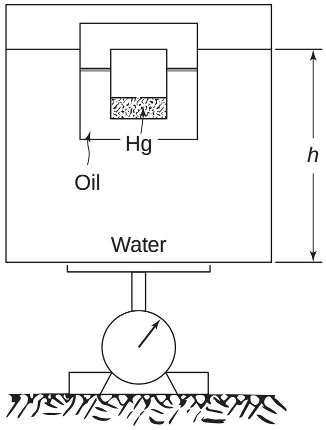


Figure 5.41 Water barrel, oil container, and mercury beaker.

5.25 Determine the force in lb (N) that each connection between the 90° elbow and the 12-in. (0.3048-m)-diameter pipe line in Fig. 5.42 will resist. Assume the water in the system is static (no flow condition) and the pipeline is pressured to 100 psig (694 kPa gauge).

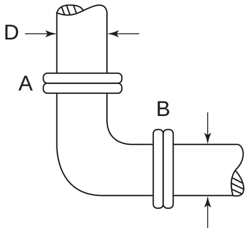


Figure 5.42 Elbow connecting two pipes.

5.26 A conical reducing section shown in Fig. 5.43 connects an existing 6 in. (152.4 mm) pipeline with a new 4-in. (101.6-mm)-diameter pipeline. Assume the pipeline is at 100 psig (694 kPa = 694 kN/m^2) static pressure under no flow conditions and there is no

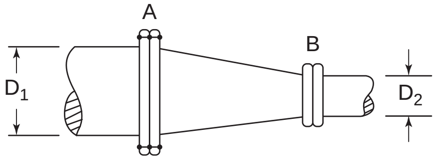


Figure 5.43 Conical reducer connecting two pipes.

end restraint from the pipes. Determine the tensile force in lb (kN), which is exerted on the connecting reducer joint.

5.27 A rectangular access port 2 ft by 2 ft (0.6096 m by 0.6096 m) in size, shown in Fig. 5.44, seals an environmental test chamber that is pressurized to 16 psi (111.04 kPa = 111.04 kN/m^2) above external pressure. What force in lb (kN) does the port exert upon its retaining structure?

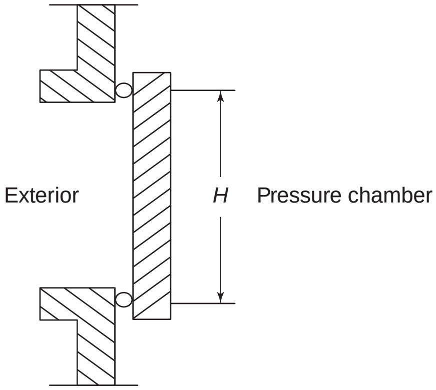


Figure 5.44 Access port of a pressure chamber.

5.28 Commercial chemical liquid stored in an above-ground spherical steel tank at 102°F (38.89°C) generates a gauge pressure of 80 psig (555.2 kPa = 555.2 kN/m^2). If the chemical storage tank is 6 ft (1.829 m) in diameter and has walls 0.25 in. (6.35 mm) thick, determine the maximum tensile stress in psi (kN/m^2) developed in the steel.

5.29 A steel-reinforced fiber glass cylinder butane storage tank 16 ft (4.88 m) long shown in Fig. 5.45 has hemispherically domed ends 6 ft (1.829 m) in diameter. Butane has a vapor pressure of 46 psig (319.24 kN/m^2 = 319.24 kPa) at 102°F (38.89°C) and the tank walls are 0.25 in. (6.35 mm) thick. Determine the maximum tensile stress in lb/in^2 (kN/m^2) developed in the tank.

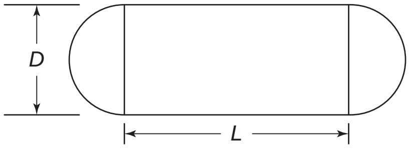


Figure 5.45 Cylindrical liquid storage with hemispherical domed ends.

5.30 Ice in an iceberg has a specific gravity of 0.91. Seawater has a specific gravity of 1.03. Determine the iceberg's percentage volume exposed in air when it is floating in seawater.

5.31 A vertical sliding gate 25 ft (7.62 m) wide and 35 ft (10.668 m) high is submerged in 45 ft (13.76 m) of water. It has a coefficient of friction equals 0.2 between its guides and its edges. The gate weighs 6.5 tons (13,000 lb = 5,902 kg). Assume friction is due to the normal force of water pressure on the gate and neglect the buoyant force of the water. Determine the vertical force that is required to just lift the gate. The following values for the distances in Fig. 5.46 are a = 10 ft (3.048 m); b = 27.5 ft (8.382 m); c = 45 ft (13.716 m); and h = 35 ft (10.668 m).

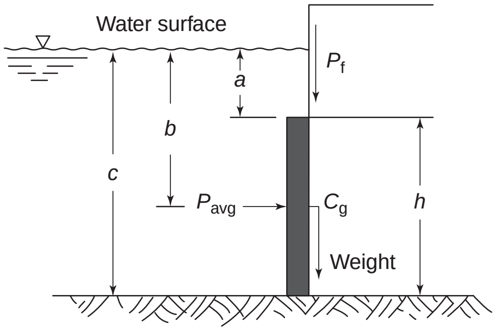


Figure 5.46 Vertical force to lift a sliding gate.

5.32 A cylinder of cork is floating upright in a container partially filled with water. A vacuum is applied to the container such that the air within the vessel is partially removed. The cork will

1. Rise somewhat in the water
2. Sink somewhat in the water
3. Remain stationary
4. Turn over on its side
5. Sink to the bottom of the container

Select the correct answer and explain why.

5.33 An incompressible fluid ($\gamma = 52$ lb/ft^3 = 8.175 kN/m^3) enters and leaves a hydraulic system with the following energy in ft-lb/lb (m-kg/kg) of fluid:

1. Potential energy z above datum:

   Entering = 6 ft = 1.8288 m.
   Leaving = 16 ft = 4.8768 m.

2. Kinetic energy, $v^2/2g$:

   Entering = 6 ft = 1.8288 m.
   Leaving = 11 ft = 3.3528 m.

3. Pressure energy, P/γ:

   Entering = 32 ft = 9.7536 m.
   Leaving = 152 ft = 46.3296 m.

4. Total energy:

Entering = 44 ft = 13.4112 m.
Leaving = 179 ft = 55.5592 m.

Determine the pressure increase in psi (kN/m^2) between entering and leaving liquid streams.

5.34 Consider the parallel pipe system in Fig. 5.47. The following data are known:

Pipe c is a 10 in. (254 mm) water line.
Pipe d is a 12 in. (304.8 mm) water main.
Pipe a is a 6 in. (152.4 mm) line, 1,000 ft (304.8 m) long.
Pipe b is a 6 in. (152.4 mm) line, 1,440 ft (438.9 m) long.
Water velocity in pipe b is 10 ft/s (3.048 m/s).

Friction factors in the two pipes a and b are the same and the incidental losses are equal. Determine the water velocity in pipe a.

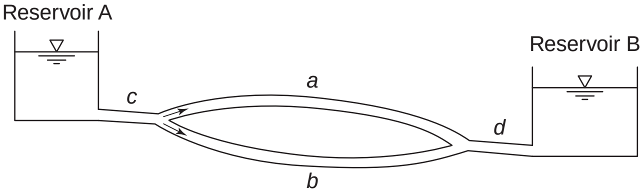


Figure 5.47 Parallel water pipes.

5.35 The expression for the Reynolds number R for a circular pipe with circular cross-section was given in Eq. (5.11) as follows:

$$\boldsymbol{R} = vd\rho/\mu = vd/v \qquad \textbf{(5.11)}$$

where d is the pipe diameter, v is the water velocity, μ is the absolute viscosity, $\underline{v} = \mu/\rho$ is the kinematic viscosity of the fluid, and ρ is its density. Develop an expression for Reynolds number for an open channel with rectangular cross-section in terms of the hydraulic radius instead of the pipe diameter.

5.36 Summarize the differences between laminar flow and turbulent flow in terms of

1. Motion of fluid particles
2. Energy loss
3. Velocity distribution in pipe
4. Reynolds number

5.37 The vena contracta of a sharp-edged hydraulic orifice usually occurs (select the correct answer)

1. At the geometric center of the orifice
2. At a distance of about 20% of the orifice diameter upstream from the plane of the orifice
3. At a distance equal to about one orifice diameter downstream from the plane of the orifice
4. At a distance equal to about one-half the orifice diameter downstream from the plane of the orifice
5. At a distance within 20% of the orifice diameter upstream from the plane of the orifice

5.38 Determine the coefficients of velocity, discharge, and contraction for a jet of liquid flow through an orifice. Assume the actual velocity in the contracted section of the liquid jet flowing from a 2-in. (50.8-mm)-diameter orifice is 30 ft/s (9.144 m/s), under a head of 16 ft (4.877 m). Actual flow is 0.4 ft^3/s (0.0113 m^3/s).

5.39 A flat plate, 4 ft by 4 ft (1.22 m by 1.22 m), moves at 23 ft/s (7.01 m/s) normal to its plane at standard pressure. Determine the resistance of the plate assuming the drag coefficient = 1.16 for length/width ratio equal to 1 and γ_{air} = 0.0752 lb/ft^3 (0.01181 kN/m^3).

5.40 A standard orifice discharges under a head H as shown in Fig. 5.48. Apply Bernoulli equation from W to J, with datum at J. Assume the head loss of orifice is represented by Eq. (5.49):

$$h_{\text{f}} = \{[1/(C_v)^2] - 1\}(v_{\text{jet}})^2/2g \qquad \textbf{(5.49)}$$

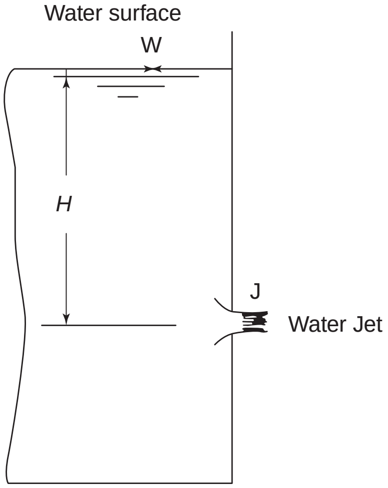


Figure 5.48 Standard orifice.

1. Develop the jet velocity (Eq. 5.47b):

$$v_{\text{jet}} = v = C_v(2gH)^{0.5} \qquad \textbf{(5.47a)}$$

2. Develop the jet flow rate (Eq. 5.45):

$$Q = C_{\text{d}}A\,(2gH)^{0.5} \qquad \textbf{(5.45)}$$

5.41 The pitot tube shown in Fig. 5.49 is used to measure the pressure at a point where the velocity is zero. This point is technically called the stagnation point. The pressure there is called the stagnation (or total) pressure. Assume the tube is shaped and positioned properly; a point of zero velocity is developed at B in front of the open end of the tube. Assume H_{A} and H_{B} are known, and there is no head loss. Apply the Bernoulli equation from A to B in Fig. 5.49, datum at B. Develop the equations for the determination of the velocity at A (v_{A}) and the pressure at B (P_{B}).

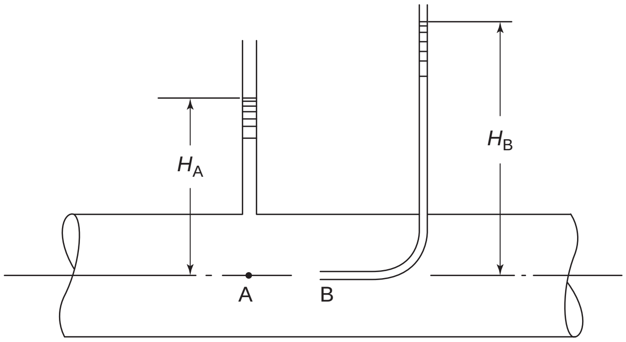


Figure 5.49 Standard pitot tube.

5.42 Determine the flow from a standard orifice J shown in Fig. 5.48. The head above the orifice $H = 20$ ft (6.096 m), the orifice diameter equals 4.5 in. (114.3 mm), and discharge coefficient $C_d = 0.594$.

5.43 The pitot tube shown in Fig. 5.49 has a coefficient of 0.9850. It is used to measure the velocity of water at the center of a pipe. Assume a stagnation pressure head H_B of 19 ft (5.7912 m) and the static pressure head H_A in the pipe of 16 ft (4.8768 m). Determine the velocity at A and the pressure at B.

5.44 Given an over-simplified Moody diagram (Fig. 5.50) and the original Moody diagram (Fig. 5.11), explain the applications of curves A, B, C, D, and E.

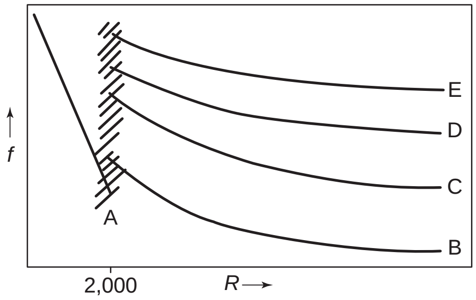


Figure 5.50 Simplified Moody diagram.

5.45 Briefly and precisely define

1. The Moody diagram
2. The Bernoulli equation
3. The Darcy–Weisbach formula
4. The Chezy formula
5. Hazen–Williams formula
6. Manning equation

5.46 Briefly and precisely define

1. Pitot tube
2. Stagnation pressure
3. Hydraulic gradient
4. Energy gradient
5. Friction head loss

5.47 At point A in a pipeline, the elevation is 155 ft (47.244 m); the pressure is 35 psig (242.9 kPa = 242.9 kN/m^2). At point B 5,000 ft (1,524 m) downstream of the pipe, the elevation is 135 ft (41.148 m) and the pressure is 40 psig (277.6 kPa = 277.6 kN/m^2). Determine the head loss between points A and B.

5.48 A 24 in. (609.6 mm), 5,000 ft (1524 m) pipeline carries 1.6 ft^3/s (0.453 m^3/s) of water between points A and B. The head loss between A and B is 8.46 ft (2.56 m). Determine the velocity of flow and the pipe friction factor *f*.

5.49 A water jet shown in Fig. 5.51 flows vertically upward from a nozzle with a vertical velocity of 16 ft/s (4.8768 m/s) and a flow rate of 0.03 ft^3/s (0.0008496 m^3/s). Above the nozzle at a distance $h = 6$ in. (152.4 mm), there is a horizontal plate weighing 0.5 lb (0.227 kg). Determine the reaction force *F* which is required to hold the plate stationary.

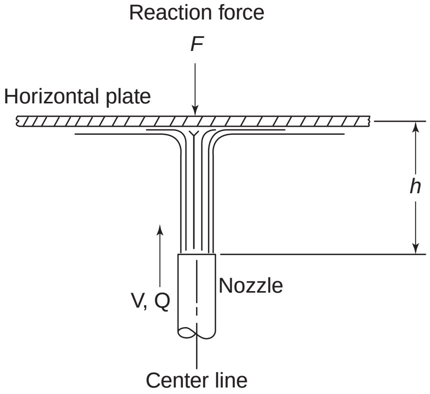


Figure 5.51 Water jet from a nozzle.

5.50 A rectangular wood block floats in water with 6.2 in. (157.48 mm) projecting above the water surface. If the same wood block is placed in an organic solvent of specific gravity 0.81, the wood block projects 4.2 in. (106.68 mm) above the surface of the organic solvent. Determine the specific gravity of the wood block.

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Chapter 6

Water Distribution Systems: Components, Design, and Operation

6.1 DISTRIBUTION SYSTEMS

Apart from a few scattered taps and takeoffs along their feeder conduits, distribution systems for public water supplies are networks of pipes within networks of streets. Street plan, topography, and location of supply works, together with service storage, determine the type of distribution system and the type of flow through it. Although service reservoirs are often placed along lines of supply, where they may usefully reduce conduit pressures, their principal purpose is to satisfy network requirements. Accordingly they are, in fact, components of the distribution system, not of the transmission system.

6.1.1 One- and Two-Directional Flow

The type of flow creates four systems, as sketched in Fig. 6.1. Hydraulic grade lines and residual pressures within the areas served, together with the volume of distribution storage, govern the pipe sizes within the network. It is plain that flows from opposite directions increase system capacity. With *two-directional flow* in the main arteries, a *pumped* or *gravity supply*, or a *service reservoir*, feeds into opposite ends of the distribution system or through the system to *elevated storage* in a reservoir, tank, or standpipe situated at the far end of the area of greatest water demand. Volume and location of service storage depend on topography and water needs.

6.1.2 Distribution Patterns

Two distribution patterns emerge from the street plan: (a) a *branching pattern* on the outskirts of the community, in which ribbon development follows the primary arteries of roads (Fig. 6.2a), and (b) a *gridiron pattern* within the built-up portions of the community where streets crisscross and water mains are interconnected (Figs. 6.2b and 6.2c). Hydraulically, the gridiron system has the advantage of delivering water to any spot from more than one direction and of avoiding dead-ends. The system is strengthened by substituting for a central feeder a *loop* or belt of feeders that supply water to the *congested*, or *high-value*, district from at least two directions. This more or less doubles the delivery of the grid (Fig. 6.2c). In large systems, feeders are constructed as pressure tunnels, pressure aqueducts, steel pipes, or reinforced-concrete pipes. In smaller communities the entire distribution system may consist of ductile-iron pipes. Ductile iron is, indeed, the most common material for water mains, but plastics, in general, in the case of small supplies, are also important.

6.1.3 Pipe Grids

The gridiron system of pipes stretching over all but the outlying sections of a community (Fig. 6.2) may consist of *single* or *dual mains*. In the Northern Hemisphere, single mains are customarily laid on the north and east sides of streets for protection against freezing. In the Southern Hemisphere, the south and east sides are used. Valves are generally installed as follows: three at crosses, two at tees, and one on single-hydrant branches. In dual-main systems, *service headers* are added on the south (north in Southern Hemisphere) and west sides of streets, and piping is generally placed beneath the sidewalks. Hydraulically, the advantages of dual-main systems over single-main systems are that they permit the arrangement of valves and hydrants in such ways that breaks in mains do not impair the usefulness of hydrants and do not *dead-end* mains.

Dual-main systems must not be confused with dual-water supplies: a high-grade supply for some purposes and a low-grade supply for others.

6.1.4 High and Low Services

Sections of the community too high to be supplied directly from the principal, or *low-service*, works are generally incorporated into separate distribution systems with independent piping and service storage. The resulting *high services* are normally fed by pumps that take water from the main supply and boost its pressure as required. Areas varying widely

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Water Engineering: Hydraulics, Distribution and Treatment, First Edition. Nazih K. Shammas and Lawrence K. Wang.

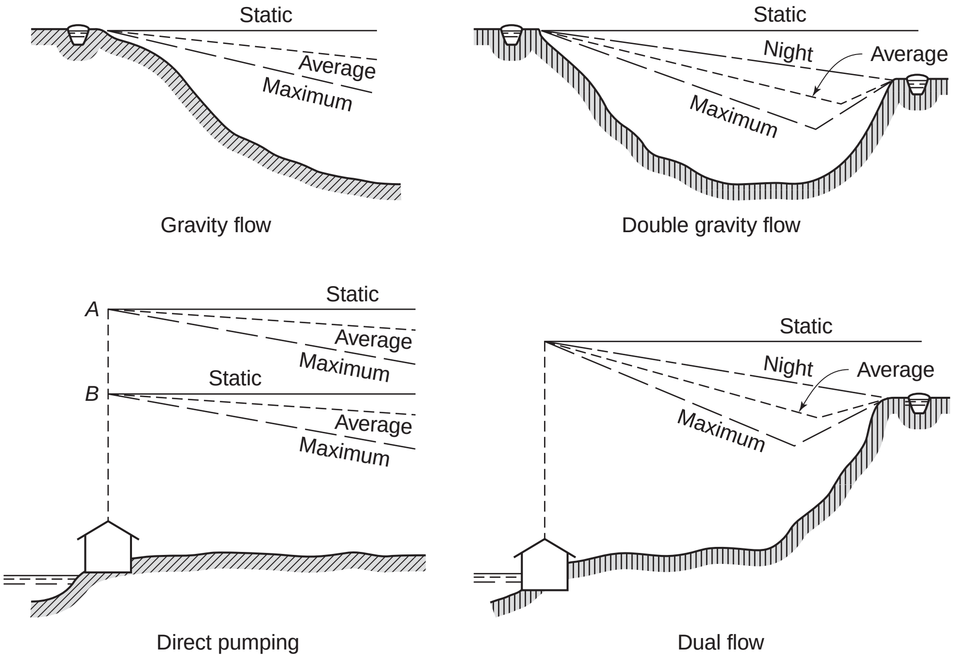


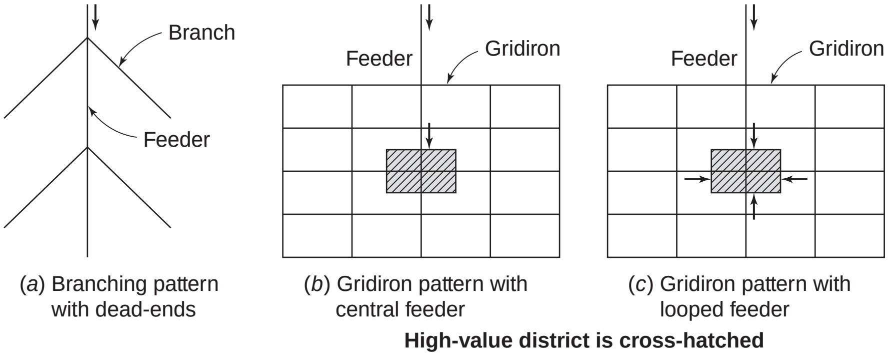
Figure 6.1 One- and two-directional flow in distribution systems.

in elevation may be formed into intermediate districts or zones. Gated connections between the different systems are opened by hand during emergencies or go into operation automatically through pressure-regulating valves. Because high-service areas are commonly small and low-service areas are commonly large, support from high-service storage during breakdowns of the main supply is generally disappointing.

Before the days of high-capacity, high-pressure, motorized fire engines, conflagrations in the congested central, or *high-value*, district of some large cities were fought with water drawn from independent high-pressure systems of pipes and hydrants (Boston, MA, still maintains a separate fire supply). Large industrial establishments, with heavy investments in plant, equipment, raw materials, and finished products, that are concentrated in a small area are generally equipped with high-pressure fire supplies and distribution networks of their own. When such supplies are drawn from sources of questionable quality, some regulatory agencies enforce rigid separation of private fire supplies and public systems. Others prescribe *protected cross-connections* incorporating *backflow preventers* that are regularly inspected for tightness.

6.1.5 Service to Premises

Water reaches individual premises from the street main through one or more *service pipes* that tap into the distribution system. The building supply between the public main and the takeoffs to the various plumbing fixtures or other points of water use is illustrated in Fig. 6.3. Small services are made of cement-lined iron or steel, brass of varying copper content, copper, and plastics such as polyethylene (PE) or polyvinyl chloride (PVC). Because lead and lead-lined pipes may corrode and release lead to the water, they are no longer installed afresh. For large services, coated or lined ductile-iron pipe is often employed. For dwellings and similar buildings, the minimum desirable size of service is 3/4 in. (19 mm). Pipe-tapping machines connect services to the main without shutting off the water. They also make large connections within water distribution systems.

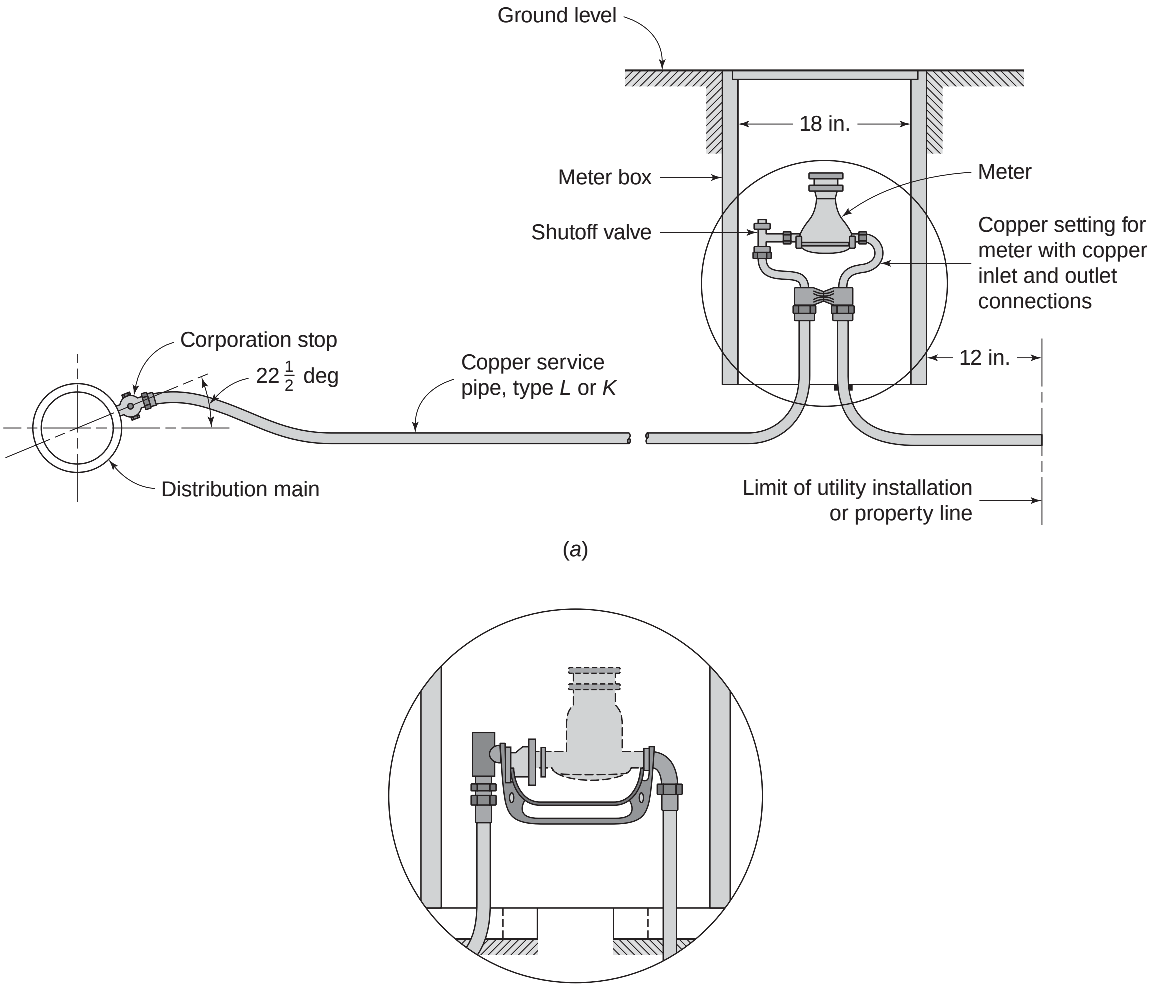


(*a*) Branching pattern with dead-ends

(*b*) Gridiron pattern with central feeder

(*c*) Gridiron pattern with looped feeder

Figure 6.2 Patterns of water distribution systems.



(*a*)

(*b*)

Figure 6.3 (a) Typical house service; (b) alternate method of mounting meter. Conversion factor: 1 in. = 25.4 mm.

6.2 SYSTEM COMPONENTS

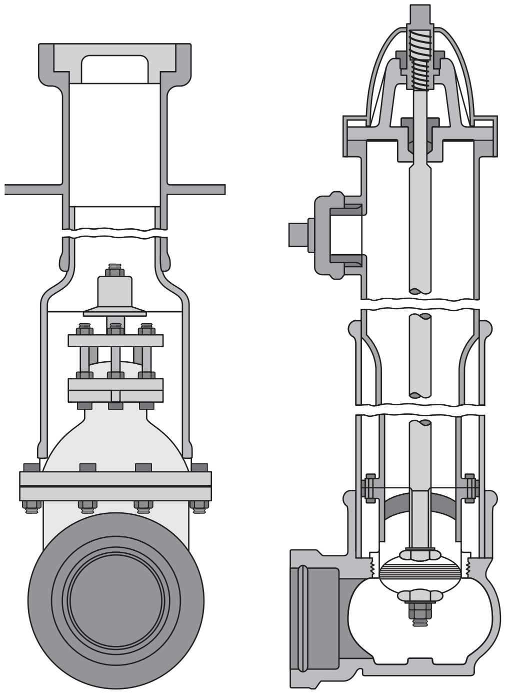
Pipes, *valves* (see Chapter 5), and *hydrants* are the basic elements of reticulation systems (Figs. 6.4, 6.5, 6.6, and 6.7). Their dimensioning and spacing are based on experience that is normally precise enough in its minimum standards to permit roughing in all but the main arteries and feeders. Common standards include the following:

Pipes	
Smallest pipes in gridiron	6 in. (150 mm)
Smallest branching pipes (dead-ends)	8 in. (200 mm)
Largest spacing of 6 in. grid (8 in. pipe used beyond this value)	600 ft (180 m)
Smallest pipes in high-value district	8 in. (200 mm)
Smallest pipes on principal streets in central district	12 in. (300 mm)
Largest spacing of supply mains or feeders	2,000 ft (600 m)

Gates	
Largest spacing on long branches	800 ft (240 m)
Largest spacing in high-value district	500 ft (150 m)
Hydrants	
Largest spacing between hydrants	See Table 6.1
Maximum distance from any point on street/road frontage to a hydrant	See Table 6.1

The choice of pipe sizes depends on occupancy of the properties along mains (whether residential, commercial, or industrial), their water uses, and the fire risks. Hazards at refineries, chemical plants, and lumber yards require special consideration.

According to the *International Fire Code* (IFC; 2006), fire hydrants are required to be provided where a building is more than 400 ft (122 m) from a hydrant, except for buildings equipped with automatic sprinkler systems for which the



(*a*) (*b*)

Figure 6.4 (a) Gate valve and extendable valve box; (b) post fire hydrant with compression valve.

Figure 6.5 Fire hydrant (*Source*: http://en.wikipedia.org/wiki/Image:Downtown_Charlottesville_fire_hydrant.jpg).

Figure 6.6 Cast-iron gate valve being installed on a new connection to a fire hydrant (*Source*: http://en.wikipedia.org/wiki/Image:MunicipalGateValve.JPG).

Figure 6.7 Internals of a large butterfly valve (*Source*: http://upload.wikimedia.org/wikipedia/commons/d/d5/Yagisawa_power_station_inlet_valve.jpg).

Table 6.1 Number and distribution of fire hydrants

Fire flow requirement (gpm)	Minimum number of hydrants	Average spacing between hydrants[a,b,c] (ft)	Maximum distance from any point on street or road frontage to a hydrant[d] (ft)
1,750 or less	1	500	250
2,000–2,250	2	450	225
2,500	3	450	225
3,000	3	400	225
3,500–4,000	4	350	210
4,500–5,000	5	300	180
5,500	6	300	180
6,000	6	250	150
6,500–7,000	7	250	150
7,500 or more	8 or more[e]	200	120

Conversion factors: 1 ft = 0.3048 m; 1 gpm = 3.785 L/min.

[a] Reduce by 100 ft (30 m) for dead-end streets or roads.

[b] Where streets are provided with median dividers, which can be crossed by firefighters pulling hose lines, or where arterial streets are provided with four or more traffic lanes and have a traffic count of more than 30,000 vehicles per day, hydrant spacing shall average 500 ft (150 m) on each side of the street and be arranged on an alternating basis up to a fire flow requirement of 7,000 gpm (26,500 L/min) and 400 ft (122 m) for higher fire flow requirements.

[c] Where new water mains are extended along streets where hydrants are not needed for protection of structures or similar fire problems, fire hydrants shall be provided at spacing not to exceed 1,000 ft (3,800 m) to provide for transportation hazards.

[d] Reduce by 50 ft (15 m) for dead-end streets or roads.

[e] One hydrant for each 1,000 gpm (3,800 L/min) or fraction thereof.

distance requirement is 600 ft (183 m). A 3 ft (914 mm) clear space is required around the hydrants.

The minimum number of fire hydrants and their average spacing has to be within the requirements listed in Table 6.1. The placement of hydrants should be chosen in such a way that the maximum distance of all points on streets and access roads adjacent to a building is not more than the distance specified by the IFC as shown in Table 6.1.

6.3 SYSTEM CAPACITY

The capacity of distribution systems is dictated by domestic, industrial, and other normal water uses and by the *standby* or *ready-to-serve* requirements for firefighting. Pipes should be able to carry the maximum *coincident* draft at velocities that do not produce high pressure drops and surges. Velocities of 2–5 ft/s (0.60–1.50 m/s) and minimum pipe diameters of 6 in. (150 mm) are common in North American municipalities. Capacity to serve is not merely a function of available rate of draft; it is also a function of available pressure. The water must rise to the upper stories of buildings of normal height and must flow from hydrants, directly or through pumpers, to deliver needed fire streams through fire hoses long enough to reach the fire. If there were no fire hazard, the hydraulic capacity of distribution systems would have to equal the maximum demand for domestic, industrial, and other general uses.

The general firefighting requirements according to the IFC are summarized in Section 4.4.2 and Table 4.13. To these requirements for firefighting must be added a coincident demand of 40–50 gpcd (150–190 Lpcd) in excess of the average consumption rate for the area under consideration. In small communities or limited parts of large-distribution systems, pipe sizes are controlled by fire demand plus coincident draft. In the case of main feeder lines and other central works in large communities or large sections of metropolitan systems, peak hourly demands may determine the design.

6.4 SYSTEM PRESSURE

For normal drafts, water pressure at the street line must be at least 20 psig (140 kPa) to let water rise three stories and overcome the frictional resistance of the house-distribution system, but 40 psig (280 kPa) is more desirable. Business blocks are supplied more satisfactorily at pressures of 60–75 psig (420–520 kPa). To supply their upper stories, tall buildings must boost water to tanks on their roofs or in their towers and, often, also to intermediate floors.

Fire demand is commonly gauged by the *standard fire stream*: 250 gpm (946 L/min) issuing from a $1^{1}/_{8}$ in. (28.6 mm) nozzle at a pressure of 45 psig (312.3 kPa) at the base of the tip. When this amount of water flows through $2^{1}/_{2}$ in. (63.5 mm) rubber-lined hose, the frictional resistance is about 15 psi per 100 ft of hose (3.42 kPa/m). Adding the hydrant resistance and required nozzle pressure of 45 psig (312.3 kPa) then gives the pressure needs at the hydrant, as shown in Table 6.2. A standard fire stream is effective to a height of 70 ft (21.34 m) and has a horizontal carry of 63 ft (19.20 m).

Because hydrants are normally planned to control areas within a radius of 200 ft (61 m), Table 6.2 shows that direct attachment of fire hose to hydrants (hydrant streams) calls for a residual pressure at the hydrant of about 75 psig (520.5 kPa).

To maintain this pressure during times of fire, system pressures must approach 100 psig (694 kPa). This has its disadvantages, among them danger of breaks and leakage or waste of water approximately in proportion to the square

Table 6.2 Hydrant pressures for different lengths of fire hose

Length of hose ft (m)	Required pressure psig (kPa)
100 (30)	63 (437)
200 (60)	77 (534)
300 (90)	92 (638)
400 (120)	106 (736)
500 (150)	121 (840)
600 (180)	135 (937)

Conversion factors: 1 ft = 0.3048 m; 1 psig = gauge pressure 6.94 kPa.

root of the pressure. Minimum hydrant pressures of 50 psig (347 kPa) cannot maintain standard fire streams after passing through as little as 50 ft (15 m) of hose.

Motor pumpers commonly deliver up to 1,500 gpm (5,677 L/min) at adequate pressures. Capacities of 20,000 gpm (75,700 L/min) are in sight, with single streams discharging as much as 1,000 gpm (3,785 L/min) from 2 in. (50 mm) nozzles. To furnish domestic and industrial draft and keep pollution from entering water mains by seepage or failure under a vacuum, fire engines should not lower pressures in the mains to less than 20 psig (140 kPa). For large *hydrant* outlets, the safe limit is sometimes set at 10 psig (70 kPa). In a real way, modern firefighting equipment has eliminated the necessity for pressures much in excess of 60 psig (420 kPa), except in small towns that cannot afford a full-time, well-equipped fire department.

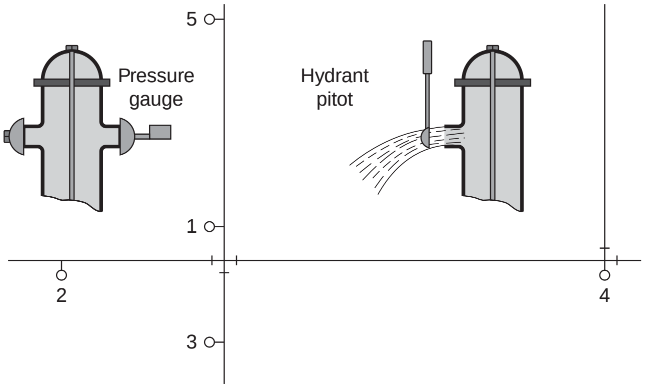


Figure 6.8 Location of pipes and hydrants in flow test and use of hydrant pitot and pressure gauge (see Table 6.3 and Fig. 6.9).

6.5 FIELD PERFORMANCE OF EXISTING SYSTEMS

The hydraulic performance of existing distribution systems is determined most directly and expeditiously by pressure surveys and hydrant-flow tests. Such tests should cover all typical portions of the community: the high-value district, residential neighborhoods and industrial areas of different kinds, the outskirts, and high-service zones. If need be, tests can be extended into every block. The results will establish available pressures and flows and existing deficiencies. These can then be made the basis of hydraulic calculations for extensions, reinforcements, and new gridiron layouts. Follow-up tests can show how successful the desired changes have been.

Pressure surveys yield the most rudimentary information about networks; if they are conducted both at night (minimum flow) and during the day (normal demand), they will indicate the hydraulic efficiency of the system in meeting common requirements. However, they will not establish the probable behavior of the system under stress, for example, during a serious conflagration.

Hydrant-flow tests commonly include (a) observation of the pressure at a centrally situated hydrant during the conduct of the test and (b) measurement of the combined flow from a group of neighboring hydrants. Velocity heads in the jets issuing from the hydrants are usually measured by hydrant *pitot tubes*. If the tests are to be significant, (a) the hydrants tested should form a group such as might be called into play in fighting a serious fire in the district under study, (b) water should be drawn at a rate that will drop the pressure enough to keep it from being measurably affected by normal fluctuations in draft within the system, and (c) the time of test should coincide with drafts (domestic, industrial, and the like) in the remainder of the system, reasonably close to *coincident* values.

The requirements of the IFC are valuable aids in planning hydrant-flow tests. A layout of pipes and hydrants in a typical flow test is shown in Fig. 6.8, and observed values are summarized in Table 6.3. This table is more or less self-explanatory. The initial and residual pressure was read from a Bourdon gauge at hydrant 1. Hydrants 2, 3, 4, and 5 were opened in quick succession, and their rates of discharge were measured simultaneously by means of hydrant pitots. A test such as this does not consume more than 5 min, if it is conducted by a well-trained crew.

Necessary hydrant-flow calculations for the flow test may be worked out using Eqs. (6.1) and (6.2) and are recorded as shown in Table 6.3.

Table 6.3 Record of a typical hydrant-flow test

Conditions of test	Observed pressure at hydrant 1 (psig)	Discharge velocity head (psig)	Calculated flow (Q) (gpm)	Remarks
All hydrants closed	74	…	…	All hydrant outlets are $2\frac{1}{2}$ in. in diameter
Hydrant 2 opened, 1 outlet	—	13.2	610	Total Q = 2,980 gpm; calculated engine streams = 4,200 gpm
Hydrant 3 opened, 2 outlets	…	9.6	2 × 520	
Hydrant 4 opened, 1 outlet	…	16.8	690	
Hydrant 5 opened, 1 outlet	46	14.5	640	
All hydrants closed	74	…	…	

Conversion factors: 1 psig = gauge pressure 6.94 kPa; 1 gpm = 3.785 L/min; 1 in. = 25.4 mm.

For outlets of diameter d in. the discharge Q in gpm is

$$Q = 30cd^2\sqrt{p} \qquad \text{(US customary units)} \qquad \textbf{(6.1)}$$

where p is the pitot reading in psig and c is the coefficient of hydrant discharge.

Equation (6.2) is the hydrant discharge equation using the SI units:

$$Q = 0.0668cd^2\sqrt{p} \qquad \text{(SI units)} \qquad \textbf{(6.2)}$$

where p is the pitot reading in kPa, c is the hydrant discharge coefficient, d is the outlet diameter in mm, and Q is the hydrant discharge in L/min.

Pressure–discharge relations established in this test are illustrated in Fig. 6.9. If the true static pressure is known, a more exact calculation is possible, although the additional labor involved is seldom justified. In accordance with the common hydraulic analysis of Borda's mouthpiece, a pressure gauge inserted in a hydrant in juxtaposition to the hydrant outlet to be opened will also record the discharge pressure otherwise measured by hydrant pitots.

Hydrant tests are sometimes made to ascertain the capacity of individual hydrants and advertise it to firefighters (particularly to engine companies summoned from neighboring towns) by painting the bonnet a suitable color. The weakness of this practice is its restriction of flow measurements to single hydrants. In firefighting, groups of hydrants are normally brought into action. Tests of individual hydrants may be quite misleading.

6.6 OFFICE STUDIES OF PIPE NETWORKS

No matter how energetically distribution systems are field-tested, needed extensions and reinforcements of old networks and the design of new ones can be adequately identified only by office studies. Necessary analysis presupposes familiarity with processes of hydraulic computation, including high-speed computers. Even without computers, however, the best processes can be so systematized as to make their application a matter of simple arithmetic and pipe-flow tables, diagrams, or slide rules. Useful methods of analysis are

1. Sectioning
2. Relaxation
3. Pipe equivalence
4. Computer programming and electrical analogy

6.6.1 Sectioning

Sectioning is an approximate and, in a sense, *exploratory method*, simple in concept and application and widely useful provided its limitations are clearly understood. Similar in concept is the circle method, which is usually confined to

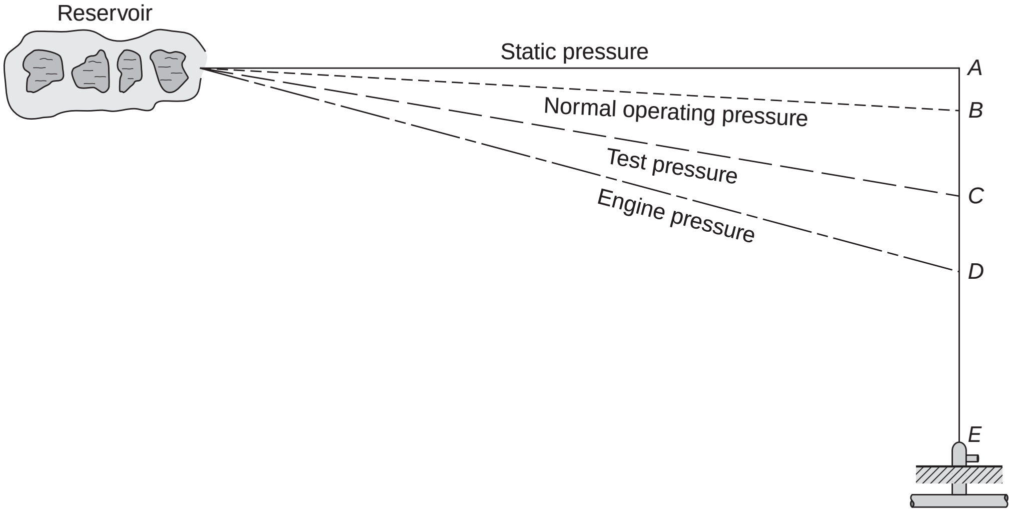


Figure 6.9 Pressure and discharge relations established by hydrant-flow test (see Fig. 6.8 and Table 6.3).

A: Static water table

B: No hydrant discharge. Pressure = 74 psig (514 kPa gauge); pressure drop P_0 due to coincident draft Q

C: Hydrant discharge. Pressure = 46 psig (319 kPa gauge); pressure drop $P_1 = (74 - 46) = 28$ psi (194 kPa) accompanies discharge of $Q_1 = 2{,}980$ gpm (908 L/min)

D: Engine streams. Pressure 20 psig (140 kPa gauge); pressure drop $P_2 = (74 - 20) = 54$ psi (375 kPa) accompanies discharge $Q_2 = 4{,}200$ gpm (1,280 L/min)

E: Hydrant 1, recording residual pressure of hydrant groups shown in Fig. 6.5

EXAMPLE 6.1 HYDRANT DISCHARGE

Determine the discharge rate of a hydrant using a smooth well rounded 2.5 in. (63.5 mm) outlet at pressures of 13.2, 9.6, 16.8, and 14.5 psig (91.61, 66.62, 116.59, and 100.63 kPa).

Solution 1 (US Customary System):

For smooth well rounded 2.5 in. outlets, $c = 0.9$

Refer to Table 6.3:

$$Q = 30cd^2\sqrt{p}$$
$$Q = 30 \times 0.9 \times 2.5^2\sqrt{p}$$
$$Q = 169\sqrt{p}.$$

For $p = 13.2$ psig $Q = 169\sqrt{13.2} = 610$ gpm.
For $p = 9.6$ psig $Q = 169\sqrt{9.6} = 520$ gpm.
For $p = 16.8$ psig $Q = 169\sqrt{16.8} = 690$ gpm.
For $p = 14.5$ psig $Q = 169\sqrt{14.5} = 640$ gpm.

Solution 2 (SI System):

For smooth well rounded 63.5 mm outlets, $c = 0.9$

Refer to Table 6.3:

$$Q = 0.0668cd^2\sqrt{p}$$
$$Q = 0.0668 \times 0.9 \times 63.5^2\sqrt{p}$$
$$Q = 242.42\sqrt{p}.$$

For $\boldsymbol{p} = \mathbf{91.61}$ **kPa** $\boldsymbol{Q} = \mathbf{2{,}320}$ **L/min**.
For $\boldsymbol{p} = \mathbf{66.62}$ **kPa** $\boldsymbol{Q} = \mathbf{1{,}979}$ **L/min**.
For $\boldsymbol{p} = \mathbf{116.59}$ **kPa** $\boldsymbol{Q} = \mathbf{2{,}618}$ **L/min**.
For $\boldsymbol{p} = \mathbf{100.63}$ **kPa** $\boldsymbol{Q} = \mathbf{2{,}432}$ **L/min**.

cutting pipes tributary to a central fire hydrant or group of hydrants at the center of a circle.

Use of the section method is illustrated in Fig. 6.10 and Example 6.2. These are the required steps:

1. Cut the network by a *series of lines*, not necessarily straight or regularly spaced but chosen with due regard to the assumed sources and loads and the estimated location of the piezometric contours. A first series of lines may well cut the distribution piping substantially *at right angles* to the general direction of flow, that is, *perpendicular* to a line drawn from the supply conduit to the high-value district (Fig. 6.10). Further series may be oriented in some other critical direction, for example, horizontally and vertically in Fig. 6.10. For more than one supply conduit, the sections may be curved to intercept the flow from each conduit.
2. Estimate how much water must be supplied to areas *beyond* each section. Base estimates on a knowledge of the population density and the general characteristics of the zone: residential, commercial, and industrial. The water requirements comprise (a) the normal, coincident draft, here called the

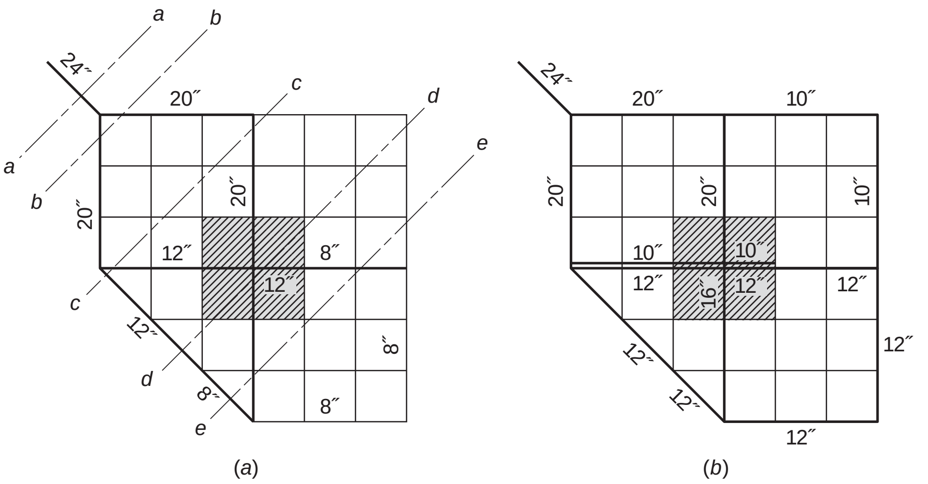


Figure 6.10 Plan of network analyzed by method of sections (Example 6.2): (a) existing system; (b) recommended system (unless otherwise indicated, pipe diameters are 6 in. (150 mm). The high-value district is cross-hatched). Conversion factor: 1″ = 1 in. = 25.4 mm.

domestic draft, and (b) the fire demand (Table 4.13). Domestic use decreases progressively from section to section, as population or industry is left behind; fire demand remains the same until the high-value district has been passed, after which it drops to a figure applicable to the type of outskirt area.

3. Estimate the distribution system *capacity* at each section across the piping. To do this (a) tabulate the number of pipes of each size cut; count only pipes that deliver water in the general direction of flow; and (b) determine the average available hydraulic gradient or frictional resistance, which depends on the pressure to be maintained in the system and the allowable pipe velocity. Ordinarily, hydraulic gradients lie between 1‰ (per thousand) and 3‰, and velocities range from 2 to 5 ft/s (0.60–1.5 m/s).
4. For the available, or desirable, *hydraulic gradient*, determine the capacities of existing pipes and sum them for total capacity.
5. Calculate the *deficiency* or difference between required and existing capacity.
6. For the available, or desirable, hydraulic gradient, *select the sizes* and routes of pipes that will offset the deficiency. General familiarity with the community and studies of the network plan will aid judgment. Some existing small pipes may have to be removed to make way for larger mains.
7. Determine the size of the *equivalent pipe* for the reinforced system and calculate the velocity of flow. Excessive velocities may make for dangerous water hammer. They should be avoided, if necessary, by lowering the hydraulic gradients actually called into play.
8. Check important pressure requirements against the plan of the reinforced network.

The method of sections is particularly useful (a) in preliminary studies of large and complicated distribution systems, (b) as a check on other methods of analysis, and (c) as a basis for further investigations and more exact calculations.

EXAMPLE 6.2 ANALYSIS OF A WATER NETWORK USING THE SECTIONS METHOD

Analyze the network of Fig. 6.10 by sectioning. The hydraulic gradient available within the network proper is estimated to lie close to 2‰. The value of *C* in the Hazen–Williams formula is assumed to be 100, and the domestic (coincident) draft, in this case, only 150 gpcd (568 Lpcd). The fire demand is taken from Table 4.13. Assume the population for each section as follows: section a-a, 16,000; section b-b, 16,000; section c-c, 14,700; section d-d, 8,000, and section e-e, 3,000. Also assume that the type of building construction in the high-value district is combustible and unprotected and that the maximum total surface area per building is 20,000 ft^2 (1,858 m^2). The area downstream of the high-value district is residential with one- and two-family dwellings having a maximum area of 6,000 ft^2 (557.4 m^2).

Solution:

Calculations are shown only for the first three sections.

1. Section a-a population = 16,000:
   (a) Demand: domestic = $16{,}000 \times 150/10^6$ = 2.4 MGD (9.1 MLD); fire (from Table 4.13) = 3,750 gpm = 5.4 MGD (20.4 MLD); total = 7.8 MGD (29.5 MLD).
   (b) Existing pipes: one 24 in. (600 mm); capacity = 6.0 MGD (22.7 MLD).
   (c) Deficiency: 7.8 − 6.0 = 1.8 MGD or (29.5 − 22.7 = 6.8 MLD).
   (d) If no pipes are added, the 24 in. (600 mm) pipe must carry 7.8 MGD (29.5 MLD).
   This it will do with a loss of head of 3.2‰ at a velocity of 3.8 ft/s (1.16 m/s).
2. Section b-b population and flow as in section a-a:
   (a) Total demand = 7.8 MGD (29.5 MLD).
   (b) Existing pipes: two 20 in. (500 mm) at 3.7 MGD = 7.4 MGD (28.0 MLD).
   (c) Deficiency = 7.8 − 7.4 = 0.4 MGD or(29.5 − 28.0 = 1.5 MLD).
   (d) If no pipes are added, existing pipes will carry 7.8 MGD (29.5 MLD) with a loss of head of 2.2‰, at a velocity of 2.8 ft/s (0.85 m/s).
3. Section c-c population = 14,700:
   (a) Demands: domestic = $14{,}700 \times 150/10^6$ = 2.2 MGD (8.3 MLD); fire = 5.4 MGD (20.4 MLD); total = 7.6 MGD (28.7 MLD).
   (b) Existing pipes: one 20 in. (500 mm) at 3.7 MGD; two 12 in. (300 mm) at 1.0 MGD = 2.0 MGD; five 6 in. (150 mm) at 0.16 MGD = 0.8 MGD; total = 6.5 MGD (24.6 MLD).
   (c) Deficiency = 7.6 − 6.5 = 1.1 MGD or (28.7 − 24.6 = 4.1 MLD).

(d) **Pipes added: two 10 in.** (250 mm) at 0.6 MGD = 1.2 MGD (4.5 MLD).
Pipes removed: one 6 in. at 0.2 MGD (one 150 mm at 0.76 MLD).
Net added capacity: 1.2 − 0.2 = 1.0 MGD (3.8 MLD).
Reinforced capacity = 6.5 + 1.0 = 7.5 MGD or (24.6 + 3.8 = 28.4 MLD).

(e) The reinforced system equivalent pipe at 7.5 MGD (28.4 MLD) and a hydraulic gradient of 2‰ is 26.0 in. (650 mm) This will carry 7.6 MGD (28.8 MLD) with a loss of head of 2.1‰.

6.6.2 Relaxation (Hardy Cross)

A method of *relaxation*, or *controlled trial and error*, was introduced by Hardy Cross, whose procedures are followed here with only a few modifications. In applying a method of this kind, calculations become speedier if pipe–flow relationships are expressed by an exponential formula with unvarying capacity coefficient, and notation becomes simpler if the exponential formula is written:

$$H = kQ^n \tag{6.1}$$

where, for a given pipe, k is a numerical constant depending on C, d, and L, and Q is the flow, n being a constant exponent for all pipes. In the Hazen–Williams equation, for example,

$$Q = 405\ Cd^{2.63}s^{0.54} \quad \text{(US customary units)}$$

where Q = rate of discharge, gpd; d = diameter of circular conduits, in.; C = Hazen–Williams coefficient, dimensionless; S = H/L = hydraulic gradient, dimensionless; H = loss of head, ft; L = conduit length, ft.

$$Q = 0.278\ Cd^{2.63}S^{0.54} \quad \text{(SI units)}$$

where Q = rate of discharge, m^3/s; d = diameter of circular conduits, m; C = Hazen–Williams coefficient, dimensionless; S = H/L = hydraulic gradient, dimensionless; H = loss of head, m; L = conduit length, m. For the Hazen–Williams using either the US customary units or the SI units, the following relationship hold true:

$$\begin{aligned} s &= k'Q^{1/0.54} = k'Q^{1.85} \\ H &= sL \\ H &= kQ^{1.85} \end{aligned} \tag{6.2}$$

Two procedures may be involved, depending on whether (a) the quantities of water entering and leaving the network or (b) the *piezometric levels*, *pressures*, or *water table* elevations at inlets and outlets are known.

In balancing heads by correcting assumed flows, necessary formulations are made algebraically consistent by arbitrarily assigning *positive signs* to *clockwise flows* and associated head losses, and *negative signs* to *counterclockwise flows* and associated head losses. For the simple network shown in Fig. 6.11a, inflow Q_i and outflow Q_o are equal and known, inflow being split between two branches in such a manner that the sum of the balanced head losses H_1 (clockwise) and $-H_2$ (counterclockwise) or $\Sigma H = H_1 - H_2 = 0$. If the assumed split flows Q_1 and $-Q_2$ are each in error by the same small amount q, then

$$\Sigma H = \Sigma k\,(Q + q)^n = 0$$

Expanding this binomial and neglecting all but its first two terms, because higher powers of q are presumably very small, we get

$$\Sigma H = \Sigma k(Q + q)^n = \Sigma kQ^n + \Sigma nkqQ^{n-1} = 0, \text{whence}$$

$$q = -\frac{\Sigma kQ^n}{n\Sigma kQ^{n-1}} = -\frac{\Sigma H}{n\Sigma H/Q} \tag{6.3}$$

If a takeoff is added to the system as in Fig. 6.11b, both head losses and flows are affected.

In balancing flows by correcting assumed heads, necessary formulations become algebraically consistent when positive signs are arbitrarily assigned to *flows toward junctions* other than inlet and outlet junctions (for which water table elevations are known) and *negative signs to flows away from* these *intermediate junctions*, the sum of the balanced flows at

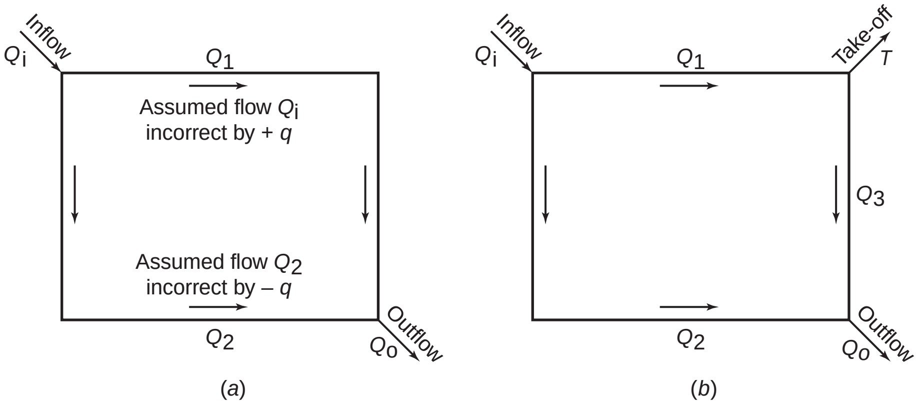


Figure 6.11 Simple network illustrating (a) the derivation of the Hardy Cross method and (b) the effect of changing flows.

the junctions being zero. If the assumed water table elevation at a junction, such as the takeoff junction in Fig. 6.8b, is in error by a height h, different small errors q are created in the individual flows Q leading to and leaving from the junction. For any one pipe, therefore, $H + h = k(Q + q^n) = kQ^n + h$, where H is the loss of head associated with the flow Q. Moreover, as before,

$$h = nkqQ^{n-1} = nq(H/Q) \text{ and}$$

$$q = (h/n)(Q/H)$$

Because $\Sigma(Q + q) = 0$ at each junction,

$$\Sigma Q = -\Sigma q \text{ and}$$

$$\Sigma q = (h/n)\,\Sigma\,(Q/H)\,, \text{or}$$

$$\Sigma Q = -\,(h/n)\,\Sigma\,(Q/H)\,, \text{therefore,}$$

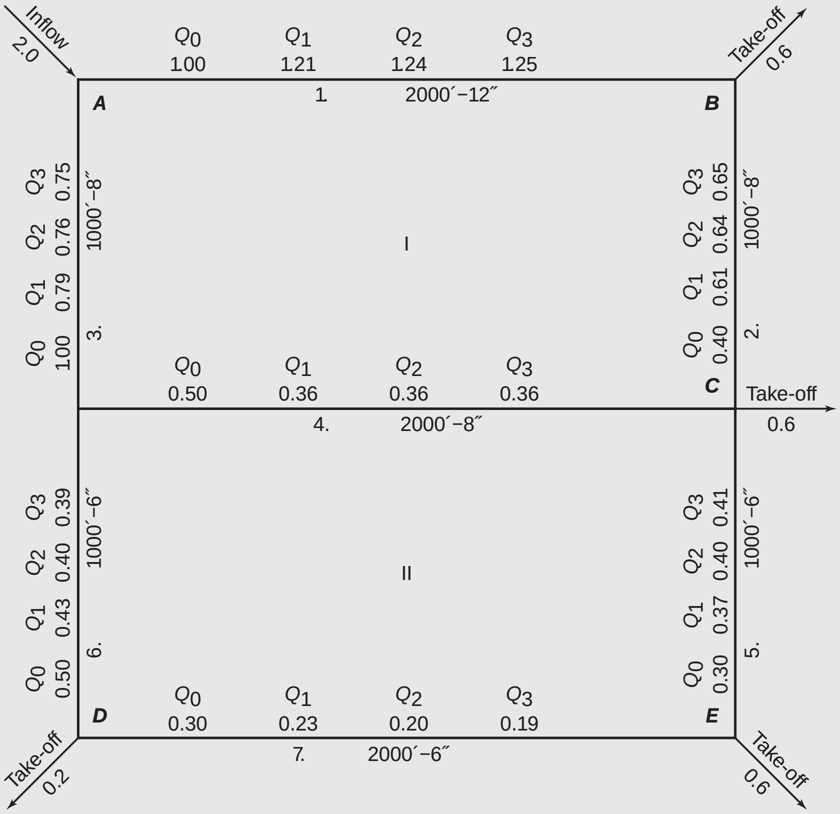
$$h = -\frac{n\Sigma Q}{\Sigma\,(Q/H)} \tag{6.4}$$

The corrections q and h are only approximate. After they have been applied once to the assumed flows, the network is more nearly in balance than it was at the beginning, but the process of correction must be repeated until the balancing operations are perfected. The work involved is straightforward, but it is greatly facilitated by a satisfactory scheme of bookkeeping such as that outlined for the method of balancing heads in Example 6.3 for the network sketched in Fig. 6.12.

Although the network in Example 6.3 is simple, it cannot be solved conveniently by algebraic methods, because it contains two interfering hydraulic constituents: (a) a crossover (pipe 4) involved in more than one circuit and (b) a series of takeoffs representing water used along the pipelines, fire flows through hydrants, or supplies through to neighboring circuits.

EXAMPLE 6.3 ANALYSIS OF A WATER NETWORK USING THE RELAXATION METHOD OF BALANCING HEADS

Balance the network shown in Figure 6.12 using the relaxation method of balancing heads



All flows are in MGD

Figure 6.12 Plan of network analyzed by the method of balancing heads (Example 6.3). Conversion factors: 1 MGD = 3.785 MLD; 1′ = 1 ft = 0.3048 m; 1″ = 1 in. = 25.4 mm.

Solution:

The schedule of calculations (Table 6.4) includes the following:

Table 6.4 Analysis of the network of Fig. 6.12 (Example 6.3) using the balancing heads method

Network				Assumed conditions					First correction				
Circuit no. (1)	Pipe no. (2)	Length (ft) (3)	Diameter (in.) (4)	Q_0 (MGD) (5)	s_0 (‰) (6)	H_0 (ft) (7)	H_0/Q_0 (8)	q_0 (MGD) (9)	Q_1 (MGD) (10)	s_1 (‰) (11)	H_1 (ft) (12)	H_1/Q_1 (13)	q_1 (MGD) (14)
I	1	2,000	12	+1.0	2.1	+4.2	4.2	+0.21	+1.21	3.0	+6.0	5.0	+0.03
	2	1,000	8	+0.4	2.8	+2.8	7.0	+0.21	+0.61	6.1	+6.1	10.0	+0.03
	3	1,000	8	−1.0	15.1	−15.1	15.1	+0.21	−0.79	9.8	−9.8	12.4	+0.03
	4[a]	2,000	8	−0.5	4.2	−8.4	16.8	+0.21	−0.36	2.3	−4.6	12.8	+0.03
								−0.07[b]					−0.03[b]
						−16.5 ÷ (43.1 × 1.85) = −0.21						−2.3 ÷ (40.2 × 1.85) = −0.03	
II	4[c]	2,000	8	+0.5	4.2	+8.4	16.8	+0.07	+0.36	2.3	+4.6	12.8	+0.03
								−0.21[b]					−0.03[b]
	5	1,000	6	+0.3	6.6	+6.6	22.0	+0.07	+0.37	9.8	+9.8	26.5	+0.03
	6	1,000	6	−0.5	16.9	−16.9	33.8	+0.07	−0.43	12.9	−12.9	30.0	+0.03
	7	2,000	6	−0.3	6.6	−13.2	44.0	+0.07	−0.23	4.1	−8.2	35.6	+0.03
						−15.1 ÷ (116.6 × 1.85) = −0.07						−6.7 ÷ (104.9 × 1.85) = −0.03	

Network				Second correction					Result			
Circuit no. (1)	Pipe no. (2)	Length (ft) (3)	Diameter (in.) (4)	Q_2 (MGD) (15)	s_2 (‰) (16)	H_2 (ft) (17)	H_2/Q_2 (18)	q_2 (MGD) (19)	Q_3 (MGD) (20)	s_3 (‰) (21)	H_3 (ft) (22)	Loss of head *A–E* (23)
I	1	2,000	12	+1.24	3.1	+6.2	5.0	+0.01	+1.25	3.2	+6.4	1. Via pipes 1, 2, 5, 25.0 ft
	2	1,000	8	+0.64	6.6	+6.6	10.3	+0.01	+0.65	6.8	+6.8	2. Via pipes 3, 4, 5, 25.3 ft
	3	1,000	8	−0.76	9.1	−9.1	12.0	−0.01	−0.75	8.9	−8.9	3. Via pipes 3, 6, 7, 25.5 ft
	4[a]	2,000	8	−0.36	2.3	−4.6	12.8	−0.01	−0.36	2.3	−4.6	
								−0.01[b]				
						−0.9 ÷ (40.1 × 1.85) = −0.01					−0.3	
II	4[c]	2,000	8	+0.36	2.3	+4.6	12.8	+0.01	+0.36	2.6	+4.6	
								−0.01[b]				
	5	1,000	6	+0.40	11.3	+11.3	28.2	+0.01	+0.41	11.8	+11.8	
	6	1,000	6	−0.40	11.3	−11.3	28.2	−0.01	−0.39	10.8	−10.8	
	7	2,000	6	−0.20	3.1	−6.2	31.0	−0.01	−0.19	2.9	−5.8	
						−1.6 ÷ (100.2 × 1.85) = −0.01					−0.2	

Q = flow in MGD; s = slope of hydraulic gradient or friction loss in ft per 1000 (‰) by the Hazen–Williams formula for $C = 100$, H = head lost in pipe (ft).

q = flow correction in MGD; $q = \frac{\sum H}{1.85 \sum (H/Q)}$, $Q_1 = Q_0 + q_0$; $Q_2 = Q_1 + q_1$; $Q_3 = Q_2 + q_2$.

Conversion factors: 1 ft = 0.3048 m; 1 in. = 25.4 mm; 1 MGD = 3.785 MLD.

[a] Pipe serves more than one circuit; first consideration of this pipe.

[b] Corrections in this column are those calculated for the same pipe in the companion circuit; they are of opposite sign.

[c] Second consideration of this pipe.

Columns 1–4 identify the position of the pipes in the network and record their length and diameter. There are two circuits and seven pipes. Pipe 4 is shared by both circuits; "a" indicates this in connection with circuit I; "c" does so with circuit II. This dual pipe function must not be overlooked.

Columns 5–9 deal with the assumed flows and the derived flow correction. For purposes of identification the hydraulic elements Q, s, H, and q are given a subscript zero.

Column 5 lists the assumed flows Q_0 in MGD or MLD. They are preceded by positive signs if they are clockwise and by negative signs if they are counterclockwise. The distribution of flows has been purposely misjudged in order to highlight the balancing operation. At each junction the total flow remaining in the system must be accounted for.

Column 6 gives the hydraulic gradients in ft per 1,000 ft (‰) or in m per 1,000 m when the pipe is carrying the quantities Q_0 shown in Col. 5. The values of s_0 can be read directly from tables or diagrams of the Hazen–Williams formula.

Column 7 is obtained by multiplying the hydraulic gradients (s_0) by the length of the pipe in 1,000 ft, that is, Col. 7 = Col. 6 × (Col. 3/1,000). The head losses H_0 obtained are preceded by a positive sign if the flow is clockwise and by a negative sign if counterclockwise. The values in Col. 7 are totaled for each circuit, with due regard to signs, to obtain ΣH.

Column 8 is found by dividing Col. 7 by Col. 5. Division makes all signs of H_0/Q_0 positive. This column is totaled for each circuit to obtain $\Sigma(H_0/Q_0)$ in the flow correction formula.

Column 9 contains the calculated flow correction $q_0 = -\Sigma H_0/(1.85 \times \Sigma H_0/Q_0)$. For example, in circuit I, $\Sigma H_0 = -16.5, \Sigma(\mathrm{H}_0/Q_0) = 43.1$; and $(-16.5)/(1.85 \times 43.1) = -0.21$; or $q_0 = +0.21$. Because pipe 4 operates in both circuits, it draws a correction from each circuit. However, the second correction is of opposite sign. As a part of circuit I, for example, pipe 4 receives a correction of $q = -0.07$ from circuit II in addition to its basic correction of $q = +0.21$ from circuit I.

Columns 10–14 cover the once-corrected flows. Therefore, the hydraulic elements (Q, s, H, and q) are given the subscript 1. Column 10 is obtained by adding, with due regard to sign, Cols. 5 and 9; Cols. 11–14 are then found in the same manner as Cols. 6–9.

Columns 15–19 record the twice-corrected flows, and the hydraulic elements (Q, s, H, and q) carry the subscript 2. These columns are otherwise like Cols. 10–14.

Columns 20–23 present the final result, with Cols. 20–22 corresponding to Cols. 15–18 or 10–12. No further flow corrections are developed because the second flow corrections are of the order of 10,000 gpd (37,850 L/d) for a minimum flow of 200,000 gpd (757,000 L/d), or at most 5%. To test the balance obtained, the losses of head between points A and D in Fig. 6.12 via the three possible routes are given in Col. 23. The losses vary from 25.0 to 25.5 ft (7.62–7.77 m). The average loss is 25.3 ft (7.71 m) and the variation about 1%.

EXAMPLE 6.4 ANALYSIS OF A WATER NETWORK USING THE RELAXATION METHOD OF BALANCING FLOWS

Balance the network of Fig. 6.13 using the balancing flows method. Necessary calculations are given in Table 6.5.

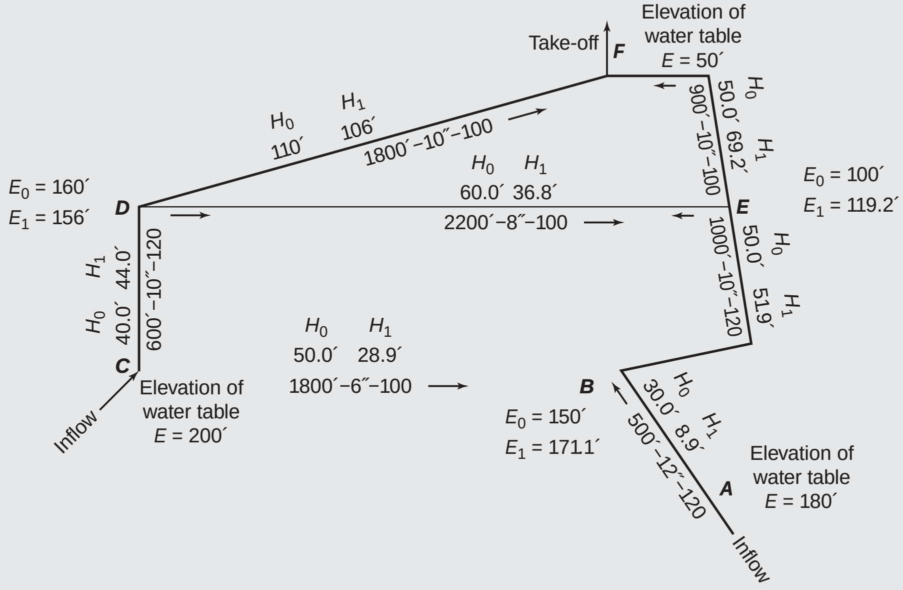


Figure 6.13 Plan of network analyzed by the method of balancing flows (Example 6.4). Conversion factors: 1′ = 1 ft = 0.3048 m; 1″ = 1 in. = 25.4 mm.

Table 6.5 Analysis of the network of Fig. 6.13 using the balancing flows method[a,b] (Example 6.4)

Junction letter (1)	Pipe (2)	Length (ft) (3)	Diameter (in.) (4)	C (5)	H_0 (ft) (6)	s_0 (%) (7)	Q_0 (MGD) (8)	Q_0/H_0 (9)	h_0 (ft) (10)		H_1 (ft) (11)
B	AB	500	12	120	+30	60.0	+7.33	0.244	−21.1		+8.9
	BE	1,000[c]	10	120	−50	50.0	−4.12	0.082	−21.1	+19.2	−51.9
	CB	1,800	6	100	+50	27.8	+0.66	0.013	−21.1		+28.9
						1.85	× (+3.87) ÷	0.339 =	+21.1		
D	CD	600	10	120	+40	66.7	+4.8	0.120	+4.01		+44.0
	DE	2,200[d]	8	100	−60	27.3	−1.37	0.023	+4.01	+19.2	−36.8
	DF	1,800	10	100	−110	61.1	−3.82	0.037	+4.01		−106.0
						1.85	× (−0.39) ÷	0.180 =	−4.01		
E	BE	1,000[e]	10	120	+50	50.0	+4.12	0.082	−19.2	+21.1	+51.9
	DE	2,200[f]	8	100	+60	27.3	+1.37	0.023	−19.2	−4.01	+36.8
	EF	900	10	100	−50	55.6	−3.64	0.073	−19.2		−69.2
						1.85	× (+1.85) ÷	0.178 =	+19.2		

Conversion factors: 1 ft = 0.3048 m; 1 in. = 25.4 mm; 1 MGD = 3.785 MLD.

[a] For Example 6.4.

[b] Only the first head correction is calculated for purposes of illustration.

[c] First consideration of pipe BE.

[d] First consideration of pipe DE.

[e] Second consideration of pipe BE.

[f] Second consideration of pipe DE.

Solution:

The schedule of calculations includes the following:

Columns 1–5 identify the pipes at the three free junctions.

Columns 6 and 7 give the assumed head loss and the derived hydraulic gradient that determines the rate of flow shown in Col. 8 and the flow–head ratio recorded in Col. 9 = (Col. 8 divided by Col. 6).

Column 10 contains the head correction h_0 as the negative value of 1.85 times the sum of Col. 8 divided by the sum of Col. 9, for each junction in accordance with Eq. (6.4):

$$h = \frac{n\Sigma Q}{\Sigma(Q/H)} \tag{6.4}$$

A subsidiary head correction is made for shared pipes as in Example 6.2.

Column 11 gives the corrected head $H_1 = H_0 + h_0$ and provides the basis for the second flow correction by determining s, Q, and Q_1/H_1 in that order.

6.6.3 Pipe Equivalence

In this method, a complex system of pipes is replaced by a single *hydraulically equivalent* line. The method cannot be applied directly to pipe systems containing crossovers or takeoffs. However, it is frequently possible, by judicious skeletonizing of the network, to obtain significant information on the quantity and pressure of water available at important points or to reduce the number of circuits to be considered. In paring the system down to a workable frame, the analyst can be guided by the fact that pipes contribute little to flow (a) when they are small (6 in. (150 mm) and under in most systems and 8 or 10 in. (200 or 250 mm) in large systems) and (b) when they are at right angles to the general direction of flow and there is no appreciable pressure differential between their junctions in the system.

Pipe equivalence makes use of the two hydraulic axioms:

1. Head losses through *pipes in series*, such as *AB* and *BD* in Fig. 6.14, are additive:

$$\begin{aligned} H_{AD} &= H_{AB} + H_{BD} \\ Q_{AB} &= Q_{BD} \\ H_{AD} &= H_{AC} + H_{CD} \\ Q_{AC} &= Q_{CD} \end{aligned}$$

2. Flows through *pipes in parallel*, such as *ABD* and *ACD* in Fig. 6.14, must be so distributed that the head losses are identical:

$$H_{ABD} = H_{ACD}$$

EXAMPLE 6.5 ANALYSIS OF A WATER NETWORK USING THE EQUIVALENT PIPE METHOD

Find an equivalent pipe for the network of Fig. 6.14. Express Q in MGD or MLD; s in ‰; H in ft or m; and assume a Hazen–Williams coefficient C of 100.

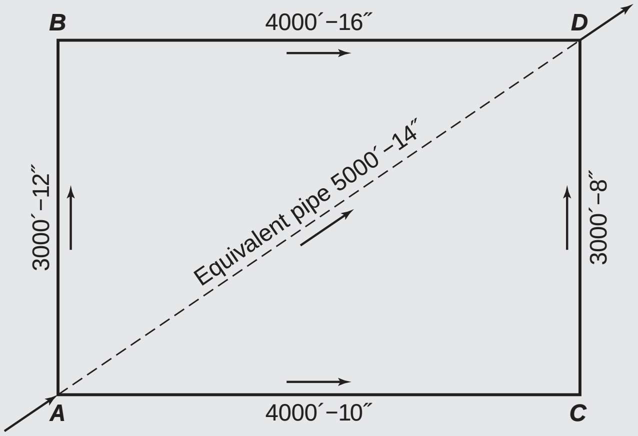


Figure 6.14 Plan of network analyzed by the method of equivalent pipes (Example 6.5).

Solution:

1. Line ABD. Assume $Q = 1$ MGD (3.785 MLD).
   (a) Pipe AB, 3,000 ft, 12 in.; $s = 2.1$; $H = 2.1 \times 3 = 6.3$ ft (1.92 m).
   (b) Pipe BD, 4,000 ft, 16 in.; $s = 0.52$; $H = 0.52 \times 4 = 2.1$ ft (0.64 m).
   (c) Total $H = 6.3 + 2.1 = 8.4$ ft (2.56 m).
   (d) Equivalent length of 12 in. (300 mm) pipe: $1{,}000 \times 8.4/2.1 =$ **4,000 ft (1,219 m)**.
2. Line ACD. Assume $Q = 0.5$ MGD (1.89 MLD).
   (a) Pipe AC, 4,000 ft, l0 in.; $s = 1.42$; $H = 1.42 \times 4 = 5.7$ ft (1.74 m).
   (b) Pipe CD, 3,000 ft, 8 in.; $s = 4.2$; $H = 4.2 \times 3 = 12.6$ ft (3.84 m).
   (c) Total $H = 5.7 + 12.6 = 18.3$ ft (5.58 m).
   (d) **Equivalent length of 8 in.** (200 mm) pipe: $1{,}000 \times 18.3/4.2 =$ **4,360 ft** (1,329 m).
3. Equivalent line AD. Assume $H = 8.4$ ft (2.56 m).
   (a) Line ABD, 4,000 ft, 12 in.; $s = 8.4/4.00 = 2.1$; $Q = 1.00$ MGD (3.785 MLD).
   (b) Line ACD, 4360 ft, 8 in.; $s = 8.4/4.36 = 1.92$; $Q = 0.33$ MGD (1.25 MLD).
   (c) Total $Q = 1 + 0.33 = 1.33$ MGD (5.03 MLD).
   (d) Equivalent length of 14 in. (350 mm) pipe: $Q = 1.33$, $s = 1.68$, $1{,}000 \times 8.4/1.68 = 5{,}000$ ft (1,524 m).
   (e) Result: **5,000 ft of 14 in. (1,524 m of 350 mm) pipe.**

Necessary calculations are as follows:

1. Because line ABD consists of two pipes in series, the losses of head created by a given flow of water are additive. Find, therefore, from the Hazen–Williams diagram (Chapter 5 and/or Appendix 14) the frictional resistance s for some reasonable flow (1 MGD or 3.78 MLD) (a) in pipe AB and (b) in pipe BD. Multiply these resistances by the length of pipe to obtain the loss of head H. Add the two losses to find the total loss $H = 8.4$ ft (2.56 m). Line ABD, therefore, must carry 1 MGD (3.78 MLD) with a total loss of head of 8.4 ft (2.56 m). Any pipe that will do this is an equivalent pipe. Because a 12 in. (300 mm) pipe has a resistance s of 2.1‰ when it carries 1 MGD (3.78 MLD) of water, a 12 in. (300 mm) pipe, to be an equivalent pipe, must be $1{,}000 \times 8.4/2.1 = 400$ ft (122 m) long.
2. Proceed in the same general way with line ACD to find a length of 4360 ft (1329 m) for the equivalent 8 in. (200 mm) pipe.

3. Because ABD and ACD together constitute two lines in parallel, the flows through them at a given loss of head are additive. If some convenient loss is assumed, such as the loss already calculated for one of the lines, the missing companion flow can be found from the Hazen–Williams diagram. Assuming a loss of 8.4 ft (2.56 m), which is associated with a flow through ABD of 1 MGD (3.78 MLD), it is only necessary to find from the diagram that the quantity of water that will flow through the equivalent pipe ACD, when the loss of head is 8.4 ft or 2.56 m (or $s = 8.4/4.36 = 1.92‰$), amounts to 0.33 MGD (1.25 MGD). Add this quantity to the flow through line ABD (1.0 MGD) or 3.78 MLD to obtain 1.33 MGD (5.03 MLD). Line AD, therefore, must carry 1.33 MGD (5.03 MLD) with a loss of head of 8.4 ft (2.56 m). If the equivalent pipe is assumed to be 14 in. (350 mm) in diameter, it will discharge 1.33 MGD (5.03 MLD) with a frictional resistance $s = 1.68‰$, and its length must be $1{,}000 \times 8.4/1.68 = 5{,}000$ ft (1,524 m). Thence, the network can be replaced by a single 14 in. (350 mm) pipe 5,000 ft (1,524 m) long.

No matter what the original assumptions for quantity, diameter, and loss of head, the calculated equivalent pipe will perform hydraulically in the same way as the network it replaces.

Different in principle is the operational replacement of every pipe in a given network by equivalent pipes with identical diameters and capacity coefficients, but variable length. The purpose, in this instance, is to simplify subsequent calculations. For the Hazen–Williams relationship using the US customary units Q (gpd), d (in.), H (ft), L (ft), and C and s (dimensionless), for example,

$$Q = 405Cd^{2.63}s^{0.54}$$

$$s^{0.54} = Q/405Cd^{2.63}s^{0.54} \tag{6.5}$$

$$s = Q^{1.85}/(405)^{1.85}C^{1.85}d^{4.87}$$

$$H/L = Q^{1.85}/(405)^{1.85}C^{1.85}d^{4.87}$$

$$L = (405)^{1.85}C^{1.85}d^{4.87}H/Q^{1.85}$$

$$L_e/L = (100/C)^{1.85}(d_e/d)^{4.87}$$

where L_e is the length of a pipe of diameter d_e; discharge coefficient $C = 100$; and L, d, and C are the corresponding properties of the existing pipe. The desired values for L can be found readily from a logarithmic plot of L_e/L against d_e/d at given values of C, as shown in Fig. 6.15.

The readers are referred to Chapters 5 for all Hazen–Willliams equations using the SI units.

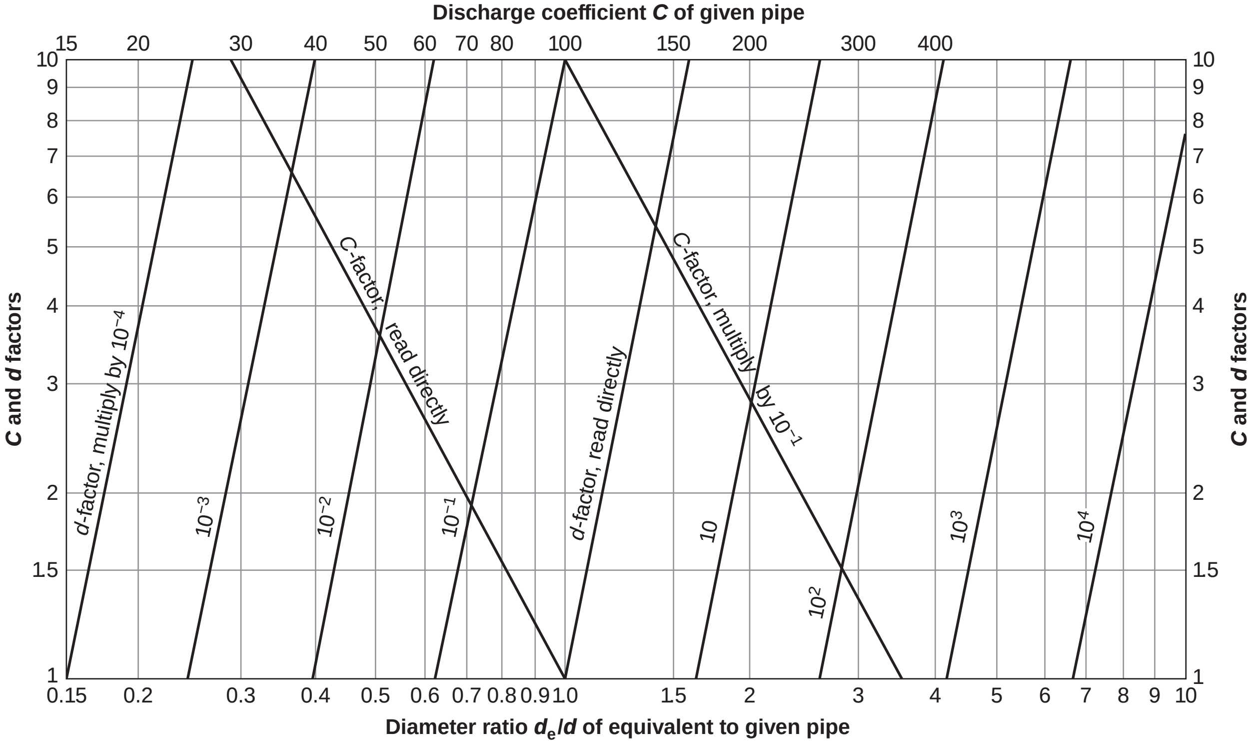


Figure 6.15 Length, diameter, and coefficient (L_e, d_e, and $C_e = 100$) of pipe hydraulically equivalent to an existing pipe of given L, d, and C.

EXAMPLE 6.6 DETERMINATION OF EQUIVALENT PIPE

Find the length of a 24 in. (600 mm) pipe with $C = 100$ that is equivalent to a 12 in. (300 mm) pipe with $C = 130$ and $L = 2{,}000$ ft (610 m).

Solution:

At $C = 130$ read the C-factor 6.2×10^{-1}.

At $d_e/d = 2.0$, read the d-factor 2.9×10.

Hence $L_e/L = (6.2 \times 10^{-1})(2.9 \times 10) = 18$.

$L_e = 2{,}000 \times 18 = 36{,}000$ ft (10,973 m).

6.6.4 Computer Programming

High-speed computers can be programmed to solve network problems in a number of different ways. Convergence formulas need not be introduced as such. Instead, the computer can be assigned the task of adjusting the water table or pressure at each junction not controlled by a service reservoir until the circuit laws discussed in connection with the method of relaxation are satisfied throughout the system. These *laws* can be summarized as follows:

1. At each junction $\Sigma Q_{\text{inflow}} = \Sigma Q_{\text{outflow}}$
2. In each circuit $\Sigma H = 0$
3. In each pipe $H = kQ^n$ or $Q = (H/k)^{1/n}$

To program the operation, number each pipe and junction and identify pipe ends by junction numbers. Then tabulate pipe resistances, junction pressures (including assumed values where pressures are unknown), and net inflows at each junction (zero at all but entrance and exit points of the system), and feed the tabulated information into the computer. The computer instructions are then as follows: calculate by circuit law 3 the total flow into the first junction for which the water table elevation is unknown; adjust the assumed value until the total inflow and outflow are balanced in accordance with *circuit law* 1; proceed in sequence to the remaining junctions; and readjust the first water table elevation. Repeat the cycle of operations until *all circuit laws* are satisfied.

Camp and Hazen (1934) built the first electric analyzer designed specifically for the hydraulic analysis of water distribution systems. Electric analyzers use *nonlinear resistors*, called *fluistors* in the *McIlroy analyzer*, to simulate pipe resistances. For each branch of the system, the pipe equation, $H = kQ^{1.85}$, for example, is replaced by an electrical equation, $V = K_e I^{1.85}$, where V is the voltage drop in the branch, I is the current, and K_e is the nonlinear-resistor coefficient suited to pipe coefficient k for the selected voltage drop (head loss) and amperage (water flow) scale ratios. If the current inputs and takeoffs are made proportional to the water flowing into and out of the system, the head losses will be proportional to the measured voltage drops. Some large, rapidly developing communities have found it economical to acquire electric analyzers suited to their own systems.

Of course at present, several handy commercial software programs are available for modeling and design of water systems (see Chapter 7). Software is available as a standalone interface for Windows or integrated into GIS or CAD systems, for example:

1. Haestad Methods Solutions (Bentley Systems Inc.): WaterGEMS and WaterCAD
2. MWH Soft: InfoWater
3. Wallingford Software: InfoWorks WS (for water supply)

6.7 INDUSTRIAL WATER SYSTEMS

Large industrial establishments, with a heavy investment in plant, equipment, raw materials, and finished products, concentrated in a small area, are generally equipped with high-pressure fire supplies and distribution networks of their own. Because such supplies may be drawn from sources of questionable quality, some regulatory agencies require rigid separation of all private fire supplies and public distribution systems. Others permit the use of *protected cross-connections* and require their regular inspection for tightness. How the two sources of supply can be divorced without denying the protective benefit and general convenience of a dual supply to industry is illustrated in Fig. 6.16. Ground-level storage and pumping are less advantageous.

A widely approved arrangement of double check valves in vaults accessible for inspection and test by the provision of valves, gauges, and bleeders is shown in Fig. 6.17. No outbreak of waterborne disease has been traced to approved and properly supervised cross-connections of this kind. Automatic chlorination of the auxiliary supply can introduce a further safeguard.

6.8 MANAGEMENT, OPERATION, AND MAINTENANCE OF DISTRIBUTION SYSTEMS

For intelligent management of distribution storage, reservoir levels must be known at all times of the day and night. Where levels cannot be observed directly by gauges or floats,

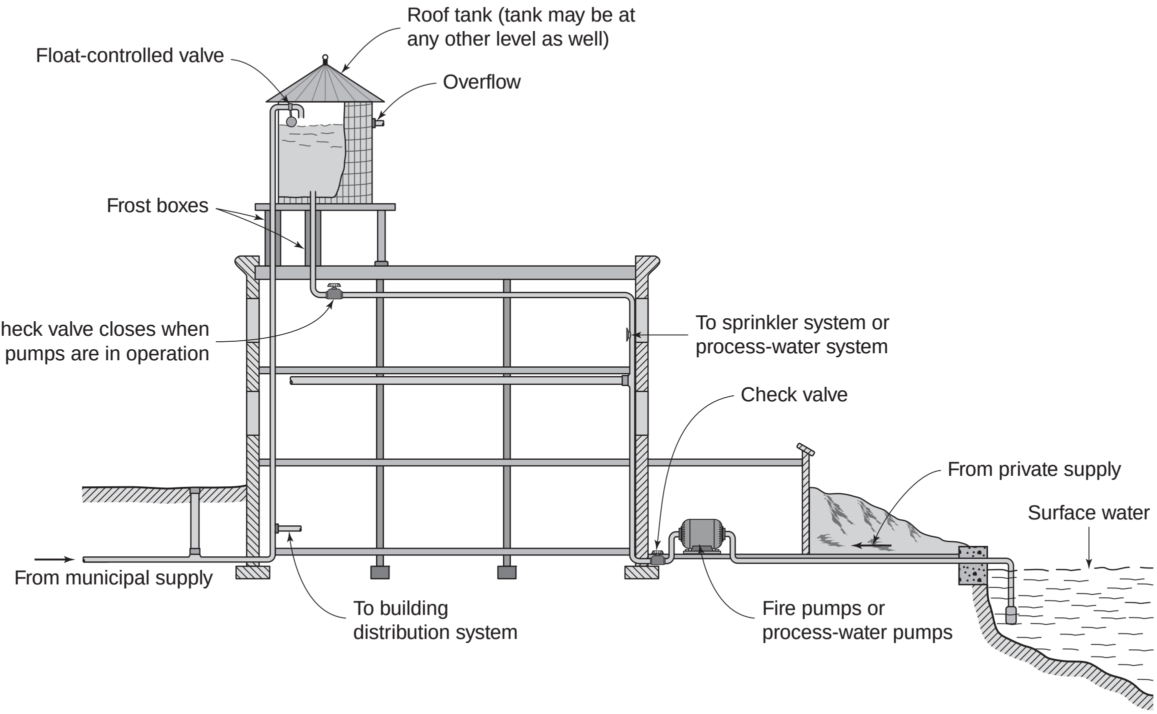


Figure 6.16 Use of industrial water supply without cross-connection (After Minnesota State Board of Health).

electrically operated sensors and recorders can transmit the required information to operating headquarters.

Well-kept *records* and *maps of pipes* and *appurtenances* are essential to the efficient operation and maintenance of distribution systems. To avoid the occasional discharge of roiled water, piping should be flushed systematically, usually through hydrants. Dead-ends need particular attention; a bleeder on a dead-end will counteract the effects of sluggish water movements. Disinfecting newly laid or newly repaired pipe is important.

There is little flow through service pipes at night, and they may freeze in very cold weather. If water mains themselves are placed at a reasonable depth and enough flow is maintained in the system, they should not freeze. Pipes deprived of adequate cover by the regrading of streets or subjected to protracted and exceptionally cold weather can be protected by drawing water from them through services. Pipes exposed on bridges or similar crossings should be insulated. Large and important lines may be heated where exposure is severe. In very cold climates, water and sewer pipes are often laid in a heated boxlike conduit, known as a *utilidor.*

Frozen pipes are usually thawed by electricity. A transformer connected to an electric power circuit, or a gasoline-driven generator of the electric-welding type, supplies the current: 100,200 A at 310 V for small pipes and up to several thousand amperes at 55 or 110 V for large mains. The current applied is varied with the electrical resistance and the melting point of the pipe metals. Nonmetallic jointing and caulking compounds and or plastic pipes obstruct current flow. Electric grounds on interior water piping, or the piping itself, must be disconnected during thawing operations. Grounds are needed but are an annoyance when they carry high voltages into the pipes and shock workmen. Pipes and hydrants can also be thawed with steam generated in portable boilers and introduced through flexible block-tin tubing.

Loss of water by *leakage* from distribution systems and connected consumer premises should be kept under control by leakage surveys.

Remember that the best way to prevent problems and accidents is to minimize hazards and "design them out" early in the design process (see Chapter 20). Three illustrative examples from the National Institute for Occupational Safety and Health (NIOSH) are summarized in the following subsections.

6.8.1 General Maintenance Person Asphyxiated While Attempting to Repair Water Leak

The victim worked as a general maintenance person for a construction company, which employed 13 persons. The construction company provided construction-related maintenance for a local chain of restaurants. A supervisor for the construction company instructed a maintenance person (the victim) to inspect and repair a leaking water valve. The water valve (a screw handle type) controlled the flow of water from the municipal water system to a local restaurant. After the supervisor instructed the victim, he then left the site of the restaurant to check on another job.

Apparently the victim proceeded to the fiberglass water meter pit (14 in. in diameter by 4 ft deep or 350 mm diameter by 1.22 m deep) approximately 25 ft (7.62 m) from the side of the restaurant where the water valve was located. The water

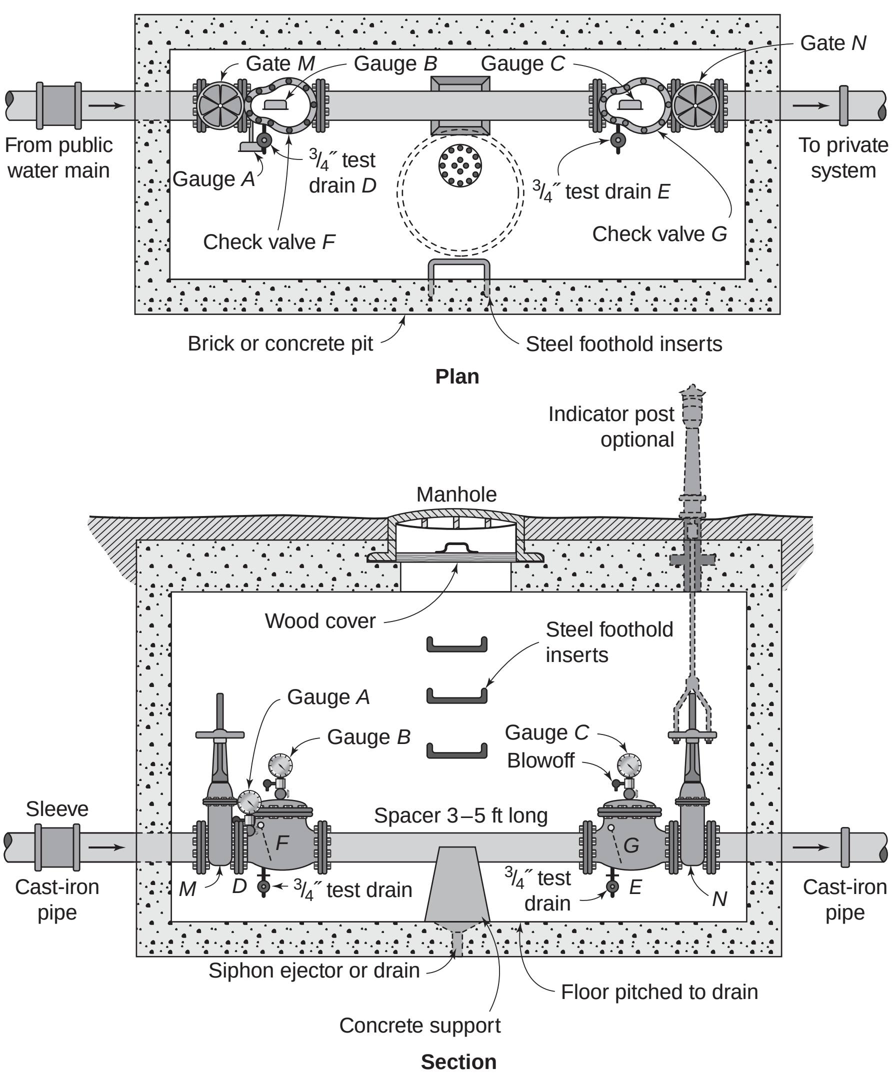


Figure 6.17 Cross-connection between municipal water supply and private (industrial) water supply protected by double check-valve installation. To test installation: (1) close gates M and N; (2) open test drain B, and observe gauges A and B; (3) open test drain E, and observe gauge C. If check valves F and G are tight, gauge A will drop to zero; gauges B and C will drop slightly owing to compression of rubber gaskets on check valves F and G. Conversion factors: 1″ = 1 in. = 25.4 mm; 1 ft = 0.3048 m.

meter pit was buried in the ground and the top of the pit was at ground level. A metal cap was attached to the rim of the water meter pit and a water meter with an in-line shutoff valve. A screw handle water valve and the municipal water line were located in the pit. The valves were approximately 36 in. (914 mm) below the top of the pit (or ground level). The victim removed the metal cap covering the pit and placed the cap on the ground next to the pit opening. He then knelt beside the opening on both knees and reached into the pit until his head, both arms, and part of his shoulders were inside the water meter pit. Apparently, the victim became stuck upside down in the opening and could not free himself, causing *asphyxiation* due to *positional deprivation of air*.

Recommendation #1: Supervisory personnel should routinely monitor employee performance to determine if employees have impaired physical and mental capabilities, which may be related to the use of alcohol, illegal or over-the-counter drugs, or prescription medications.

Recommendation #2: Supervisory personnel should identify, evaluate, and address all possible hazards associated with the job site. When employees are expected to work alone at job sites, the area should first be evaluated and all possible hazards identified and addressed by supervisory personnel. The location of the water valve inside the water meter pit required the use of extension tools, thereby eliminating the need to enter the water meter pit (even partially).

6.8.2 Plumber Repairing a Water Line Killed When Struck by a Backhoe Bucket

A 47-year-old male plumber died when he was struck by the bucket of a backhoe while he was repairing a water line in a trench for a shopping center. The victim and an apprentice plumber were at the bottom of the trench removing straight and angled cast-iron pipes and placing them in the backhoe bucket. The operator of the backhoe was an employee of the shopping center. The operator told the police investigator that when he released all controls, the bucket swerved to the left striking the victim on his left side and pinning him against the trench wall.

A trench had to be excavated to reach the pipe. The trench was an oblong shape measuring approximately 31 ft

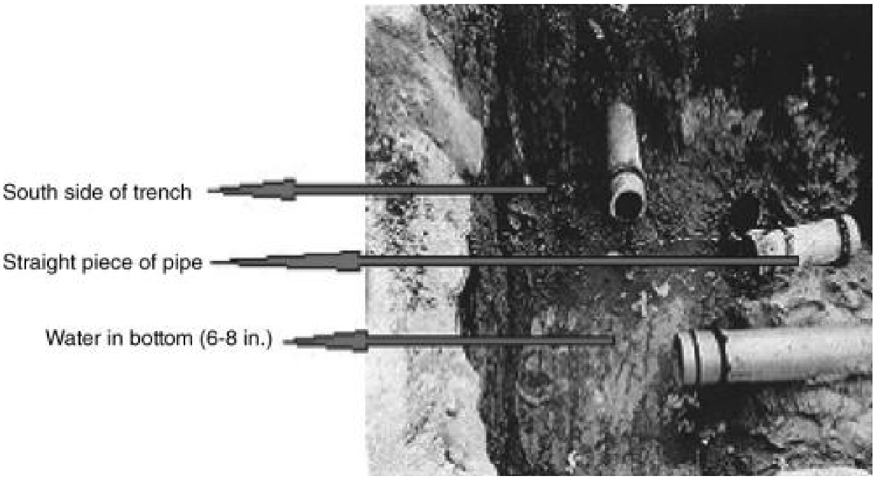


Figure 6.18 Area of incident.

(9.45 m) long. One end was about 5 ft (1.52 m) wide and the other end, where the incident occurred, was approximately 12 ft (3.66 m) wide. The trench depth was 6 ft (1.83 m) at the edges going down to 7 ft (2.13 m) where the pipe was located. About 6–8 in. (150–200 mm) of water was on the bottom of the trench (see Fig. 6.18). There was no shoring, shielding, or sloping of the trench walls.

An employee (hereinafter referred to as "operator") of the shopping center operated the backhoe to dig the trench. He had been with the firm for approximately 13 years and was in charge of the safety of personnel, property, and the public for the mall. He had training on operating backhoes prior to employment with the firm and had operated the same model of backhoe on jobs at the shopping center with the plumbing contractor. However, operating a backhoe was not part of his regular duties. The operator had requested this particular model of backhoe be used, instead of the backhoe the plumbing company owned. According to the operator, the victim asked him to operate the backhoe because he was more familiar with that particular model.

The backhoe was a farm tractor type, all-purpose backhoe/loader with rubber tires and a 75 HP (56 kW), diesel, four-cylinder engine. It had a general purpose 18 in. (457 mm) bucket weighing 255 lb (116 kg) with foot controls to swing it left (left throttle) and right (right throttle). Hand levers raised, lowered, and extended the two-part 430 lb (195 kg) boom with the bucket attached at the end (see Fig. 6.19) to a maximum of 25.5 ft (7.77 m). It had outriggers for stabilization, which were in position on the day of the incident. The backhoe was positioned about 4 ft (1.22 m) from the edge of the trench on the asphalt parking lot behind the curb. The boom with attached bucket was extended and angled over and down into the trench. The operator, if sitting in the seat of the backhoe, could not see activity down in the trench.

To repair the leak, the victim and apprentice plumber had to remove two sections of the old pipe and install new pipe. The pipe sections were cast iron and were 10 in. (250 mm) in diameter with the straight section 3 ft (0.91 m) long and the other section a 90-degree angle section or "L" joint. The straight piece weighed about 10 lb (4.54 kg) and the "L" joint weighed between 30 and 40 lb (14 and 18 kg).

The victim and apprentice plumber were in the trench trying to assemble the 3 ft (0.91 m) section to the 90-degree "L" joint but there was not enough room to work and there was still a large amount (6–8 in. or 150–200 mm) of water in the hole. They decided it would be easier to work on connecting the two pieces outside the trench and then return the assembled unit to the trench to connect to the other pipe lines. To get the pipes out, they decided to place them in the bucket and have them raised out of the trench.

The bucket of the backhoe was positioned about 4 ft (1.4 m) down in the trench close to the pipe line. The victim was facing east about 1 ft directly to the left and in the line of contact with the bucket. The apprentice plumber was also to the left of the bucket but out of its reach and was facing west looking at the victim.

The "L" pipe had just been placed in the bucket by the victim and apprentice plumber when, unexpectedly, the

Figure 6.19 Bucket and boom of backhoe.

255 lb (102 kg) bucket with the approximately 40 lb (18 kg) of pipe swung swiftly to the left and struck the victim in the left arm and upper chest, shoving him about 6 in. and pinning him into the south trench wall. The apprentice plumber yelled at the operator who maneuvered the bucket to the right to release the victim, who then slumped to the ground, groaning.

The operator called emergency personnel, who arrived at the scene in 3 min. The victim had no pulse or respiration. Ambulance personnel removed the victim from the hole and transported him to a hospital where he was pronounced dead of blunt force injuries to the chest. The manner of death was determined to be *accidental*. Hospital personnel told a police officer that death was caused by a *ruptured myocardium* (*heart*).

Recommendation #1: Employers should ensure that backhoe operators shut down backhoes according to the manufacturer's directions and direct workers to remain an adequate distance from operating backhoes. According to the apprentice plumber, the backhoe was running with its bucket positioned about 4 ft (1.22 m) down in the trench directly in line of contact with the victim. If the backhoe had been shut down properly and had workers been directed to remain an adequate distance from the backhoe the fatality may have been averted.

Recommendation #2: Employers should designate a competent person to supervise trench activities. In work involving trenches the Occupational Safety and Health Administration (OSHA) requires a competent person to be responsible for trench activities. A competent trench worker should be able to perform the following duties:

1. View the "big picture" and recognize by proper assessment the present and potential hazards at a site.
2. Classify soil types and changes to the soil composition.
3. Determine the need for sloping, shoring, or shielding.
4. Examine and approve or disapprove material or equipment used for protective systems.
5. Identify utilities located parallel to or crossing the trench.
6. Conduct daily inspections prior to the start of work and as needed. Inspect the adjacent areas and protective systems for indications of failure and hazardous atmospheres and conditions.
7. Inspect the trench after every rainstorm or other hazard-increasing occurrence.
8. Determine suitability of job site for continued work after water accumulation.

Having employees in the trench when the backhoe was running was a hazardous condition for several factors: the vibration of the soil caused by the running machine, the overhead load of the bucket and boom, and the limited visibility for the backhoe operator of activities in the trench. Under any of these conditions, a competent person would remove the exposed employees from the trench until hazards have been controlled.

Recommendation #3: Employer should ensure that workers are protected from cave-ins by an adequate protective system. A protective system designed for the soil conditions found in this excavation could have included a trench shield (also known as a trench box), shoring, or a combination of shoring and shielding. Employers can consult with manufacturers of protective systems to obtain detailed guidance for the appropriate use of these products. In this incident, no protective system had been placed at any point in the 31-ft (9.45-m)-long trench. Although the victim in this incident was not killed because of a trench cave-in, protective systems should always be used in trenches greater than 5 ft (1.52 m) deep.

6.8.3 Welder Killed Following a 100 ft (30 m) Fall from a Water Tower

A welder was killed after falling 100 ft (30 m) from a leg of a municipal water tower. The victim was a 25-year-old male welder who had been working for the company as a tower hand for 4 years. He had previously worked as a welder for a municipality. The incident occurred while the victim and a coworker were welding antenna support brackets onto the leg of the tower. The victim apparently disconnected his fall protection and was climbing the leg of the tower when he fell approximately 100 ft (30 m) to the ground.

The water tower consisted of a 40 ft (12.2 m) diameter steel water tank mounted on six structural steel legs. Each of the legs was made of welded plate steel with structural steel braces. Although not specifically designed for climbing, these braces were spaced near enough that they could be used as a ladder. The top of the tower was accessed by a steel ladder with a ladder cage that was built on one of the legs. A pump house was located near the bottom of the ladder and was the terminating point for a number of antennas and utility cables leading from the tower.

The day of the incident was clear and sunny. The foreman and three tower hands arrived in the morning and set up the hoist system on the tower leg. The victim was using a lineman's belt that he owned and had modified to make the seat strap more comfortable. Work proceeded uneventfully until the victim and a coworker were finishing the welding of a bracket about 100 ft (30 m) up the tower leg. The coworker stated that they had been up on the leg for a while and that the victim had been tied off to the leg at the same location for about 15 minutes before the incident. The coworker had completed his weld and handed the welding equipment down to the victim, who was working below him. The victim then finished his weld and began to chip and paint. Noticing that there was a fault in the weld above his head, the victim called down to the foreman to start up the welder so he could redo some spots. He then started to climb up the tower. As the foreman turned to start the machine, he heard the victim yell and saw him fall to the ground. The police and emergency medical services personnel were called and arrived within a few minutes of the fall and began treating the victim, who was still breathing. During this time the shaken coworker was lowered to the ground on the material hoist line. The victim was transported to the local medical center where he was pronounced dead.

Investigators concluded that these *guidelines* should be followed in order to prevent similar incidents in the future:

1. Employers must thoroughly plan all work and perform a job hazard analysis of the site prior to starting work.
2. When practical, employers should provide and require the use of a stable work platform for working at elevated worksites.
3. Employers should provide a system of fall protection that protects employees at all times when working at elevations.
4. Employers should ensure that fall protection equipment is appropriate and maintained in good condition. Employees should inspect fall protection equipment before each use to ensure that all components are in operational order.
5. Employers should ensure that material hoists are not used for raising or lowering employees to or from the worksite.
6. Employers should ensure that electrical safety practices are followed when welding.
7. Owners of water towers and similar structures should design and install a permanent static safety line system on the tower to facilitate the use of fall protection devices.

6.9 PRACTICAL DESIGN AND ANALYSIS OF WATER DISTRIBUTION SYSTEMS

The technical information presented in this section is important to practical design of a new water distribution system from scratch, or analysis of an existing (or a newly designed) system. A design engineer or a government reviewer must design/review a water distribution in accordance with appropriate government requirements and/or professional standards. The authors selected the *Recommended Standards for Water Works* (i.e., *Ten-State Standards; 2007*), the State of Iowa's *Statewide Urban Design and Specifications – Design Manual* (*2012*), and the State of Florida Hillsborough County's subdivision review guidelines, *Drinking Water Distribution Systems* (*2011*) as the typical examples of regional, statewide, and countywide standards. A professional engineer must carefully select appropriate standards for design and/or review of a water distribution system.

Water distribution systems shall be designed to maintain treated water quantity and quality. Special consideration should be given to sizing of distribution main, providing for multidirectional flow design, adequate valving for flow and pressure controls, and provisions for air release and adequate flushing. Water systems should be designed according to the federal, state, and/or county standards to maximize water turnover and to minimize residence times while delivering acceptable water flows, and pressures. In the absence of any government standards, design criteria and materials meeting applicable water professional standards (such as American Water Works Association standards and National Sanitation Foundation standards) can be accepted by the reviewing authority for adoption.

6.9.1 Minimum Design Period Requirements

Water mains should have a minimum size based on a hydraulic analysis utilizing 20-year design for a specified water demand. The projected land uses and water demand based on full service area development should be considered. The specified water demand selected for design depends on the area to be serviced and the type of water main (feeder, arterial, distribution).

6.9.2 Water Pressure Requirements

The water system shall be designed to maintain a minimum of 20 psi (140 kPa) at ground level at all points in the water distribution system under all conditions of flow (such as fire flow conditions), at any hydrant or any point in the water system. The normal working pressure in the water distribution system should be approximately 60–80 psi (410–500 kPa), and no less than 35 psi (240 kPa). The State of Florida Hillsborough County requires a minimum water pressure of 35 psi (240 kPa) in a water transmission line, under any flow conditions. The State of Iowa recommends that individual or system pressure reducing devices be used when operating pressure exceeds 100 psi (625 kPa).

6.9.3 Minimum Size Requirements

All water mains should be sized large enough to provide existing and future residential, commercial, and industrial water demands and fire protection flows to the area to be served. The Ten-State Standards state that (a) the minimum size of a water main which provides for fire protection and serving fire hydrants shall be 6 in (152.4 mm) diameter; and (b) the minimum size of water main in the water distribution system where fire protection is not to be provided should be a minimum of 3 in. (26.2 mm) in diameter. The minimum pipe sizes for distribution mains shall be 4 in. (101.6 mm) in diameter in the State of Florida Hillsborough County, but 8 in. (203.2 mm) in the State of Iowa, unless otherwise approved by the jurisdictional authority.

Arterial or feeder mains, typically 12 in. (304.8 mm) and larger, should conform to an existing grid pattern. Readers are referred to Section 6.2 for more information on other common standards for sizing pipes.

6.9.4 Velocity Requirements

Velocity of flow is another factor recommended by the State of Iowa in determining the flow of water pipes and, therefore, the required pipe diameter. Velocities should normally be 5 ft/s (1.52 m/s) or less, due to high friction losses occurring at greater velocities. Of course, this velocity requirement may be exceeded under fire flow conditions.

The Ten-State Standards recommends that dead-end mains be equipped with a means to provide adequate flushing and flushing devices be sized to provide flows which will give a velocity of at least 2.5 ft/s (0.762 m/s) in the water main being flushed.

6.9.5 Pipes and Valves Spacing Requirements

A sufficient number of valves shall be provided on water mains to minimize inconvenience and hazards during water system repairs. Common standards on the pipe spacing and gate valve spacing can be found from Section 6.2.

6.9.6 Hydrant Spacing, Location, and Fire Flow Requirements

There are two kinds of hydrants: (a) fire hydrants and (b) flushing hydrants. Fire hydrants should be provided at each street intersection and at intermediate points between intersections as recommended by the Ten-State Standards. Generally, fire hydrant spacing ranges from 150 to 1,000 ft (45.72–304.8 m) depending on the area being served and the location where the water system is constructed. The following is a summary of three sources of standards:

1. Common standards: The number, spacing distribution, and fire flow requirements of fire hydrants are presented in Table 6.1.
2. The Ten-State Standards: (a) Fire hydrant spacing ranges from 350 to 600 ft (106.7–182.9 m) depending on the area being served; and (b) flushing hydrants (instead of fire hydrants) are provided on the water systems not designed to carry any fire flows and should give a velocity of at least 2.5 ft/s (0.76 m/s) in water main during flushing.
3. The State of Florida Hillsborough County Standards: (a) In manufacturing and industrial areas, fire hydrants shall be placed every 300 ft (91.4 m) along the right-of-way with a maximum of 150 ft (45.7 m) to the last lot, and the minimum required fire flow shall be 1,000 gpm (3,785 L/min) provided by either each hydrant individually or multiple hydrants flowing simultaneously; (b) in commercial and apartment areas, fire hydrants shall be placed every 500 ft (152.4 m) along the right-of-way with a maximum of 250 ft (76.2 m) to the last lot, and the minimum required fire flow shall be 1,000 gpm (3,785 L/min) provided by either each hydrant individually or multiple hydrants flowing simultaneously; (c) in residential areas, fire hydrants shall be placed a maximum of 500 ft (152.4 m) apart along the right-of-way with a maximum of 500 ft (152. m) to the last lot, and the minimum flow from each hydrant shall be 750 gpm (2,839 L/min).

6.9.7 Air Relief Valve Requirements

At high points in water mains where air can accumulate, provisions shall be made to remove the air by means of air relief valves (Fig. 5.26; Section 5.9.3). Use of manual air relief valves is recommended. Automatic air relief valves may be used in situations where floods of the manhole or chamber may not occur.

6.9.8 Depth of Cover Requirements

Water mains shall be covered with sufficient earth or other insulation to prevent freezing. In Florida, cover as measured from finished grade to top of the pipeline shall be a minimum of 36 in. (914.4 mm) for pipe diameters up to and including 12 in. (304.8 mm). Depth of cover for pipes 14 in. (355.6 mm) or greater in diameter shall be a minimum of 48 in. (1219.2 mm). Deeper cover depth is needed when automatic air release valves are used.

Section 5.7.3 introduces the authors' engineering equations for determination of the depth of cover in cold weather regions.

6.9.9 Separation of Water Mains from Sources of Contamination

In accordance with the Ten-State Standards, water mains shall be laid at least 10 ft (3.05 m) horizontally from any existing or proposed gravity sanitary sewer, gravity storm sewer, sanitary sewer force main, septic tank, sewer manholes, or subsoil treatment system. The horizontal separation distance shall be measured edge to edge. In cases where it is not practical to maintain a 10 ft (3.05 m) separation distance, the reviewing authority may allow deviation on a case-by-case basis, if supported by data from the design engineer.

Water mains crossing gravity sanitary sewer, gravity storm sewer, or sanitary sewer force main shall be laid to provide a minimum vertical separation distance of 18 in. (45.72 cm) between the outside of the water main and the outside of the sewer. This shall be the case where the water main is either above or below the sewer with preference to the water main located above the sewer. Besides, one full length of water pipe shall be located at crossings, so both joints will be as far from the sewer as possible. Special structural support for the water and sewer pipes may be required.

When it is impossible to obtain the minimum specified horizontal separation distances, the reviewing authority must specifically approve any variance from the above requirements, if the following corrective methods of installation may be used:

1. Such deviation may allow installation of the water main closer to a sewer, provided that the water main is laid in a separate trench or on an undisturbed earth shelf located on one side of the sewer at such an elevation that the bottom of the water main is at least 18 in. (45.72 cm) above the top of the sewer.
2. The sewer materials shall be water works grade 150 psi (1,035 kPa) pressure rated pipe meeting AWWA standards or pipe approved by the reviewing authority and shall be pressure tested to ensure water tightness.

Surface water crossings, whether over or under water, present special problems. The following are the recommended Ten-State Standards:

1. For the above-water crossings, the pipe shall be adequately supported and anchored, protected from vandalism, damage, and freezing, and accessible for repair or replacement.
2. For the underwater crossings, a minimum cover of 5 ft (1.52 m) shall be provided over the pipe unless otherwise approved by the reviewing authority. When crossing water courses, which are greater than 15 ft (4.57 m) in width, the following shall be provided: (a) the pipe shall be of special construction, having flexible, restrained, or welded watertight joints; (b) valves shall be provided at both ends of water crossings so that the section can be isolated for testing or repair, and the valves shall be easily accessible, and not subject to flooding; (c) permanent taps or other provisions shall be provided to allow insertion of a small meter to determine leakage and obtain water samples on each side of the valve closest to the supply source.

6.9.10 Head Loss of Water System Fittings

Appendix 25 shows the equivalent pipe length to the head loss in fittings. A design engineer may calculate the equivalent pipe lengths of all major fittings for a detailed water distribution system analysis. Contact the manufacturers for the missing fittings not listed in Appendix 25.

EXAMPLE 6.7 DETERMINATION OF EQUIVALENT PIPE LENGTH FOR A STANDARD ELBOW

Determine the equivalent pipe length of a 6 in. (152.4 mm) standard elbow.

Solution:

The dotted line in Appendix 25 shows that the resistance (or head loss) of a 6 in. (152.4 mm) standard elbow is equivalent to approximately 16 ft (4.88 m) of 6 in. (152.4 mm) standard pipe.

EXAMPLE 6.8 DETERMINATION OF EQUIVALENT PIPE LENGTH FOR A GATE VALVE

Determine the equivalent pipe lengths of a 6 in. (152.4 mm) gate valve when it is (a) fully open and (b) $^1/_4$ closed.

Solution:

a. From Appendix 25, the resistance of a 6 in. (152.4 mm) gate valve is equivalent to approximately 3.5 ft (1.07 m) of 6 in. (152.4 mm) standard pipe if the gate valve is fully open.
b. The resistance of a 6 in. (152.4 mm) gate valve is equivalent to approximately 19 ft (5.79 m) of 6 in. (152.4 mm) standard pipe if the gate valve is $^1/_4$ closed.

EXAMPLE 6.9 DETERMINATION OF EQUIVALENT PIPE LENGTH FOR A SUDDEN ENLARGEMENT

Determine the equivalent pipe length of a sudden enlargement from 6 in. (152.4 mm) to 12 in. (304.8 mm) in diameter.

Solution:

Use Appendix 25 to find the solution.

$$d/D = 6\text{ in.}/(12\text{ in.}) = 152.4\text{ mm}/(304.8\text{ mm}) = {}^1/_2.$$

The resistance of the sudden enlargement ($d/D = {}^1/_2$) is equivalent to approximately 9 ft (2.74 m) of a 6 in. (152.4 mm) standard pipe.

PROBLEMS/QUESTIONS

6.1 An elevated water reservoir (water level at 210 m) is to supply a lower groundwater reservoir (water level at 138 m). The reservoirs are connected by 800 m of 400 mm ductile cast pipe ($C = 100$) and 1200 m of 300 mm ductile cast-iron pipe ($C = 100$) in series.

(a) What will be the discharge delivered from the upper reservoir to the lower one?

(b) If the water demand increases by 50% in the future, determine the required size of pipe that has to be installed in parallel with the existing piping.

6.2 The following data are given for the water supply network shown in Fig. 6.20:

Pipe	*d* (in.)	*L* (ft)
AB	8	2,500
BC	8	500
AD	8	500
DC	12	2,500

The elevation of point A is 100 ft and of point B is 90 ft. The C value for all pipes is 100. Find the following:

(a) The length of a single 18 in. pipe that can replace the water network from A to C.

(b) The residual pressure at point C, given that the flow through the network is 4,000 gpm and the pressure at point A is 40 psi.

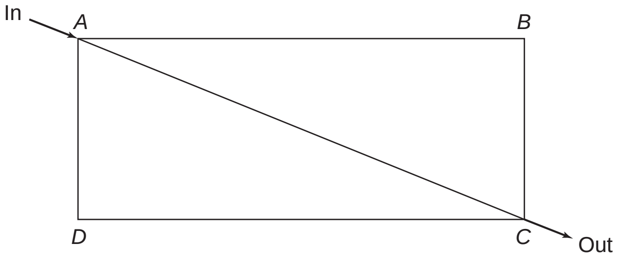


Figure 6.20 Water network for Problems 6.2 and 6.3.

6.3 The following data are given for the water supply network shown in Fig. 6.20:

Pipe	*d* (mm)	*L* (m)
AB	200	800
BC	200	200
AD	200	200
DC	300	800

The elevation of point A is 600 m and of point B is 590 m. The C value for all pipes is 100. Find the following:

(a) The length of a single 400 mm pipe that can replace the water network from A to C.

(b) The residual pressure at point C, given that the flow through the network is 250 L/s and the pressure at A is 300 kPa.

6.4 For the water supply network shown in Fig. 6.21, the following data are given:

Pipe	*d* (mm)	*L* (m)
AB	300	1,200
BD	300	300
AC	200	300
CD	200	1,200
SA	d_{SA}	10,000

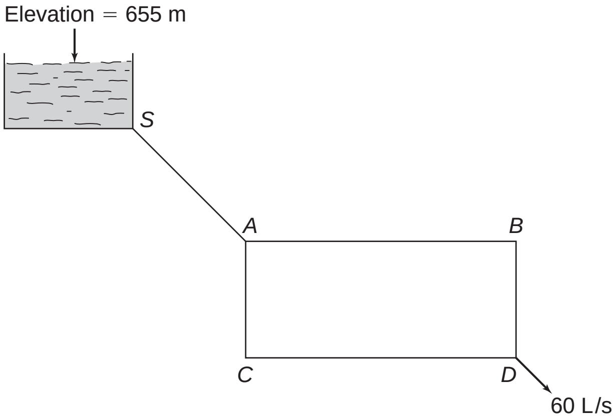


Figure 6.21 Water network for Problem 6.4.

Elevation of loop ABCD is 600 m and the value of C for all pipes = 100.

Determine the needed diameter of main SA so that the residual pressure at any point in the network ABCD does not drop below 250 kPa.

6.5 Potable water is supplied to a city via an 800 mm transmission line at a flow rate of 550 L/s (see Fig. 6.22). A pressure gauge located 500 m upstream from point A registers 6.5 bars at normal operation. The following data are given:

Pipe	*d* (mm)	*L* (m)
PA	800	500
AmnB	500	1,500
AxyB	300	200
BC	800	—

All pipes are made of the same material with a Hazen–Williams coefficient $C = 100$. The elevations of points P, A, and B are 650, 660, and 635 m, respectively.

(a) Determine the length of an equivalent 500 mm single pipe between points A and B.

(b) Determine the actual flows in the two branches AmnB and AxyB.

(c) Find the residual pressures at both points A and B.

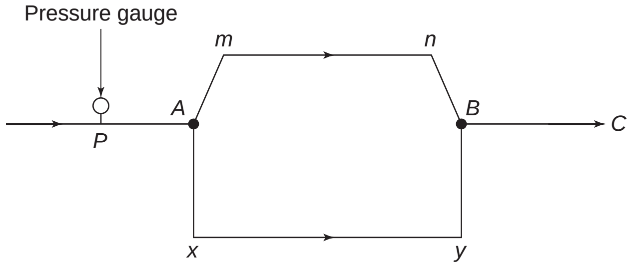


Figure 6.22 Water network for Problem 6.5.

6.6 The layout of a water distribution system is shown in Fig. 6.23. All pipes of the network are made of plastic ($C = 150$). The maximum flow in the system $Q = 5{,}000$ gpm. The following data are given:

Pipe	d (in.)	L (ft)
AB	14	1,000
BC	14	2,500
CD	14	500
DE	14	2,000
AE	10	3,000
AF	12	1,500
FE	10	2,500

(a) Find the length of a single 16 in. diameter pipe that can replace the system from A to E.

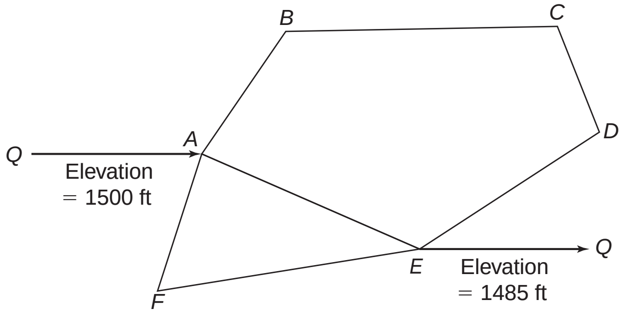


Figure 6.23 Water network for Problem 6.6.

(b) Find the residual pressure at point E, if the available pressure at point A is 60 psi.

6.7 Check and reinforce (if required) sections d-d and e-e in Example 6.2.

6.8 The layout of a water distribution system is shown in Fig. 6.24. Elevation of water in reservoir = 2,000 ft and elevation of point A = 1,770 ft. All pipes are made of the same material with a Hazen–Williams coefficient $C = 100$. The following data are given:

Pipe	d (in.)	L (ft)
RA	d_{RA}	20,000
AB	18	5,000
BC	8	2,500
BE	6	1,000
CD	8	2,500
DE	6	1,000
AE	10	4,000
AD	d_{AD}	5,000

(a) Determine the required diameter of main from water reservoir to point A, if the minimum required pressure at point A is 60 psi.

(b) Determine the required diameter of pipe AD, assuming that the maximum allowable head loss in the network (ABCDE) is 3 ft/1,000 ft.

(c) Determine the required pipe diameter that is to be constructed in parallel with BC assuming that the maximum allowable head loss in the network (ABCDE) is 3 ft/1,000 ft.

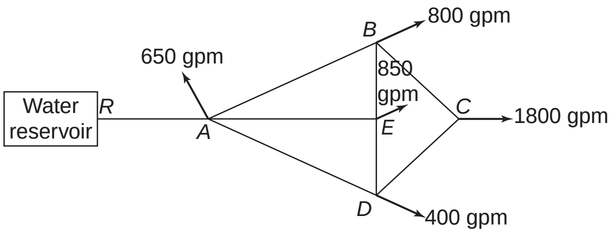


Figure 6.24 Water network for Problem 6.8.

6.9 The layout of a water distribution system is shown in Fig. 6.25. Elevation of water in reservoir = 600 m and elevation of points A and C are 531 and 500 m, respectively. All pipes are made of the same material with a Hazen–Williams coefficient $C = 100$. The following data are given:

Pipe	d (mm)	L (m)
RA	d_{RA}	6,000
AB	400	1,500
BC	200	750
BE	150	300
CD	300	750
DE	150	300
AE	250	1,200
AD	d_{AD}	1,500

(a) Determine the required diameter of main from water reservoir to point A, if the minimum required pressure at point A is 450 kPa.

(b) Determine the required diameter of pipe AD, assuming that the maximum allowable head loss in the network (ABCDE) is 3‰.

(c) Determine the required pipe diameter that is to be constructed in parallel with BC assuming that the maximum allowable head loss in the network (ABCDE) is 3‰.

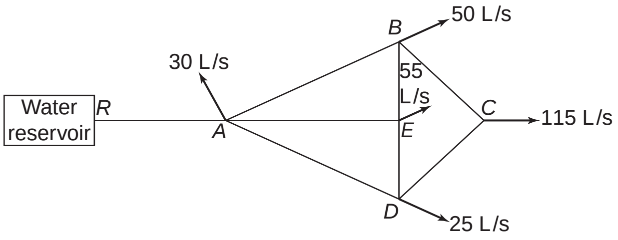


Figure 6.25 Water network for Problem 6.9.

6.10 The water distribution system shown in Fig. 6.26 was designed to serve a small city. Water flows by gravity through a 400 mm water main (ABC) from the water reservoir and feeds into the network at point C. The length of the main, ABC, is 10 km.

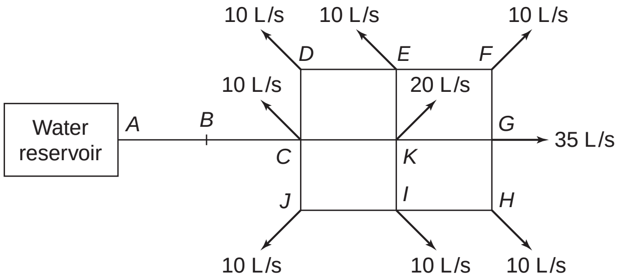


Figure 6.26 Water network for Problem 6.10.

At present, the water demand has increased to the values shown in the figure. The water level in the reservoir is at 400 m, and the whole network is located at an elevation of 340 m. Each quadrant in the network is 1,000 m long and 500 m wide. Assume all pipes are ductile iron with $C = 100$.

(a) If the pressure at point C is not allowed to drop below 300 kPa, determine the required size of an additional ductile pipe to be installed between the reservoir and point B, 7 km away along the line AC.

(b) Determine the required diameters of pipes DE, CK, and JI, assuming that all three pipes will have identical diameters and that the maximum allowable head loss in the network is 3 m/1,000 m (3‰).

6.11 The main pipe layout of a water distribution system is shown in Fig. 6.27. The water demand for the system (domestic + fire) is 2,300 gpm and is supplied at point A. At point I, the most unfavorable as far as loss of head is concerned, it is assumed that a fire plus the domestic flow requires 900 gpm. At other intersection points it is estimated that the amounts shown will be required to satisfy the normal domestic demand. The elevations at intersections of mains are as follows:

Point	Elevation (ft)	Pipe	d (in.)	L (ft)
A	95	AB	d_{AB}	500
B	95	BC	d_{BC}	400
C	100	BE	8	1,600
D	95	CF	d_{CF}	1,600
E	100	AD	16	1,600
F	105	DE	6	500
G	100	EF	6	500
H	105	DG	14	2,000
I	109	EH	6	2,000
		FI	d_{FI}	2,000
		GH	12	500
		HI	12	400

All pipes have a C value of 100.

(a) Determine the required diameters of pipes AB, BC, CF, and FG, assuming that the maximum, allowable head loss in the system is 2‰. (*Hint:* Use sections aa, bb, cc, and dd in your analysis.)

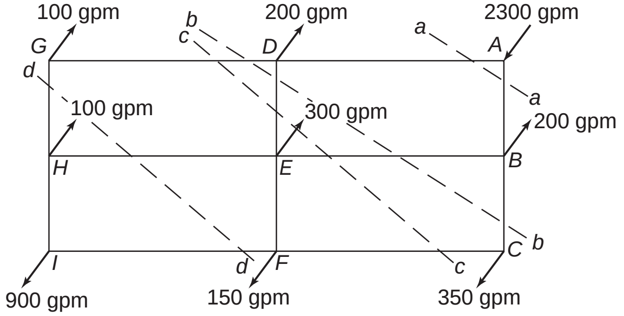


Figure 6.27 Water network for Problem 6.11.

(b) What is the minimum possible pressure at point I? Is it acceptable? Assume that a pressure of 30 psi is available at point A.

6.12 The elevations at various points in the water distribution system shown in Fig. 6.28 are as follows:

A, B, and D	528.5 m
C, E, and G	530.0 m
F and H	531.5 m
I	532.5 m

The known lengths and diameters of pipes are as shown below:

Pipe	d (mm)	L (m)
AB	d_{AB}	200
BC	250	200
DE	200	200
EF	150	200
GH	300	200
HI	d_{HI}	200
AD	400	500
BE	200	500
CF	d_{CF}	500
DG	d_{DG}	600
EH	150	600
FI	200	600

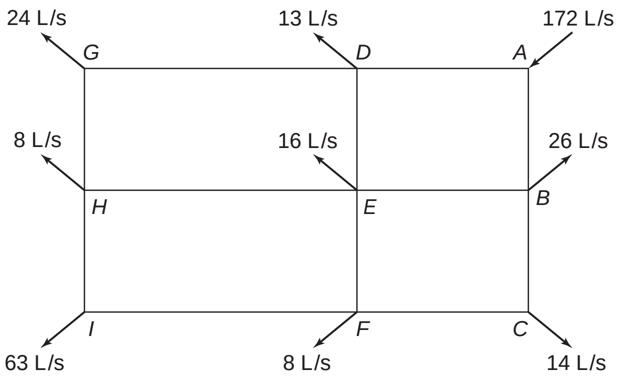


Figure 6.28 Water network for Problem 6.12.

The pressure at point A is 230 kPa and C for all pipes is 100. If the maximum allowable head loss in the system is 2.5‰, find

(a) Diameter of pipe AB.

(b) Diameter of pipe CF.

(c) Diameter of pipe DG.

(d) Diameter of pipe HI.

(e) The minimum possible pressure at point I.

6.13 Assuming water is to be delivered to a fire through not more than 500 ft of hose, find the water available (in gpm) for a fire in the center of the loop shown in Fig. 6.29. All pipes are located at the same elevation. The pressure in the 12 in. feeder pipes is 40 psi and the residual hydrant pressure is not to be less than 20 psi. C for all pipes is 100.

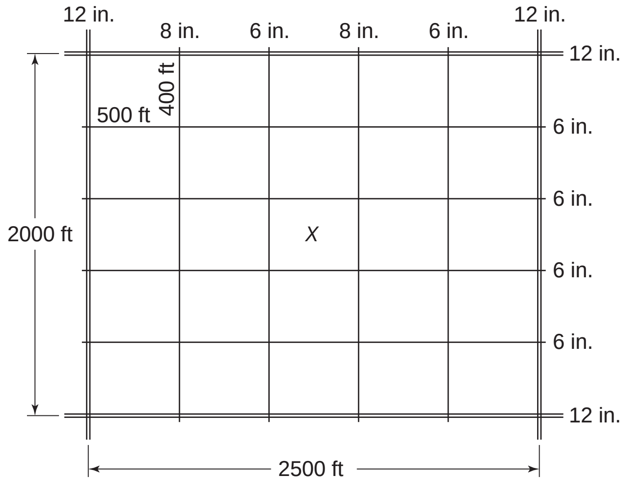


Figure 6.29 Water network for Problem 6.13.

6.14 A gridiron layout of a distribution system located on a flat area is shown in Fig. 6.30. All pipes of the network are made of ductile iron ($C = 100$). The grid serves an area that requires a design fire flow of 10,000 gpm. The pressure in the primary feeder mains ABCDA is 35 psi. All distribution pipes are 8 in. in diameter. The fire hose lines available are 400 ft long.

Supposing there is a fire at the middle of the grid (point x),

(a) Find the residual pressure at the hydrants.

(b) How many distribution lines (minimum) should be upgraded from 8 in. to 10 in. in order to maintain a residual pressure of at least 20 psi at the hydrants?

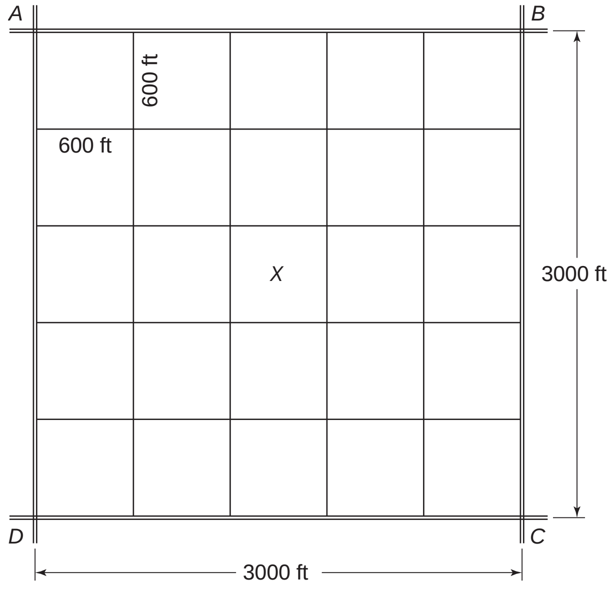


Figure 6.30 Water network for Problem 6.14.

6.15 The layout of a loop for a city water distribution system located on a flat area is shown in Fig. 6.31. All pipes of the network have a Hazen–Williams C of 100. The fire hose lines available are 150 m long. If the residual pressure in the 300 mm mains is 300 kPa, how much water (Q_F) can be withdrawn for firefighting at the center of loop without lowering the residual pressure at the hydrants below 150 kPa?

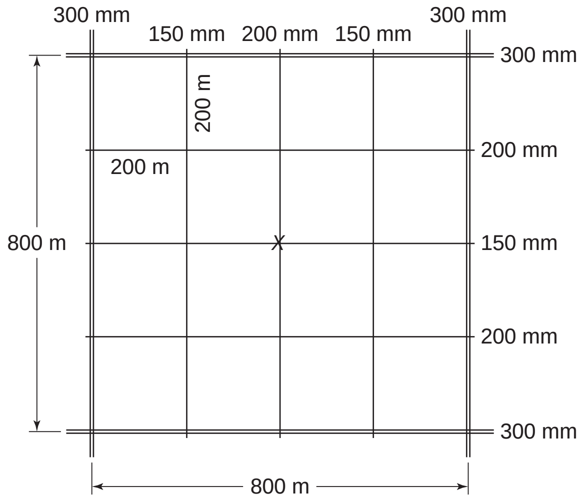


Figure 6.31 Water network for Problem 6.15.

6.16 Mains of a city water distribution loop (ABCDA) are located at an elevation of 635 m (Fig. 6.32). All distribution pipes 1, 2, 3, 4, 5, and 6 have a diameter of 150 mm and a Hazen–Williams C of 100. The fire hose lines available are 150 m long and the required fire flow is 12,200 L/min. The residual pressure in mains ABCDA is 4 bars. Domestic demand can be neglected during firefighting.

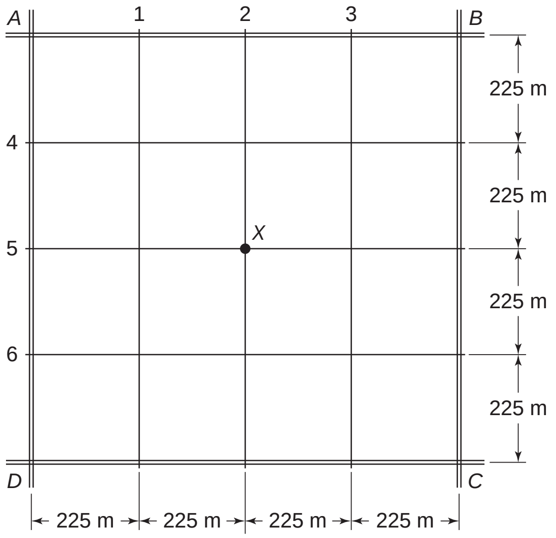


Figure 6.32 Water network for Problem 6.16.

(a) Determine the residual pressure at the hydrants (elevation = 645 m) if there is a fire at the center of the loop.

(b) What sizes should lines 2 and 5 be replaced with if the minimum required pressure at the hydrants is 150 kPa?

6.17 The layout of a loop in a water distribution system is shown in Fig. 6.33. All pipes of the gridiron network have the same elevation and a Hazen–Williams *C* of 100. The grid ABCDA serves an area that requires a design fire flow of 9,000 gpm. The pressure in the primary feeders ABCDA is 30 psi, and the fire hose lines available are 410 ft long.

Supposing there is a fire at the middle of the grid (point X),

(a) Find the residual pressure at the hydrants.

(b) How many lines (minimum) should be changed from 8 in. to 10 in. in order to maintain a residual pressure of at least 20 psi at the hydrants?

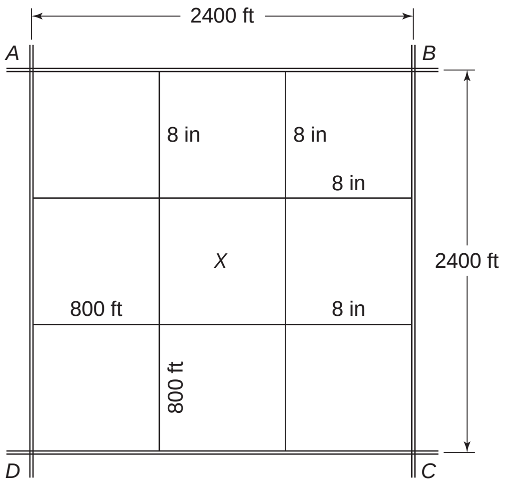


Figure 6.33 Water network for Problem 6.17.

6.18 The main pipes of a city water distribution loop (ABCDA) are located downhill at an elevation of 600 m (Fig. 6.34). All pipes in the network have a Hazen–Williams *C* of 100. The fire hose lines available are 150 m long. If the residual pressure in the mains is 4 bars, how much water (Q_{F} in L/s) can be withdrawn for firefighting at point x located at the top of the hill (elevation 619 m) without lowering the residual pressure at the hydrants below 150 kPa?

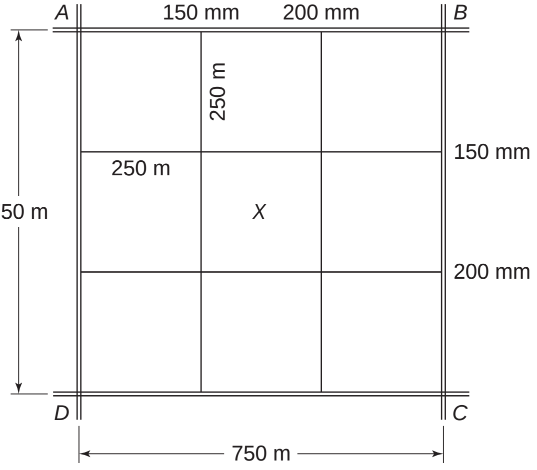


Figure 6.34 Water network for Problem 6.18.

6.19 In the water network shown in Fig. 6.35, the Hazen–Williams coefficient *C* for all pipes is 100.

(a) Using the Hardy Cross method, determine the flow in each pipe to the nearest 20 gpm.

(b) Find the residual pressures at points B and C, given that the pressure at point A is 50 psi.

(c) If the flow input at A is doubled, what would be the residual pressure at C?

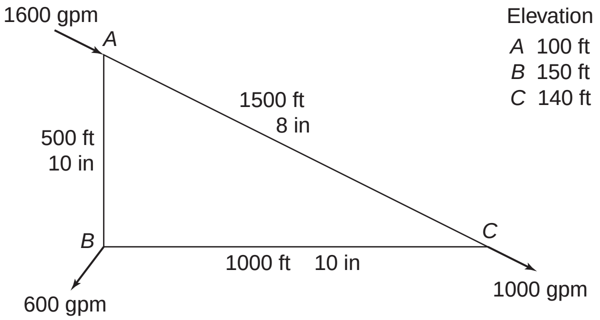


Figure 6.35 Water network for Problem 6.19.

6.20 The water supply system shown in Fig. 6.36 is designed to serve a summer resort. The flow from the reservoir is delivered to network ABC by gravity through a 14 in. main. The elevations of points A, B, and C are as follows:

A = 100 ft;
B = 150 ft;
C = 140 ft.

(a) Find the flow in each pipe.

(b) If the pressure at any point in loop ABC is not allowed to drop below 60 psi, determine the required elevation of the water reservoir.

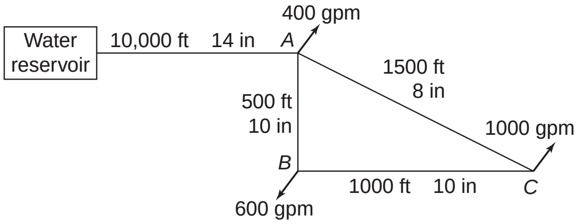


Figure 6.36 Water network for Problem 6.20.

6.21 The water supply system shown in Fig. 6.37 is designed to serve a small town ABDCA. The flow from the reservoir is delivered to the town by gravity through a 400 mm main. All pipes have a Hazen–Williams C of 100. Determine the flow rate in each pipe.

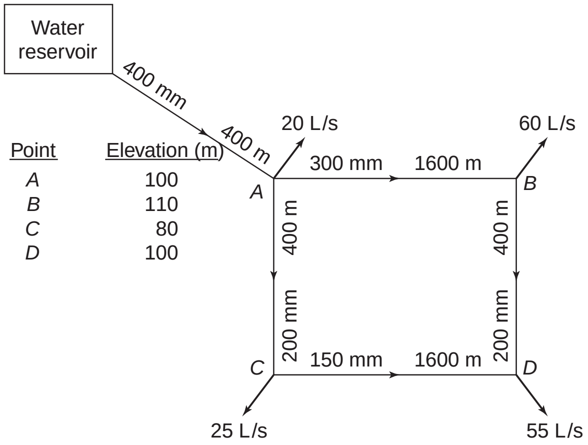


Figure 6.37 Water network for Problem 6.21.

6.22 The water supply system shown in Fig. 6.38 was designed 10 years ago to serve a small town. A water pump P delivers the required flow through a 12 in. water main from the water reservoir to the water distribution loop ABDCA.

At present the water demand has increased to 2,000 gpm, which is assumed to be withdrawn at points A, B, C, and D as shown in the figure. At these drafts the pump can deliver a head of 200 ft. All pipes are made of PVC with a Hazen–Williams coefficient C of 140. Find the flow in each pipe.

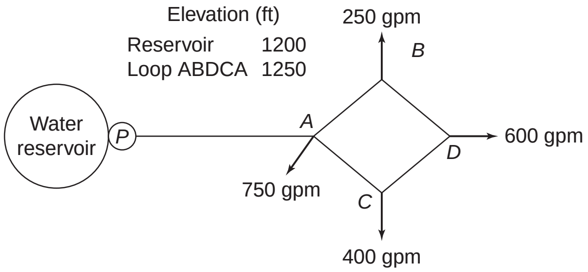


Figure 6.38 Water network for Problem 6.22.

6.23 The regional water supply system shown in Fig. 6.39 is designed to serve four small towns W, X, Y, and Z. Water is delivered from the reservoir to the four towns by pump P. The Hazen–Williams C for all pipes is 130. Pipe diameters and lengths are as follows:

Pipe	d (in.)	L (ft)
WX	10	10,000
XY	6	12,000
WZ	10	10,000
ZY	6	11,000
PW	16	40,000

Compute the flow in each pipe using an accuracy of 20 gpm.

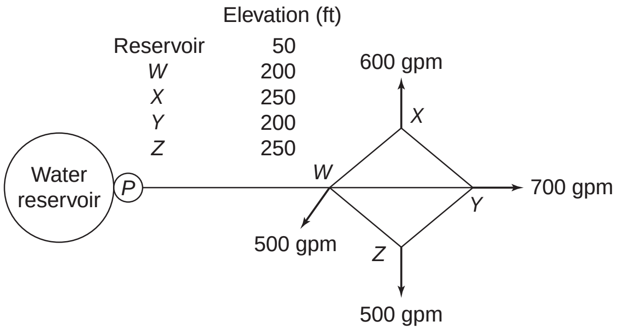


Figure 6.39 Water network for Problem 6.23.

6.24 Determine the equivalent pipe length of a 6 in (152.4 mm) gate valve when it is $^{3}/_{4}$ closed.

6.25 Determine the equivalent pipe length of a sudden contraction from 12 in. (304.8 mm) to 6 in. (152.4 mm) in diameter.

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Chapter 7

Water Distribution Systems: Modeling and Computer Applications

7.1 WATERGEMS SOFTWARE

This chapter deals primarily with the topic of pressure piping as it relates to water distribution systems. If designed correctly, the network of interconnected pipes, storage tanks, pumps, and regulating valves provides adequate pressure, adequate supply, and good water quality throughout the system. If incorrectly designed, some areas may have low pressures, poor fire protection, and even present health risks.

WaterGEMS (Haestad Methods Water Solutions by Bentley) is used in this chapter as a tool to illustrate the application of various available software programs that can help civil and environmental engineers design and analyze water distribution systems. It is also used by water utility managers as a tool to aid in the efficient operation of distribution systems. This software can be used as a stand-alone program, integrated with AutoCAD, or linked to a geographical information system (GIS) via the GEMS component.

WaterGEMS is used primarily for the modeling and analysis of water distribution systems. Although the emphasis is on water distribution systems, the methodology is applicable to any fluid system with the following characteristics: (a) steady or slowly changing turbulent flow; (b) incompressible, Newtonian, single-phase fluids; and (c) full, closed conduits (pressure system). Examples of systems with these characteristics include potable water systems, sewage force mains, fire protection systems, well pumps, and raw water pumping.

WaterGEMS can analyze complex distribution systems under a variety of conditions. For a typical WaterGEMS project, you may be interested in determining *system pressures* and *flow rates* under average loading, peak *loading*, or fire flow conditions. Extended-period analysis tools also allow you to model the system's response to varying *supply and demand schedules* over a period of time; you can even track *chlorine residuals* or determine the source of the water at any point in the distribution system. In summary, you can use WaterGEMS for

1. Pipe sizing
2. Pump sizing
3. Master planning
4. Construction and operation costs
5. Operational studies
6. Rehabilitation studies
7. Vulnerability studies
8. Water quality studies

The WaterGEMS program, in addition to other useful software, is available free to users of this book. Educational versions of the software can be accessed online or from the CD that accompanies this textbook.

7.2 WATER DEMAND PATTERNS

Using a representative diurnal curve for domestic water demand (Fig. 7.1), we see that there is a peak in the *diurnal curve* in the morning as people take showers and prepare breakfast, another slight peak around noon, and a third peak in the evening as people arrive home from work and prepare dinner. Throughout the night, the pattern reflects the relative inactivity of the system, with very low flows compared to the average.

Two basic forms are used to represent the patterns of water demand: *stepwise* and *continuous*. A stepwise pattern is one that assumes a constant level of usage over a period of time, and then jumps instantaneously to another level where it again remains steady until the next jump. A continuous pattern is one for which several points in the pattern are known and sections in between are transitional, resulting in a smoother pattern. Notice that, for the continuous pattern in Fig. 7.1, the magnitude and slope of the pattern at the start and end times are the same, a continuity that is recommended for patterns that repeat.

Because of the finite time steps used in the calculations, most computer programs convert continuous patterns into stepwise patterns for use by the algorithms, with the duration of each step equal to the time step of the analysis.

Water Engineering: Hydraulics, Distribution and Treatment, First Edition. Nazih K. Shammas and Lawrence K. Wang.

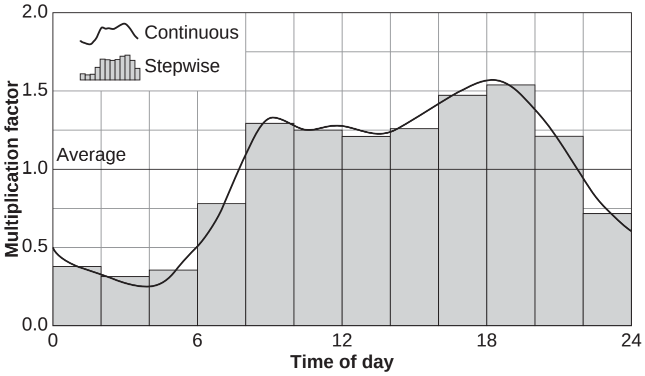

Figure 7.1 Typical diurnal curve.

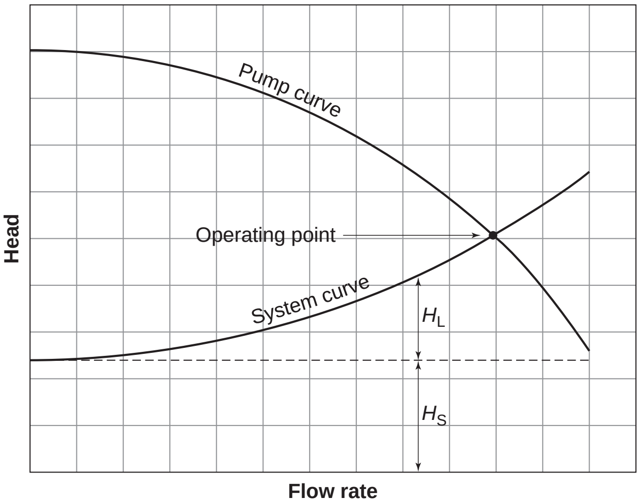

Figure 7.2 System operating point.

7.3 ENERGY LOSSES AND GAINS

The hydraulic theory behind friction losses is the same for pressure piping as it is for open channel hydraulics. The most commonly used methods for determining head losses in pressure piping systems are the *Hazen–Williams equation* and the *Darcy–Weisbach equation* (see Chapter 5). Many of the general friction loss equations can be simplified and revised because of the following assumptions that can be made for a pressure pipe system:

1. Pressure piping is almost always circular, so the flow area, wetted perimeter, and hydraulic radius can be directly related to diameter.
2. Pressure systems flow full (by definition) throughout the length of a given pipe, so the friction slope is constant for a given flow rate. This means that the energy grade line and the hydraulic grade line (HGL) drop linearly in the direction of flow.
3. Because the flow rate and cross-sectional area are constant, the velocity must also be constant. By definition, then, the energy grade line and HGL are parallel, separated by the constant velocity head ($v^2/2g$).

These simplifications allow for pressure pipe networks to be analyzed much more quickly than systems of open channels or partially full gravity piping. Several hydraulic components that are unique to pressure piping systems, such as regulating valves and pumps, add complexity to the analysis.

Pumps are an integral part of many pressure systems and are an important part of modeling head change in a network. Pumps add energy (head gains) to the flow to counteract head losses and hydraulic grade differentials within the system. Several types of pumps are used for various purposes (see Chapter 8); pressurized water systems typically have centrifugal pumps.

To model the behavior of the pump system, additional information is needed to ascertain the actual point at which the pump will be operating. The *system operating point* is the point at which the pump curve crosses the system curve—the curve representing the *static lift* and *head losses* due to friction and *minor losses*. When these curves are superimposed (as in Fig. 7.2), the operating point is easily located.

As water surface elevations and demands throughout the system change, the static head and head losses vary. These changes cause the system curve to move around, whereas the pump characteristic curve remains constant. These shifts in the system curve result in a shifting operating point over time (see Chapter 8).

A centrifugal pump's characteristic curve is fixed for a given motor speed and impeller diameter, but can be determined for any speed and any diameter by applying the *affinity laws*. For variable-speed pumps, these affinity laws are presented in Eq. (7.1):

$$\frac{Q_1}{Q_2} = \frac{n_1}{n_2} \quad \text{and} \quad \frac{H_1}{H_2} = \left(\frac{n_1}{n_2}\right)^2 \tag{7.1}$$

where

Q = pump flow rate, m^3/s (ft^3/s)
H = pump head, m (ft)
n = pump speed, rpm

Thus, *pump discharge rate* is proportional to pump speed, and the *pump discharge head* is proportional to the square of the speed. Using this relationship, once the pump curve is known, the curve at another speed can be predicted. Figure 7.3 illustrates the affinity laws applied to a variable-speed pump. The line labeled "Best Efficiency Point" indicates how the best efficiency point changes at various speeds.

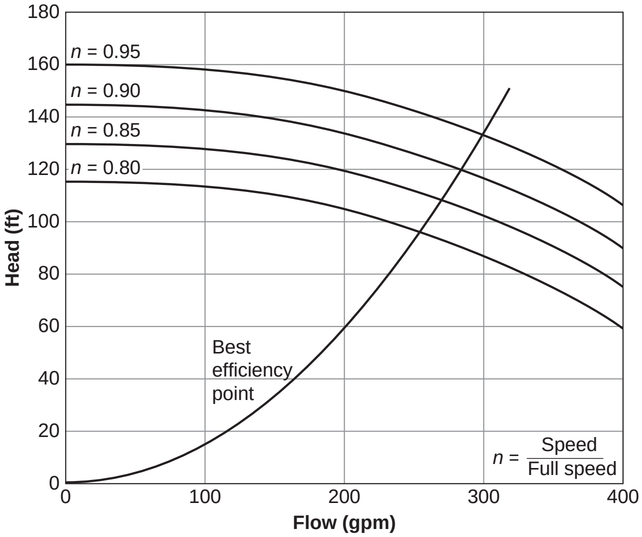

Figure 7.3 Relative speed factors for variable-speed pumps. Conversion factors: 1 gpm = 3.785 L/min; 1 ft = 0.3048 m.

7.4 PIPE NETWORKS

In practice, pipe networks consist not only of pipes, but also of miscellaneous fittings, services, storage tanks, reservoirs, meters, regulating valves, pumps, and electronic and mechanical controls. For modeling purposes, these system elements can be organized into four fundamental categories:

1. *Junction nodes:* Junctions are specific points (nodes) in the system where an event of interest is occurring. Junctions include points where pipes intersect, points where major demands on the system (such as a large industry, a cluster of houses, or a fire hydrant) are located, or critical points in the system where pressures are important for analysis purposes.
2. *Boundary nodes:* Boundaries are nodes in the system where the hydraulic grade is known, and they define the initial hydraulic grades for any computational cycle. They set the HGL used to determine the condition of all other nodes during system operation. Boundary nodes are elements such as tanks, reservoirs, and pressure sources. A model must contain at least one boundary node for the HGLs and pressures to be calculated.
3. *Links:* Links are system components such as pipes that connect to junctions or boundaries and control the flow rates and energy losses (or gains) between nodes.
4. *Pumps and valves:* Pumps and valves are similar to nodes in that they occupy a single point in space, but they also have link properties because head changes occur across them.

An event or condition at one point in the system can affect all other locations in the system. Although this fact complicates the approach that the engineer must take to find a solution, there are some governing principles that drive the behavior of the network, such as the *conservation of mass* and the *conservation of energy*.

7.4.1 Conservation of Mass

The conservation of mass principle is a simple one. At any node in the system under incompressible flow conditions, the total volumetric or mass flow entering must equal the mass flow leaving (plus the change in storage).

Separating the total volumetric flow into flows from connecting pipes, demands, and storage, we obtain the following equation:

$$\sum Q_{\text{in}}\Delta t = \sum Q_{\text{out}}\Delta t + \Delta s \tag{7.2}$$

where

$\sum Q_{\text{in}}$ = total flow into the node
$\sum Q_{\text{out}}$ = total flow out of the node
Δs = change in storage volume
Δt = change in time

7.4.2 Conservation of Energy

The principle of conservation of energy dictates that the head losses through the system must balance at each point (Fig. 7.4). For pressure networks, this means that the total head loss between any two nodes in the system must be the same regardless of the path taken between the two points. The head loss must be “sign consistent” with the assumed flow direction (i.e., head loss occurs in the direction of flow,

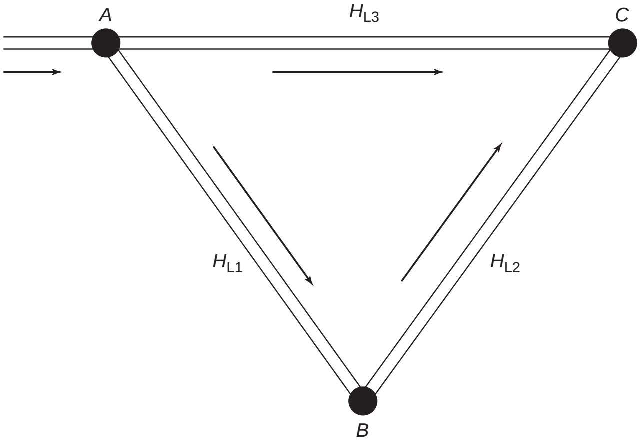

Figure 7.4 Conservation of energy.

and head gain occurs in the direction opposite to that of the flow). For the paths in Fig. 7.4,

$$\text{Path from A to C:} \quad H_{L3} = H_{L1} + H_{L2} \tag{7.3}$$

$$\text{Path from A to B:} \quad H_{L1} = H_{L3} - H_{L2} \tag{7.4}$$

$$\text{Loop from A to A:} \quad 0 = H_{L1} + H_{L2} - H_{L3} \tag{7.5}$$

Although the equality can become more complicated with minor losses and controlling valves, the same basic principle can be applied to any path between two points. As shown in Fig. 7.4, the combined head loss around a loop must equal zero in order to compute the same hydraulic grade for a given point.

7.5 NETWORK ANALYSIS

7.5.1 Steady-State Network Hydraulics

Steady-state analysis is used to determine the operating behavior of a system at a specific point in time or under steady-state conditions. This type of analysis can be useful in discovering the short-term effect of fire flows or average demand conditions on the system.

For this type of analysis, the network equations are determined and solved with tanks being treated as fixed-grade boundaries. The results that are obtained from this type of analysis are instantaneous values, and they may not be representative of the values of the system a few hours—or even a few minutes—later in time.

7.5.2 Extended-Period Simulation

An *extended-period simulation* is used to determine the behavior of the system over time. This type of analysis allows the user to model tanks filling and draining, regulating valves opening and closing, and model pressures and flow rates changing throughout the system in response to varying demand conditions and automatic control strategies formulated by the modeler.

Whereas a steady-state model may tell the user whether the system has the capability to meet a specific demand, an extended-period simulation indicates whether the system has the ability to provide acceptable levels of service over a period of minutes, hours, or days. Extended-period simulations can also be used for energy consumption and cost studies, as well as for water quality modeling.

Data requirements for an extended-period simulation go beyond what is needed for a steady-state analysis. The user must determine water usage patterns, provide more detailed tank information, and enter operational rules for pumps and valves.

7.6 WATER QUALITY MODELING

In the past, water distribution systems were designed and operated with little consideration of water quality, due in part to the difficulty and expense of analyzing a dynamic system. The cost of extensive sampling and the complex interaction between fluids and constituents make *numeric modeling* the ideal method for predicting water quality.

To predict water quality parameters, an assumption is made that there is complete mixing across finite distances, such as at a junction node or in a short segment of pipe. Complete mixing is essentially a mass balance given by

$$C_a = \frac{\sum Q_i C_i}{\sum Q_i} \tag{7.6}$$

where

C_a = average (mixed) constituent concentration
Q_i = inflow rates
C_i = constituent concentrations of the inflows

7.6.1 Age Modeling

Water age provides a general indication of the overall water quality at any given point in the system. Age is typically measured from the time that the water enters the system from a tank or reservoir until it reaches a junction. Along a given link, water age is computed as follows:

$$A_j = A_{j-1} + \frac{x}{v} \tag{7.7}$$

where

A_j = age of water at jth mode
A_{j-1} = age of water at $j - 1$ mode
x = distance from node $j - 1$ to node j
v = velocity from node $j - 1$ to node j

If there are several paths for water to travel to the jth node, the water age is computed as a weighted average using Eq. (7.8):

$$\mathrm{AA}_j = \frac{\sum Q_i \left[\mathrm{AA}_i + \left(\frac{x}{v}\right)_i\right]}{\sum Q_i} \tag{7.8}$$

where AA_j is the average age at the node immediately upstream of node j; AA_i is the average age at the node immediately upstream of node i; x_i is the distance from the ith node to the jth node; v_i is the velocity from the ith node to the jth node; and Q_i is the flow rate from the ith node to the jth node.

7.6.2 Trace Modeling

Identifying the origin of flow at a point in the system is referred to as *flow tracking* or *trace modeling*. In systems that receive water from more than one source, trace studies can be used to determine the percentage of flow from each source at each point in the system. These studies can be very useful in delineating the area influenced by an individual source, observing the degree of mixing of water from several sources, and viewing changes in origins over time.

7.6.3 Constituents Modeling

Reactions can occur within pipes that cause the concentration of substances to change as water travels through the system. Based on *conservation of mass* for a substance within a link (for extended-period simulations only),

$$\frac{\partial c}{\partial t} = v\frac{\partial c}{\partial x} + \theta(c) \tag{7.9}$$

where

c = substance concentration as a function of distance and time
t = time increment
v = velocity
x = distance along the link
$\theta(c)$ = substance rate of reaction within the link

In some applications, there is an additional term for *dispersion*, but this term is usually negligible (*plug flow* is assumed through the system).

Assuming that complete and instantaneous mixing occurs at all junction nodes, additional equations can be written for each junction node with the following conservation of mass equation:

$$C_{k|x=0} = \frac{\sum Q_j C_{j|x=L} + Q_{\mathrm{e}} C_{\mathrm{e}}}{\sum Q_j + Q_{\mathrm{e}}} \tag{7.10}$$

where

C_k = concentration at node k
j = pipe flowing into node k
L = length of pipe j
Q_j = flow in pipe j
C_j = concentration in pipe j
Q_{e} = external source flow into node k
C_{e} = external source concentration into node k

Once the hydraulic model has solved the network, the velocities and the mixing at the nodes are known. Using this information, the water quality behavior can be derived using a numerical method.

7.6.4 Initial Conditions

Just as a hydraulic simulation starts with some amount of water in each storage tank, initial conditions must be set for a water age, trace, or constituent concentration analysis. These initial water quality conditions are usually unknown, so the modeler must estimate these values from field data, a previous water quality model, or some other source of information.

To overcome the problem of unknown initial conditions at the vast majority of locations within the water distribution model, the duration of the analysis must be long enough for the system to reach *equilibrium conditions*. Note that a constant value does not have to be reached for equilibrium to be achieved; rather, equilibrium conditions are reached when a repeating pattern in age, trace, or constituent concentration is established.

Pipes usually reach equilibrium conditions in a short time, but storage tanks are much slower to show a repeating pattern. For this reason, extra care must be taken when setting a tank's initial conditions, in order to ensure the model's accuracy.

7.6.5 Numerical Methods

Several theoretical approaches are available for solving water quality models. These methods can generally be grouped as either *Eulerian* or *Lagrangian* in nature, depending on the volumetric control approach that is taken. Eulerian models divide the system into fixed pipe segments, and then track the changes that occur as water flows through these segments. Lagrangian models also break the system into control volumes, but then track these water volumes as they travel through the system. This chapter presents two alternative approaches for performing water quality constituent analyses.

7.6.6 Discrete Volume Method

The *discrete volume method* (DVM) is an Eulerian approach that divides each pipe into equal *segments* with completely mixed volumes (Fig. 7.5). Reactions are calculated within

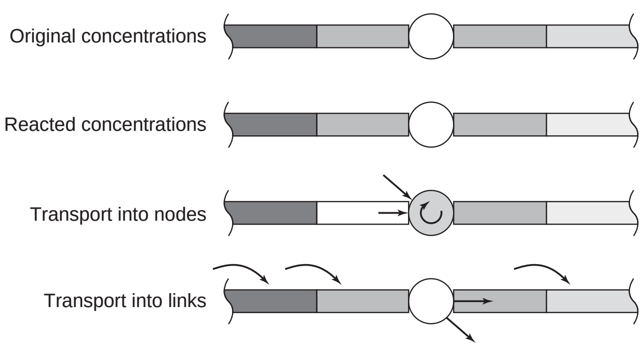

Figure 7.5 Eulerian discrete volume method (DVM).

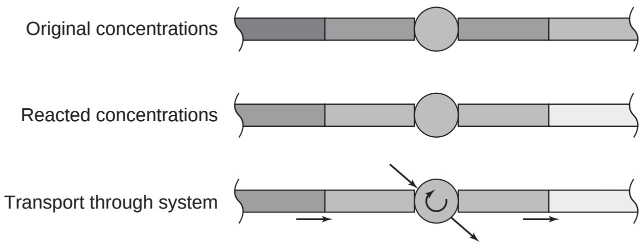

Figure 7.6 Lagrangian time-driven method (TDM).

each segment, and the constituents are then transferred to the adjacent downstream segment. At nodes, mass and flow entering from all connecting pipes are combined (assuming total mixing). The resulting concentration is then transported to all adjacent downstream pipe segments. This process is repeated for each water quality time step until a different hydraulic condition is encountered. When this occurs, the pipes are divided again under the new hydraulic conditions, and the process continues.

7.6.7 Time-Driven Method

The *time-driven method* (TDM) is an example of a Lagrangian approach (Fig. 7.6). This method also breaks the system into *segments*, but rather than using fixed control volumes as in Eulerian methods, the concentration and size of water *parcels* are tracked as they travel through the pipes. With each time step, the farthest upstream parcel of each pipe elongates as water travels into the pipe, and the farthest downstream parcel shortens as water exits the pipe.

Similar to the DVM, the reactions of a constituent within each parcel are calculated, and the mass and flow entering each node are summed to determine the resulting concentration. If the resulting nodal concentration is significantly different from the concentration of a downstream parcel, a new parcel will be created rather than elongating the existing one. These calculations are repeated for each water quality time step until the next hydraulic change is encountered and the procedure begins again.

7.7 AUTOMATED OPTIMIZATION

WaterGEMS has the capability to optimize a model based on *field data* or *design criteria*. Oftentimes, water utility managers will use a model to make design decisions or gather field data to *calibrate a model*. This process is typically a trial-and-error approach in which the modeler will modify a few parameters in a model to either compare design solutions based on cost or benefit or have the model better predict the real conditions. Because this can be very time consuming, WaterGEMS has the capability to create many potential solutions and provide a measure of which solution is the "better" solution based on specific *boundary conditions* and *input criteria*.

WaterGEMS employs a *genetic algorithm* search method to find "better" solutions based on the principles of natural selection and biological reproduction. This genetic algorithm program first creates a population of trial solutions based on modeled parameters. The hydraulic solver then simulates each trial solution to predict the HGL and flow rates within the network and compares them to any input criteria. Based on this comparison, a *goodness-to-fit value* is assigned. This information is now used to create a new population of trial solutions. These solutions are then again used to find new solutions. The program compares these solutions to the specific boundary conditions and input criteria until the goodness-to-fit value is optimized. In other words, comparisons are made until no better solution can be generated.

7.7.1 Model Calibration

Model calibration is the process of modifying parameters or values in a model so it better matches what is happening in the real system. The calibration of water distribution models is very complicated. Many values and parameters that are unknown are needed at any one time to reduce the discrepancy between the model and the real system. Oftentimes the pipe roughness value is adjusted to make the model results match the measured or expected values in the real system. However, many other parameters could influence the modeled results. For example, the water demand at junctions and the status of pipes and valves in the system could also be adjusted when calibrating a model.

Calibration of a model relies on accurate field measurement data. Field measurements of pressures in the system, pipe flow rates, water levels in tanks, valve status, and pump operating status and speed are all used to calibrate models. Critical to all of these measurements is the time for which the measurements are made. The times of these measurements must all be synchronized to the time frame of the model. In addition, because the conditions within a real system change throughout the day or year, field data should be collected for many different conditions and times. The calibration process is used to adjust the model to simulate multiple demand loadings and operational boundary conditions. Only then can the modeler be confident that the model is valid for many different conditions.

WaterGEMS has a module called *Darwin Calibrator* that it uses to assist in optimizing the model to match field measurement data. Darwin Calibrator allows the modeler to input field data, then request the software to determine the optimal solutions for pipe roughness values, junction demands, or status (on/off). Pipes that have the same hydraulic characteristics where one roughness value is assigned to all pipes can be grouped together. Junctions can also be grouped based on the demand pattern and location. Caution should be used when grouping pipes and junctions because this could greatly affect the model's calibration accuracy.

7.7.2 System Design

The goal of water distribution system design is to maximize the benefits of the system while minimizing the cost. The *optimal solution* is a design that meets all the needs of the system at minimal cost. Some planning is needed to account for additional future needs of the system including potential growth of the system in terms of demand and its location. The modeler must work with the system owner and planning groups to account for both the current and future needs.

Another module in WaterGEMS, called *Darwin Designer*, assists engineers with the planning and design of water distribution networks. Darwin Designer can be used to size new pipe and/or rehabilitate old pipes to minimize cost, maximize benefit, or create a scenario for trading off costs and benefits. The least cost optimization is used to determine the pipe material and size needed to satisfy the design requirements. The maximum benefit optimization is used to determine the most beneficial solution based on a known budget. Darwin Designer will generate a number of solutions that meet the design requirements at minimal cost or maximum benefit. In either case, the best solution for new pipe or rehabilitation of old pipe will be based on the following input hydraulic criteria:

- Minimum and maximum allowable pressures
- Minimum and maximum allowable pipe flow velocity
- Additional demand requirements
- Pipe, pump, tank, valve, and so on, status change requirements

Critical to creating an accurately designed system is time and peak demand requirements. The peak demand and fire flow conditions are used to size pipes since the pipe network must work for all conditions. Using average demand values to size pipe without accurately accounting for peaking factors can create networks that are either undersized and will not deliver the required water needs or oversized and much more expensive than need be. The daily and seasonal variations can also greatly affect the final design. Demand variations need to be synchronized in the model to accurately reflect what could happen in the real system.

The following examples give step-by-step instructions on how to solve problems and design water systems using WaterGEMS.

EXAMPLE 7.1 THREE PUMPS IN PARALLEL

Problem Statement

A pump station is designed to supply water to a small linen factory. The factory, at an elevation of 58.0 m, draws from a circular, constant-area tank (T-1) at a base elevation of 90.0 m with a minimum water elevation of 99.0 m, an initial water elevation of 105.5 m, a maximum water elevation of 106.0 m, and a diameter of 10.0 m.

Three main parallel pumps draw water from a source with a water surface elevation of 58.0 m. Two pumps are set aside for everyday usage, and the third is set aside for emergencies. Each pump has a set of controls that ensure it will run only when the water level in the tank reaches a certain level. Use the Hazen–Williams equation to determine friction losses in the system. The network layout is given in Fig. 7.7; the pipe and pump data are given in Tables 7.1 and 7.2, respectively.

Part 1: Can the pumping station support the factory's 20 L/s demand for a 24-h period?

Part 2: If there were a fire at the linen factory that required an additional 108 L/s of water for hours 0 through 6, would the system with the pump controls given in the problem statement be adequate? Supply the extended-period simulation report describing the system at each time step.

Part 3: How might the system be operated so that the fire flow requirement in part 2 is met?

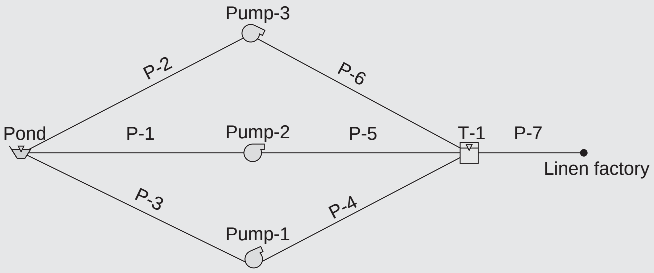

Figure 7.7 Schematic of Example 7.1.

Table 7.1 Pipe information for Example 7.1

Pipe	Length (m)	Diameter (mm)	Material	Roughness
P-1	6	150	Cast iron	90
P-2	6	150	Cast iron	90
P-3	6	150	Cast iron	90
P-4	71	150	Cast iron	90
P-5	72	150	Cast iron	90
P-6	73	150	Cast iron	90
P-7	18	200	Cast iron	90

Table 7.2 Pump information for Example 7.1

		Pump curve		
Pump	Elevation (m)	Flow (L/s)	Head (m)	Controls
		0	78.0	
PUMP-1	58.0	32	58.5	On when T-1 is below 105.5 m
		63	0	Off when T-1 is above 106.0 m
		0	78.0	
PUMP-2	58.0	32	58.5	On when T-1 is below 105.2 m
		63	0	Off when T-1 is above 106.0 m
		0	67.0	
PUMP-3	58.0	32	50.5	On when T-1 is below 99.25 m
		63	0	Off when T-1 is above 103.00 m

Solution to Part 1:

- When you start WaterGEMS, you should be prompted with the **Welcome to WaterGEMS** dialog. From this dialog, you can access the tutorials, open existing projects, and create new ones. Select **Create New Project**.
- If the **Welcome to WaterGEMS** dialog does not appear, WaterGEMS is set to **Hide Welcome Page** on startup. To start a new project, select **New** from the **File** menu. You can change from **Hide Welcome Page** mode to **Show Welcome Page** mode in the **Global Options** dialog, which is accessible by selecting **Options** from the **Tools** menu.
- As always when starting a new project, the file should be saved frequently to avoid losing data or simulations. To save a new project, select **Save As** under the **File** menu. Enter the project title **Tutorial 1** and then, at any time, you can save your project by clicking the **Save** button.
- A more descriptive project title and other general information can be entered into the **Project Properties** box found under the **File** menu.
- Before starting, you should set up the general default settings for the project. You can find the default settings in **Options** under the **Tools** menu. In the **Drawing** tab select **Schematic** from the **Drawing Mode** field. This option will allow you to define the pipe lengths and node locations without having to worry about scale and spatial placement on the *x–y* plane.
- To define the default units, go to the **Units** tab found under **Options** from the **Tools** menu. Select **System International** from the list box in the **Results Default** field if it is not already selected. You can define any of the default label units by clicking the unit field and selecting the desired unit from the list. For example, to change the **Angle** units from radians to degrees, click on **Radians** in the unit field, then select **Degrees** by locating it on the drop-down list of available units.

Laying Out the System

- Begin with the pipeline running horizontally through the center of the system. Because you selected **Schematic** in the **Drawing Mode** field, you do not have to lay out the system exactly as shown in the problem statement. You can roughly sketch the schematic by following the instructions here. You will likely have to rename many of the elements to match the names shown in Fig. 7.7. The steps below will describe how to do this.

- Click the **Pipe Layout** button on the vertical toolbar on the left side of the layout screen.
- Move the cursor to the layout screen and right-click the mouse. Select **Reservoir**. To place the reservoir, simply click the left mouse button.
- Move your mouse horizontally to the right to place a pump. Right-click and select **Pump** from the pop-up menu, then left-click to place the pump.
- Repeat the process for the tank by selecting **Tank** from the pop-up menu.
- Now, place the junction node "Linen Factory." After placing the junction, right-click and select **Done**.
- Continue by entering the remaining two pumps and four pipes in the same way as described previously. To connect a pipe to an object on the layout screen, click the object while in the pipe layout mode. The object should turn red when it is selected.
- Except for the scale, your schematic should look roughly like the one given in the problem statement.
- To exit the pipe layout mode, click the arrow button on the vertical toolbar on the left side of the layout screen.

Entering the Data

- Double-click the reservoir node to open its dialog editor. Change the name to "Pond" in the **Label** field. Enter 58 m in the **Elevation** field. Close the dialog editor.
- Double-click the tank. Enter the given diameter for the circular section and the appropriate elevations from the problem statement. Disregard the inactive volume field. Be sure that **Elevation** is selected in the **Operating Range Type** field. Close the dialog editor.
- Double-click the bottom pump. Change the name to "PUMP-1" in the **Label** field. Enter the appropriate elevation from the pump data table (Table 7.2) into the **Elevation** field. Click the **Pump Definition** field and select **Edit Pump Definitions** to open the Pump Definitions dialog. Add a new pump definition and label it "Pumps 1 and 2." In the **Head** tab select **Standard (3 Point)**. Enter the pump curve data given for PUMP-l. If you need to change the units, right-click on the Flow or Head table headings and open the "Units and Formatting" dialog. Click **Close** to close the Pump Definition dialog. Now select "Pumps 1 and 2" in the **Pump Definitions** field. Close the dialog editor.
- Repeat the above process for the other pumps. When entering the data for PUMP-3, you will have to create a new pump definition titled "Pump 3" for the **Pump Definitions** field.
- Next, enter the pump controls given in the problem statement. Click **Controls** in the **Components** menu.
- Select the **Conditions** tab to enter the five Tank conditions as described from the problem statement information. Enter each condition as **New** and **Simple**. The **Condition Type** is **Element**; select the Tank from the layout screen by clicking the ellipse button; select **Hydraulic Grade** as the **Tank Attribute**; the **Operator** and **Hydraulic Grade** are entered based on the problem statement information.
- Select the **Actions** tab to enter whether the pump is on or off. The default setting is generally with the pumps on. Enter the six actions (each pump either on or off) as **New** and **Simple**. For example, to turn off PUMP-1, the **Element** is entered by clicking the ellipse button and selecting PUMP-1 from the layout screen; the **Pump Attribute** would be Pump Status; the **Operator** would be the default " = "; then select **Off** for the **Pump Status**.
- Select the **Controls** tab to enter the six controls. The controls are all Simple and entered as If Then statements. For example, click **New** then the **Evaluate as Simple Control** box; in the IF Condition field, select {"Tank" level > 106.00 m}; in the THEN Action field, select {"PUMP-1" pump status = off}. Close the **Controls** dialog.
- Double-click the junction node. Change the name to "Linen Factory." Enter 58 m in the **Elevation** field. Click the **Demand Collection** field to enter a fixed demand of 20 L/s after clicking the ellipse button. Close both dialog editors.
- For the pipes, you can edit the data as you have been by clicking each element individually, and then entering the appropriate data. However, this method can be time consuming, especially as the number of pipe elements increases. It is often easier to edit the data in a tabular format.
- Click the **Flex Tables** button in the toolbar at the top of the screen. Select **Pipe Table** from the available tables.
- The fields highlighted in the **Pipe Table** are output fields. The fields in white are input fields and can be edited as you would edit data in a spreadsheet.
- ***Warning:*** The pipes may not be listed in the table in numerical order. You may want to sort the pipe labels in ascending order. To do this, move the cursor to the top of the table and place it on the **Label** column. Right-click, select **Sort,** and then select **Sort Ascending**. The pipes should then be listed in numerical order.
- Enter the correct pipe lengths into the **Length (User Defined)** column found on the **Pipe Table**. Also enter the pipe diameters and Hazen–Williams C value. Close the Pipe Table.
- *Note:* You can customize which columns appear in the **Pipe Table** by clicking the **Edit** button in the toolbar at the top of the table. Table columns can then be added or removed as desired.

Running the Model

- To run the model, first click the **Compute** button on the main toolbar. Arrows should appear on your layout screen indicating the flow direction in each pipe. If you click on any of the objects, you will see the results in the dialog. You could look at the results for all similar objects by opening the **Flex Tables** button. For example, if you want to look at flows in all the pipes, select the **Pipe Table**. To examine the flow through the system over a 24-h period, select the **Calculation Options** under the **Analysis** menu. Double-click the **Base Calculation Options**, then in the **Time Analysis Field** select **EPS**. Set the start time to 12:00:00 a.m. and the duration to 24 h. The **Hydraulic Time Step** of 1 h will provide sufficient output for the purpose of this tutorial. Click the **Compute** button.
- The software provides a couple of different ways to determine whether your model meets the target demand: scroll through the Calculation Summary and check to see if there are any disconnected node warnings. When the level in tank T-1 drops to the minimum tank elevation of 99 m (tank level of 9 m), the tank closes off, preventing any more water from leaving. This closure will cause the linen factory to be disconnected from the rest of the system (i.e., it will not get the required 20 L/s).

  OR

  Close the Calculation Summary window and select the Linen Factory junction. To create a graph of the pressure at this node, click the **Graphs** button in the main toolbar. Create a **Line-Series Graph** from the **New** button in the Graphs dialog. Select the **Pressure** box in the **Graph Series Options** window, then close the options window. You should see the calculated pressure at the Linen Factory and notice that it never reached zero (no water pressure).

Answer

As you will see for this problem, all the pressures at the linen factory hover around 465 kPa, and no disconnected nodes are detected. Therefore, the pumping station can support the factory’s 20 L/s demand for a 24-h period.

Solution to Part 2:

- Add another demand to the Linen Factory node. To do this, double-click the Linen Factory junction and enter into the **Demand Collection** field a second fixed demand of 108 L/s in the row below the 20 L/s demand after clicking the **Ellipse** button. Close the dialog editor.
- Select the **Calculation Options** under the **Analysis** menu, and then double-click the **Base Calculation Options**. You only need to run this model for 6 h, so change 24 to 6 in the **Duration** field. Click **Compute** to run the model.
- As you scroll through the results, you will see warning messages (yellow or red indicators instead of green) indicating a disconnected node at the linen factory after 3 h. Close the **User Notifications** window. Select the tank and create a Line-Series Graph of the water level in the tank by selecting **Level (Calculated)** in the Graph Series Options window. The graph indicates that the water level in the tank reaches the minimum level of 9 m at 3:37:30 and cannot supply water to the linen factory.

Answer

If there were a fire at the factory, the existing system would **not** be adequate.

Solution to Part 3:

Answer

PUMP-3 could be manually switched on at the beginning of the fire to supply the flow necessary to fight the fire at the linen factory. To do this, delete the pump controls for PUMP-3. Then PUMP-3 will be always on during the model simulation.

EXAMPLE 7.2 WATER QUALITY

This example demonstrates the use of WaterGEMS to simulate water quality in a water distribution system. The Scenario Manager module is used to facilitate different types of analysis on the same network (within the same project file).

Problem Statement

A local water company is concerned with the water quality in its distribution network. The company wishes to determine the age and chlorine concentration of the water as it exits the system at different junctions. The water surface at the reservoir is 70 m.

Chlorine is injected into the system at the source of flow, R-l (see Fig. 7.8), at a concentration of 1 mg/L. It has been determined through a series of bottle tests that the average bulk reaction rate of the chlorine in the system (including all pipes and tanks) is approximately 30.5 per day.

The network model may be entered in WaterGEMS using the layout in Fig. 7.8 and the data in Tables 7.3, 7.4, 7.5, and 7.6.

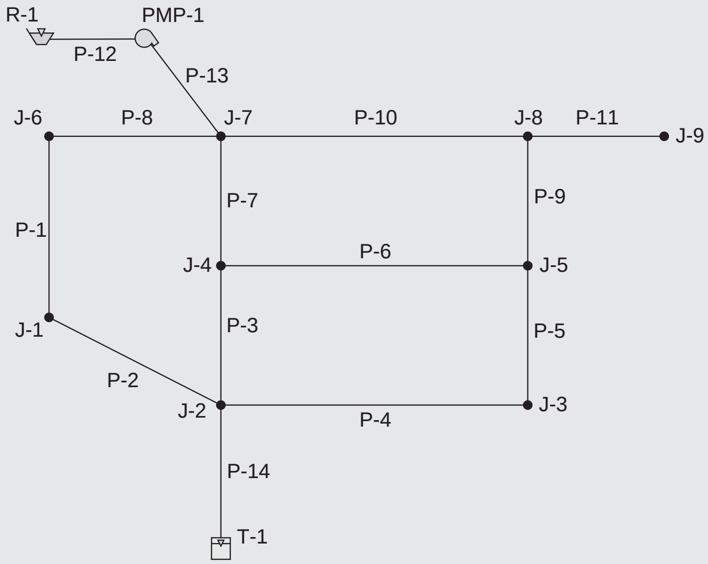

Figure 7.8 Schematic for Example 7.2.

Table 7.3 Continuous demand pattern data for Example 7.2

Time from		Time from	
Start (h)	Multiplier	Start (h)	Multiplier
Start	0.80	*e*	1.30
1	0.60	14	1.40
2	0.50	15	1.50
3	0.50	16	1.60
4	0.55	17	1.80
5	0.60	18	1.80
6	0.80	19	1.40
7	1.10	20	1.20
8	1.50	21	1.00
9	1.40	22	0.90
10	1.30	23	0.80
11	1.40	24	0.80
12	1.40		

Table 7.4 Pump information for Example 7.2

Flow (L/min)	Head (m)	Controls
0	40	Off of node T-1 above 103.5 m On if node T-1 below 99.5 m
3,000	35	
6,000	24	

Table 7.5 Pipe data for Example 7.2

Pipe	Length (m)	Diameter (mm)	Roughness
P-1	300	200	130
P-2	305	200	130
P-3	225	200	130
P-4	301	200	130
P-5	225	200	130
P-6	301	200	130
P-7	225	200	130
P-8	301	200	130
P-9	200	200	130
P-10	301	200	130
P-11	300	200	130
P-12	1	250	130
P-13	3,000	300	130
P-14	300	300	130

Table 7.6 Junction data for Example 7.2

Junction	Elevation (m)	Demand (L/min)
J-1	73	151
J-2	67	227
J-3	85	229
J-4	61	212
J-5	82	208
J-6	56	219
J-7	67	215
J-8	73	219
J-9	55	215

The tank is circular with a diameter of 15.0 m. The minimum elevation is 99.0 m. The maximum elevation is 104.0 m, and the initial elevation is 103.4 m. The base elevation is 98.0 m, and the inactive volume is 10.0 m^3. The elevation of the pump is 70.0 m and it is initially on.

Part 1: Perform an age analysis on the system using a duration of 7 days and a time step of 1 h. Determine the youngest and oldest water in the distribution system and the storage tank. Explain why water age varies.

Part 2: Perform a constituent analysis using the same duration and time step as in part 1. Determine the range of concentrations in the system and the storage tank. Explain the behavior of the system with regard to chlorine.

Part 3: Are the simulation results consistent with the known behavior of chlorine?

Part 4: Why is it necessary to run the model for such a long period of time? Do you feel 7 days is too long or too short a time period to test the model? Why?

Solution to Parts 1–4:

- Use the same steps as in Example 7.1 to set up the project, lay out the system, and enter the data. Be sure to set units to **System International** and the drawing mode to **Schematic**. Again, you will likely have to rename many of the elements after you draw the general layout to make sure data are correctly entered for each element.
- The demand pattern data can be entered by selecting **Patterns** under the **Components** menu. Right-click **Hydraulic** to select **New**. The Start Time is **12:00:00 AM**, the Starting Multiplier is **0.80**, and the Pattern Format is **Continuous**. Enter the data from the problem statement table under the **Hourly** tab.
- The demand pattern can be assigned to a selected junction by clicking a junction and entering **Hydraulic Pattern - 1** into the **Demand Collection** field, or the pattern can be assigned to all junctions as a Global Edit. In this case, assign the demand pattern to all junctions by selecting **Demand Control Center** under the **Tools** menu. Click **Yes** to continue. On the **Junctions** tab, right-click the **Pattern (Demand)** table heading to select **Global Edit**. Select **Hydraulic Pattern - 1** in the **Value:** field. Click **OK** and close the Demand Control Center dialog.

Base Scenario

- Run the model for a 24-h period by selecting the **Calculation Options** under the **Analysis** menu. Double-click the **Base Calculation Options,** then in the **Time Analysis Type** select **EPS**. The **Duration (hours)** is 24 h and the **Hydraulic Time Step (hours)** is 1.0 h. Close the Base Calculation Options dialog.
- At the bottom of the Calculation Options window are more tabs. Click on the **Scenarios** tab. Notice that the **Compute** button is in the **Scenario** window toolbar. Click the **Compute** button. WaterGEMS calculates the system parameters for a 24-h simulation period. Details of the calculation can be viewed on the **Calculation Summary** window. Close the **Calculation Summary** and **Scenarios** windows.
- Click on the tank, then the **Graphs** button in the main toolbar. Create a **Line-Series Graph** from the **New** button in the Graphs dialog. Select the **Percent Full** box under "Results" under the **Fields** field in the **Graph Series Options** window. Close the **Graph Series Options** window. Size the Graph window to fit on the layout screen such that you can see most of the layout and the toolbar in the **Graph** window to click the **Play** button. Play the 24-h simulation by clicking the **Play** button in the **Graph** window. The flow in the pipes is indicated by the arrows. Note that that there is no flow in pipes P-12 and P-13 when the pump is not operating. The flow direction reverses in pipes P-1, P-2, P-3, P-4, P-5, P-7, P-8, P-9, and P-14 over the 24-h period. The volume of the water in tank T-1 is indicated by the **Percent Full (%)** on the *y*-axis of the graph.

Age Analysis

- The analysis of the age of water within the network may be performed by defining and running an age analysis scenario. From the **Analysis** menu, select **Scenarios**.
- Create a new **Base Scenario** by clicking the **New** button. Enter "Age Analysis" as the name of the scenario.
- Click on the **Calculation Options** tab at the bottom of the window. Create a new Calculation Option by clicking the **New** button and enter "Age Analysis Calculation Options" as the name. Double-click the calculation options you just created and select **Age** in the **Calculation Type** field. The **Duration** is 168 h (7 days), and the **Hydraulic Time Step** is 1.00 h.
- Go back to the **Scenarios** tab, right-click the Age Analysis scenario, and select **Make Current**. The red check should now be on the Age Analysis scenario. Double-click on the Age Analysis scenario and select the Age Analysis Calculation Options in the **Calculation Options** field.
- Go back to the Scenarios tab and click the **Compute** button.
- Close the **Calculation Summary** window and **Scenarios** dialog to view the layout screen.

Results

The oldest water in the network will be found in tank T-1. Click on the tank then the **Graphs** button in the main toolbar. Create a **Line-Series Graph** from the **New** button in the Graphs dialog. Select the **Age Analysis** box in the **Scenarios** field and **Age (Calculated)** found under "Results (Water Quality)" in the **Fields** field in the **Graph Series Options** window. You should also unclick any other selected lines in the **Fields** field. Close the options window. The resulting graph is shown in Fig. 7.9.

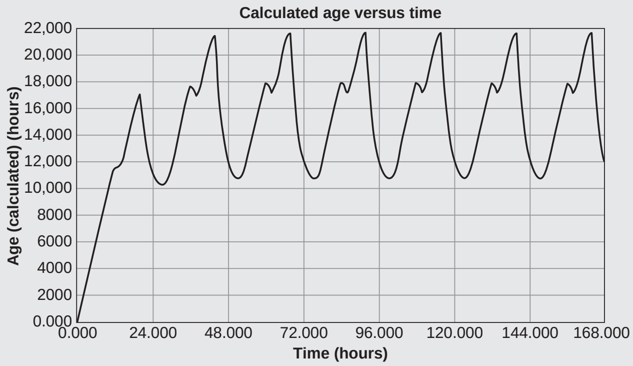

Figure 7.9 Age of water in tank T-1 for Example 7.2.

Note that the water distribution network reaches dynamic equilibrium after 2 days of the simulation. After 48 h, the maximum age at T-1 is approximately 21.5 h, and the minimum age is approximately 10.5 h.

To view the variation in age in the network, click on the tank, J-2, J-3, J-7, and J-9 while holding the shift on the keyboard to select each object. Then click the **Graphs** button in the main toolbar.

Create a **Line-Series Graph** from the **New** button in the Graphs dialog. Select the **Age Analysis** box in the **Scenarios** field and **Calculated Age**, both under Tank and Junction in the **Fields** field in the **Graph Series Options** window.

Close the options window to look at the graph. Notice that the water age in the junctions is much less (2–4 h) than the water in the tank while the pump is on and feeding freshwater into the system. Then the water age in the junctions greatly increases when the system is fed by the tank water after the pump turns off.

Water Quality Analysis

To analyze the behavior of chlorine in the network, the properties of chlorine must be defined in the engineering library.

- From the Components menu, select Engineering Libraries. Double-click the Constituent Libraries. Then right-click on the ConstituentLibrary.xlm to select Add Item.
- Rename the new constituent by right-clicking it and selecting Rename. Change the label to “Chlorine.” Click on the Chlorine dialog. Enter the Diffusivity (l.l22e–010 m^2/s). Enter the Bulk Reaction Order as 1 and the Bulk Reaction Rate as –0.5 (mg/L)$^{1-n}$/day. Because $n = 1$, the units of the rate constant are day^{-1}. Close the Engineering Libraries.
- From the Analysis menu, select Scenarios. Create a new base scenario named “Chlorine Analysis.”
- Click on the Calculation Options tab at the bottom of the window. Create a new Calculation Option by clicking the New button and enter “Chlorine Analysis Calculation Options” as the name. Double-click the calculation options you just created and select Constituent in the Calculation Type field. The Duration is 168 h (7 days), and the Hydraulic Time Step is 1.00 h.
- Go back to the Scenarios tab and right-click the Chlorine Analysis scenario and select Make Current. The red check should now be on the Chlorine Analysis scenario. Double-click on the Chlorine Analysis scenario and select the Chlorine Analysis Calculation Options in the Calculation Options field.
- Go to the Alternatives tab. Double-click on the Constituent; right-click on Base Constituent Alternative to select Open. Select the Constituent System Data tab then click the ellipse button. Click the Synchronization Options button to select Import from Library. Select Chlorine from the Constituent Libraries list. Close the Constituents dialog. Select Chlorine in the Constituent field on the Constituents: Base Constituents Alternative window. Close the Constituent Alternative window.
- Double-click the reservoir to define the loading of chlorine. Select True in the Is Constituent Source? field. The Constituent Source Type is Concentration and the baseline concentration is 1.0 mg/L. The constituent source pattern is fixed.
- The bulk reaction rate in the pipes can be adjusted using the Tables tool. Click the Flex Table button, then select the Pipe Table. Add the Bulk Reaction Rate (Local) and Specify Local Bulk Reaction Rate? to the table by clicking the Edit button in the toolbar at the top of the table. Scroll to the Specify Local Bulk Reaction Rate? column to click the box for the pipe you want to adjust. Now you can enter a reaction value for the pipe in the Bulk Reaction Rate (local) column. In this case we will not change any of the default values so close the Pipe Table.
- Double-click the tank. Set the initial chlorine concentration to 0.000 mg/L, select True in the Specify Local Bulk Rate? field, then enter the bulk reaction rate of –0.5 per day. Close the editor dialog.
- From the Analysis menu, select Scenarios. Make sure Chlorine Analysis is the current scenario then run the scenario by clicking the Compute button in the top row.
- Open the EPS Results Browser window by clicking the icon button. To create a contour map of the chlorine concentration, click the Contour button. After clicking the New button, contour by Concentration (Calculated); select all elements; set the Minimum to 0.0, Maximum to 1.0, Increment to 0.025, and Index to 0.1 mg/L. Click the Play button on the Animation Control window. The chlorine concentrations for each time step can be viewed through time as shown in Fig. 7.10.
- Save your simulation as “Tutorial 2” since this network will be used in the following tutorials.

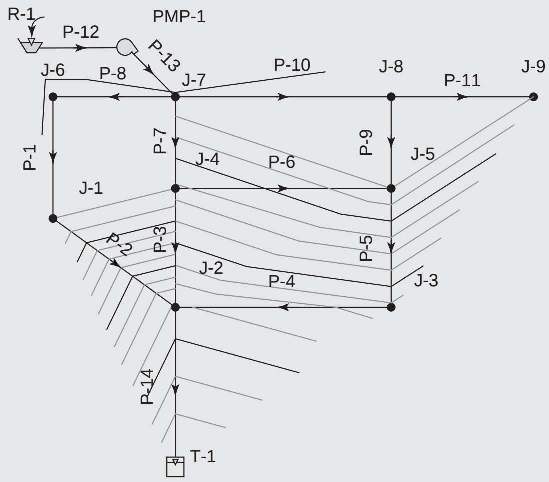

Figure 7.10 Contours of chlorine concentration at 42 h for Example 7.2.

Results

To view the variation of the chlorine concentration in the network, click on the tank, J-2, J-3, J-7, and J-9 while holding the shift on the keyboard to select each object. Then click the **Graphs** button in the main toolbar. Create a **Line-Series Graph** from the **New** button in the Graphs dialog. Select the **Chlorine Analysis** box in the **Scenarios** field, and **Concentration (Calculated)** found under "Results (Water Quality)" for both the Tank and Junction in the **Fields** field in the **Graph Series Options** window. Close the options window to look at the graph, which should look like Fig. 7.11. The lowest chlorine concentration is found in tank T-1. Junctions J-2 and J-3 each have similar chlorine concentration values. In addition, the water distribution network reaches dynamic equilibrium during the second day of the simulation. After dynamic equilibrium in achieved, the maximum chlorine concentration at tank T-l is 0.799 mg/L, and the minimum concentration is 0.687 mg/L.

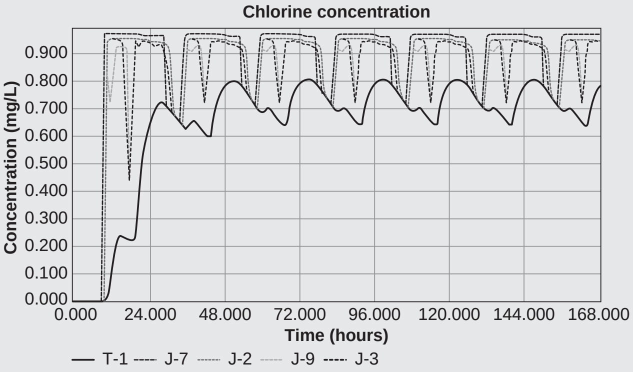

Figure 7.11 Chlorine concentration in tank T-1 and selection junctions of Example 7.2.

To compare age against chlorine concentration at a selected junction, open the graph that plotted the **Calculated Age** for the tank and junctions. The graphs of age versus time and chlorine concentration should now be open on the desktop. Move the graphs so that both are visible and the axes are aligned. Comparison of the two graphs suggests an inverse correlation between age and chlorine concentration.

Answers to Parts 1–4:

Part 1: The oldest water is found in the storage tank. It is far from the source, and incoming water is mixed with the tank's contents. In the distribution system, the oldest water is found at J-9. The newest water is found at J-7 when pump PMP-1 is running.

Part 2: The lowest chlorine concentration is in tank T-1 when it is nearly empty. In the distribution system, the lowest chlorine concentration is found at J-3 when T-1 is emptying. The highest concentration is at J-7 when pump PMP-1 is running.

Part 3: These results are consistent with the fact that chlorine concentration declines over time. For example, during the third day of operation, the minimum chlorine concentration at Junction J-3 is coincident with the maximum age, and the maximum chlorine concentration is coincident with the minimum age.

Part 4: Inspection of the graph of chlorine concentration in tank T-1 suggests that the system stabilizes into a daily pattern on the second day. However, if the initial tank level or the demands are changed, stabilization may take longer. It appears that the 7-day simulation period is adequate for this network.

EXAMPLE 7.3 PUMPING COSTS

This example demonstrates the use of WaterGEMS to calculate the energy costs associated with pumping. Calculate the daily electrical costs for the network in Example 7.2 using the following data:

Energy price	USD 0.10/kWh
Motor efficiency	90%
Pump efficiency	50% at 2,000 L/min
	60% at 2,500 L/min
	55% at 3,000 L/min

Solution:

- Open Tutorial 2 (from Example 7.2).
- The first step is to add the pump and motor efficiency data to PMP-l. On the layout view, double-click **PMP-1.** In the **Pump Definition** field, select **Edit Pump Definitions**. The pump definition dialog for PMP-1 appears.
- On the **Efficiency** tab, select the **Multiple Efficiency Points** option for the pump efficiency. In the **Efficiency Points** table, add the efficiency data from the problem statement.
- In the **Motor** section, enter 90% for the motor efficiency.
- Close the Pump Definition tool and the pump dialog.
- The Energy Cost tool is used to calculate energy costs. From the **Analysis** drop-down menu, select **Energy Costs** or click the icon button in the toolbar.
- Click the icon button to open the **Energy Pricing** field dialog.
- Create a **New** label “Energy Pricing-1” to enter the electricity cost. Enter one line in the cost table. The Time from Start is 0.000, and the Energy Price is $0.10/kWh. Close the **Energy Pricing** dialog to return to the **Energy Cost** window.
- Set the scenario to **Chlorine Analysis** then select “Energy Pricing-1” in the **Energy Pricing** field located on the “Pumps” tab, then click the **Compute** button.

Answer

On the left panel of the **Energy Cost** window, highlight the **Chlorine Analysis** line. On the right panel, select the **Summary** tab. For the 7-day simulation, the following data were calculated:

Pump energy used	4,030 kWh
Volume pumped	21,220 m^3
Pump cost	USD 403.00
Daily cost	USD 58.60

EXAMPLE 7.4 PIPE SIZING USING DARWIN DESIGNER

WaterGEMS can help size pipes and prepare project cost estimates. In this example, the Darwin Designer with the Minimum Cost function is demonstrated.

Problem Statement

Prepare a minimal cost estimate for the pipe materials and installation portion of the project in Example 7.2. The system pipes should be sized using a demand multiplier of 3.4 (peak flow factor) with a calculated pressure for each junction between 170 and 550 kPa. In addition, the system should supply to an industry located at junction 9 an additional 1500 L/min with a minimum pressure of 275 kPa. Use the cost data shown in Table 7.7.

Table 7.7 Pipe material and cost for Example 7.4

	Pipe material and cost	
Material	Diameter (mm)	Cost ($/m)
Ductile iron	75	40.32
Ductile iron	150	56.64
Ductile iron	200	79.36
Ductile iron	250	114.72
Ductile iron	300	156.16
Ductile iron	350	201.92

Do not consider the cost of the reservoir, tank, pump, or pipes P-12 and P-13.

Solution:

- Open Tutorial 2 (from Example 7.2).
- From the **Analysis** menu, select **Scenarios**. Create a new base scenario named "Designer Analysis."
- Click on the **Calculation Options** tab at the bottom of the window. Create a new Calculation Option by clicking the **New** button and enter "Designer Calculation Options" as the name. Double-click the calculation options you just created. The **Time Analysis Type** is **Steady State**.
- Go back to the **Scenarios** tab, right-click the Designer Analysis scenario, and select **Make Current**. The red check should now be on the Designer Analysis scenario. Double-click on the **Designer Analysis scenario** and select the Designer Calculation Options in the **Calculation Options** field. Click the **Compute** button.
- Click the **Darwin Designer** button or find it under the **Analysis** menu to create a scenario to determine the minimum design cost. Click the **New** button to select **New Designer Study**. Check to make sure the **Scenario** window has **Designer Analysis** selected.
- Enter the design criteria on the **Design Events** tab in the dialog by clicking the **New** button. The top window is used to enter the general design criteria; the bottom window is for entering specific design criteria for any elements (pipe, junction, tank, etc.). Pipe networks are typically designed for high flow conditions. Scroll across to set the **Demand Multiplier** to 3.4. Criteria are set to maintain a working network to avoid low-flow or low-pressure conditions: the **Minimum Pressure (Global)** to 170 kPa, and **Maximum Pressure (Global)** to 550 kPa. The flow velocity criteria will be the default settings (0–2.44 m/s).
- Enter the design criteria for the industry at junction 9 using the bottom window. In the **Demand Adjustment** tab, use the button to select junction 9 from the drawing by clicking on the J-9 junction, then the green check in the **Select** dialog. Enter 1500 L/min into the **Additional Demand** window. In the **Pressure Constraints** tab, again select J-9 from the drawing, then click the box in the **Override Defaults?** window, then enter 275 and 550 kPa, respectively, for the minimum and maximum pressures.
- Now the software is told which pipes are to be designed. Pipes with similar properties can be grouped together and will be designed the same, or the software can analyze each pipe separately. In this case, the software will analyze each pipe separately. From the problem statement, all pipes will be considered except P-12 and P-13, which are the pipes from the reservoir and pump. On the **Design Groups** tab click the button to select all the pipes. A table with all the pipes should appear, but if it does not, highlight "<All Available>" in the **Selection Set** window, then click **OK**. In the table, delete pipes P-12 and P-13 to remove them from the analysis.
- The pipe material, properties, and costs to be used in this design scenario are entered in the **Cost/Properties** tab. Open the new pipe table by highlighting **New Pipe** in the window, then select **Design Option Groups** under the New button. Rename "New Pipe-1" to "New Ductile Iron Pipe." Enter the pipe type, diameter, and cost per linear meter from the table in the problem statement. The Hazen–Williams C value is 130 for ductile-iron pipe. To change the units for the pipe cost, right-click the column heading **Unit Cost**, then select **Units and Formatting**. In the Unit field, select **$/m**, then click **OK**.
- The objective of this scenario is to size the pipes to deliver the required flow while maintaining reasonable pressures throughout the network at the minimal design cost. To do this, select the **Minimize Cost** criteria in the **Objective Type** window located in the **Design Type** tab.
- After the design criteria have been entered, you can start the simulation by right-clicking on the **New Design Study-1** in the left window to select **New Optimized Design Run**. Now you could perform many different types of design runs by selecting different design events or design groups to be analyzed. Because we have only one design run in this demonstration, you will not have to compare different potential design solutions. In many cases, different solutions will need to be compared to evaluate which would be best for a specific case. The left window helps organize these different solutions.
- On the **Design Events** tab, you can select the design criteria to be evaluated. In this case there is only one choice.
- On the **Design Groups** tab, select the ductile-iron properties and cost that were entered. The data is entered in the **Design Option Group** column. Because all designed pipe will come from the same ductile-iron table, you can enter the data as a global edit. Right-click the **Design Option Group** field and select **Global Edit**. Select the "New Ductile Iron Pipe" in the **Value** window. Click **OK**; all of the Design Option Group fields should automatically fill in.
- The **Options** tab allows the Darwin Designer parameters to be adjusted. In this case the default values will be used.
- To start the run, click the **Compute** button in the Darwin Designer toolbar. When the run is completed, close the Designing… window. The top three solutions will be listed under the "New Optimized Design Run" in the left window.

Answer

A summary table of the three solutions is shown by clicking the **Solutions** folder. In this example, the minimal pipe cost is USD 217,382. A summary of each solution cost and the design pipe diameters for Solution 1 are shown in Tables 7.8 and 7.9, respectively. The pipe diameters range from 75 to 250 mm and the cost for each pipe is determined based on the expected pipe length. Opening Solution 1 and selecting the **Simulated Results** tab, the calculated pressure at the industry (J-9) is 275.48 kPa, just within the required range.

Table 7.8 Darwin Designer solutions for Example 7.4

Darwin Designer solutions	
Solution	Total cost ($)
Solution 1	217,382
Solution 2	218,687
Solution 3	220,548

Table 7.9 Pipe diameters and costs for Solution 1 of Example 7.4

Pipe diameters and costs for Solution 1		
Pipe	Diameter (mm)	Cost ($)
P-3	250	25,812
P-10	200	23,887
P-6	200	23,887
P-14	200	23,808
P-11	200	23,808
P-8	150	17,049
P-4	150	17,049
P-9	200	15,872
P-7	150	12,744
P-2	75	12,298
P-1	75	12,096
P-5	75	9,072

Note also that this solution uses a number of 75 mm (3 in.) pipes and that the pipe connecting the network to the tank (P-14) is only a 200 mm (8 in.) pipe. If this solution was evaluated for fire flow conditions, it is likely that these pipes would not deliver the required fire flow. Further simulations should be conducted on this solution to ensure that this design can deliver the required flow during a fire event.

EXAMPLE 7.5 MODEL CALIBRATION USING DARWIN CALIBRATOR

WaterGEMS has the ability to use measured field data to calibrate a model. In many cases, data that are entered into the model are an approximation or guess. When the model results do not match field data, then parameters in the model are adjusted. Also, using the Darwin Calibrator and field data, we can locate potential differences between the real network and the model that could be caused by problems in the system (blockages, closed valves, etc.).

Adjust the Hazen–Williams *C* factor (roughness factor) for pipes P-2, P-1, and P-8 for the pipe network from Example 7.2. During the field measurements, the tank level was 3.93 m, the pump was off, and a hydrant with a measured flow of 3400 L/min at junction 7 (J-7) was opened to increase the head loss in the pipe network. The measured field data are shown below:

Junction	Pressure (kPa)
J-7	296.0
J-6	406.5
J-1	263.5
J-2	327.0

Solution:

- Open Tutorial 2 (from Example 7.2).
- From the **Analysis** menu, select **Scenarios**. Create a new base scenario named “Calibrator Analysis.”
- Click on the **Calculation Options** tab at the bottom of the window. Create a new Calculation Option by clicking the **New** button and enter “Calibrator Calculation Options” as the name. Double-click the calculation options you just created. The **Time Analysis Type** is **Steady State**.
- Go back to the **Scenarios** tab and right-click the **Calibrator Analysis** scenario and select **Make Current**. The red check should now be on the Calibrator Analysis scenario. Double-click on the **Calibrator Analysis** scenario and select the Calibrator Calculation Options in the **Calculation Options** field. Click the **Compute** button.
- Click the **Darwin Calibrator** button to create a calibration study to determine the pipe roughness factors. Click the **New** button to select **New Calibration Study**. Check to make sure the **Representative Scenario** window has Calibrator Analysis selected.

- Enter the field data in the **Field Data Snapshots** tab. Click the **New** button to enter new data. Enter the measured pressures on the **Observed Target** tab. Select the junctions from the drawing by clicking the **Select** button then clicking junctions J-7 J-6, J-l, and J-2 from the drawing. Click the green check in the **Select** dialog. Select Pressure (kPa) for each junction in the **Attribute** field. Enter the measured pressure for each junction in the **Value** field.
- Enter the tank level in the **Boundary Overrides** tab. Select the tank from the drawing by clicking the **Select** button then clicking the tank. Click the green check in the **Select** dialog. Select **Tank Level (m)** in the **Attribute** field, then enter 3.93 m in the **Value** field.
- To make sure the pump is off, click the **New** button in the **Boundary Overrides** tab. Select the pump by clicking the **Select** button, then clicking the pump. Click the green check in the **Select** dialog. Select **Pump Status** in the **Attribute** field, then select **Off** in the **Value** field.
- Enter the additional demand at junction 7 in the **Demand Adjustments** tab. Select this junction from the drawing by clicking the **Select** button, then clicking **J-7**. Click the green check in the **Select** dialog. Enter 3400 L/min in the **Value** field.
- Select the pipes where the roughness values are to be determined in the **Roughness Groups** tab. Pipes with similar roughness can be grouped together, or the software can analyze each pipe separately. In this case, the software will determine the pipe roughness values for P-2, P-l, and P-8 separately, and pipes P-3 and P-7 will be grouped since we assume they are both 200 mm (8 in.) ductile-iron pipes, and we do not have any field measurements isolating these pipes. Click the **New** button, then the **Ellipse** button in the **Elements** field. Click the select button, and then pipe P-2 from the drawing. Click the green check in the **Select** dialog. Click **OK** to enter this pipe into the table. Repeat these steps to add pipes P-1 and P-8 to the table. You should have three defined roughness groups in the table on the **Roughness Groups** tab.
- Add the grouped pipes by clicking the **New** button, then the **Ellipse** button in the **Elements** field. Click the select button, then pipes P-3 and P-7 from the drawing. Click the green check in the **Select** dialog. Click **OK** to enter these pipes into the table as a group. There should now be four roughness groups in the table on the **Roughness Groups** tab.
- The calibration method settings are found on the **Calibration Criteria** tab. In this case, these settings will be left as the default settings.
- To use these data to determine the pipe roughness values, create a new run by clicking the **New** button above the left window, then select **New Optimized Run**.
- The pipe roughness values are assumed to have a range between 5 and 140 for each pipe. This is a wide range and any chokes (blockages, partially closed valves, etc.) in a pipe could greatly reduce the pipe roughness value. It is not expected that any roughness value above 140 would not be observed for ductile-iron pipe. On the **Roughness** tab in the **Operation** field, **Set** should be selected; the expected minimum and maximum roughness values are entered in the **Minimum Value** and **Maximum Value** fields. Enter the increment for the software to analyze the roughness values by entering 5 into the **Increment** field. Other increments could be used.
- For this simulation, the top five solutions will be displayed. To do this, click the **Options** tab and enter 5 into the **Solutions to Keep** field.
- To start the run, click the **Compute** button in the Darwin Calibrator toolbar. When the run is completed, close the **Calibration…** window. The top five solutions will be listed under the “New Optimized Run” in the left window.

Answer

The fitness values of the five solutions are shown by clicking the **Solutions** folder. In this example, the fitness values ranged from 0.280 to 0.303 where a lower fitness value indicates a “better” solution. A summary of each solution with its determined roughness values is shown in Table 7.10. To view the determined roughness values for the “best” solution, click the **Solution 1** summary and highlight **Roughness** in the **Adjustments Results** window under the **Solutions** tab. To view the observed and simulated Hydraulic Grade Line (HGL), click the **Simulated Results** tab.

Table 7.10 Darwin Calibrator solutions for Example 7.5

Darwin Calibration solutions				
	Roughness value			
Solution	P-2	P-1	P-8	P-3 and P-7
Solution 1	115	55	140	100
Solution 2	120	55	140	100
Solution 3	100	45	140	100
Solution 4	115	55	125	100
Solution 5	125	55	140	100

These results indicate that pipes P-2 and P-8, and the grouped pipes P-3 and P-7 have a roughness value that is about what would be expected for the installed ductile-iron pipe. However, the results for pipe P-1 show that the roughness value is much lower. This could indicate that a valve is partially closed, the pipe is blocked, or that the pipe diameter may be smaller than expected. In this case, pipe P-1 should be investigated to determine the cause of this low roughness value. If there is a problem and that problem was fixed, new field measurements should be taken.

These roughness values can be entered into the model and further simulations can be conducted. With enough field data, a model that closely simulates the actual system can be created. Keep in mind that many times the person doing the modeling must decide what values to put into the model. The software can only calculate values based on what is entered. The person doing the modeling must judge how accurate the model is and whether the model can be used to make decisions.

7.8 PRACTICAL APPLICATIONS OF COMPUTER-AIDED WATER SUPPLY SYSTEM ANALYSIS

A complete water supply system typically consists of raw water reservoir, raw water intake, pump stations, valve stations, raw water transmission pipelines, water treatment plant, finished water storage reservoir, finished water transmission pipelines, finished water storage towers, water distribution system, fire hydrants, meters, and other appurtenances (Figs. 1.1 and 4.4). The purpose of the water supply system is to deliver adequate quantities of water at sufficient pressures at all times under continually changing conditions while at the same time protecting water quality.

Section 6.9 discusses how a new water system should be designed and how an existing water system should be operated and maintained. A properly designed water system will be able to withstand the physical stresses imposed upon it and to deliver the water supply in sufficient quantity within a proper pressure range. A well-operated and maintained water system will preserve its integrity and ensures high water quality at all times. Hydraulic analysis of a complete water supply system using the Hardy Cross Method is very complex and extremely time consuming, due to the fact that there are too many variables. Application of a computer-aided analysis will allow an engineer to repeat same analytical procedure many times using various parameters for optimization of the water system within a short period of time. The following are a few typical examples for application of computer analyses.

The major hydraulic concerns of a water supply system are water flow, flow direction, velocity, and available water pressure. Of special importance is the maintenance of a continuous positive water pressure, 20 psi or 140 kPa minimum, in the water system under all conditions, so as to protect the water system from the entrance of pathogenic or toxic substances. Excessive pressures (greater than 100 psi or 694 kPa, or 6.94 kg/cm^2) will cause pipe breaks in old water distribution systems and will damage the customers' facilities and fixtures. Any water system modifications, such as pipe addition/deletion, pipe enlargement, pipe interconnection, water storage tank relocation, new subdivision development, new raw water source, new pump station with higher head, and fires in commercial district, will all affect the water flow, flow direction, velocity, and water pressure.

Whenever a water flow in a pipe changes its direction, the direction of the slope of the HGL will also change accordingly. A major fire, leak, or other unexpected high water demand may draw the HGL down to a point that vacuum (i.e., negative pressure) is created at consumers' service connections. This negative pressure could potentially cause backflow from a non-potable contaminated source of water. The variables for a series of computer-aided water system analysis may include the number, location and magnitude of fire, leak or unexpected high water demand (such as emergency water demand request from another water system), and so on.

A SCADA (supervisory control and data acquisition) system is an industrial control system (ICS) commonly used by all modern water systems and most of improved old water systems. With a SCADA system, a water manager is able to observe and document the real-time, live information of water flow, direction, velocity, pressure, water quality data, major facility, elevation, and exact location. Since the sampling, monitoring, and remote sensing instruments are expensive and limited, a SCADA system alone cannot provide the technical information of all pipes, predict the future changes, or determine the causes of past failures. It is possible to feed the real-time SCADA data into the computer-aided Hardy Cross analytical system for generating an approximate, real-time hydraulic and water quality data of all pipes and facilities, if there is sufficient number of SCADA data points available. This could be an interesting research or engineering project to the students if a modern municipal water treatment plant can be available for collaboration.

Another factor in the computer-aided water system analysis is the roughness of all water pipes which resists water flow and causes a drop in water pressure under dynamic (flowing) conditions. Equations (5.32)–(5.37) show how the Hazen–Williams roughness coefficient C affects velocity (v), head loss (h_f), and flow (Q). The smoother the inside of the pipe, the higher the C value, the higher the velocity and flow, and the lower the head loss. The pipe roughness increases with pipe corrosion and deposition of suspended matters along the pipeline over a long period of time. Some examples in Chapter 5 illustrate how the Hazen–Williams roughness coefficient C of an existing pipe can be experimentally determined.

The loss of pressure due to pipe friction is also termed the pipe friction loss, which is not constant. The roughness of the pipe interior creates turbulence that is proportional to the water velocity in the pipe. This velocity is constantly changing with water demand. Velocities of 2.5–5 ft/s (0.76–1.52 m/s) at maximum flows are appropriate. A computer-aided water system analysis may also generate the water velocity data of all pipelines within the water system under various operational conditions.

Dead-end water mains (Fig. 6.2) may develop the water quality problems in terms of high disinfectant by-products, tastes and odors, and therefore should be avoided when possible. Pipe looping for elimination of dead-end water mains requires computer-aided water system optimization.

The valve layout and valve opening in a water distribution system are very important. Section 6.9 discusses the importance of the use of various fittings and the method for determination of their resistances.

For classroom practice or preliminary water system analysis, *C* is frequently assumed to be 100, and the head losses (resistances) of valves and other fittings are ignored. For a real water engineering project, both the true *C* value of each pipe and the resistance of each fitting shall be determined and used in the computer-aided analysis.

Normally a properly designed water supply system should be sufficient to meet the peak water demands, equalizing or operating storage demands, fire reserve, and emergency reserve. If one or more major facilities will be taken off line for repair or replacement, emergency water may come from a neighboring municipal water system. The interconnection of two water systems together requires computer analysis to ensure that both communities can be properly served even under emergency situations.

In summation, the knowledge learned in the classrooms is usually oversimplified and under assumed conditions. In real-world situation, there are too many unknowns and variables. The computer-aided water supply system analysis will allow an engineer to perform repeated analyses until the water system is optimized or its solution found. Possible applications of the computer-aided water supply system analysis include at least the following: (a) low water pressure in certain locations during peak water demand periods or high fire demand; (b) unknown Hazen–William formula coefficients for some of their pipes; (c) planned new subdivision developments in the city/town; (d) proposed new pump station, valve station, and/or interconnection of major pipelines; (e) proposed elimination of dead-end pipes; (f) optimization of a new water storage tank's location; (g) determination of a new water storage tank's elevation; (h) interconnection of the city/town's water supply system with another city/town's water supply system during an emergency situation; (i) frequent pipe breaks due to excessive water pressures in certain areas; (j) the planned lining of old pipes for structural and hydraulic (*C* value) improvements; (k) planned by-passing of major water facilities (such as water reservoir, water storage tower, and transmission pipeline) for renovation, replacement, or elimination; (l) planned partial empty (1/2) of water storage tower in order to reduce chlorination detention time, in turn, to reduce DBP concentrations; (m) incorporation of SCADA system's real-time hydraulic and water quality data into the computer-aided water system analysis, and (n) planned addition or elimination of major water fittings.

Home work problem 7.10 is designed as an Intern project or BS/MS thesis project for civil, environmental, public health, chemical, or mechanical engineering students to gain engineering experience in computer-aided water network analysis.

PROBLEMS/QUESTIONS

Solve the following problems using the WaterGEMS computer program.

7.1 The ductile-iron pipe network shown in Fig. 7.12 carries water at 203°C. Assume that the junctions all have an elevation of 0 m and the reservoir is at 30 m. Use the Hazen–Williams formula ($C = 130$) and the pipe and demand data in Tables 7.11 and 7.12 to perform a steady-state analysis and answer the following questions:

1. Which pipe has the lowest discharge? What is the discharge (in L/min)?
2. Which pipe has the highest velocity? What is the velocity (in m/s)?

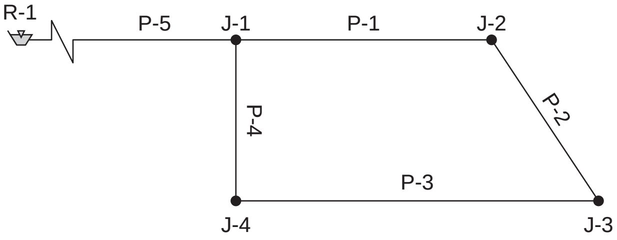

Figure 7.12 Schematic for Problem 7.1.

Table 7.11 Pipe information for Problem 7.1

Pipe	Diameter (mm)	Length (m)
P-1	150	50
P-2	100	25
P-3	100	60
P-4	100	20
P-5	250	760

Table 7.12 Junction information for Problem 7.1

Junction	Demand (L/min)
J-1	570
J-2	660
J-3	550
J-4	550

3. Calculate the problem using the Darcy–Weisbach equation ($k - 0.26$ mm) and compare the results.
4. What effect would raising the reservoir by 20 m have on the pipe flow rates? What effect would it have on the hydraulic grade lines at the junctions?

7.2 A pressure gauge reading of 288 kPa was taken at J-5 in the pipe network shown in Fig. 7.13. Assuming a reservoir elevation of 100 m, find the appropriate Darcy–Weisbach roughness height (to the hundredths place) to bring the model into agreement with these field records. Use the same roughness value for all pipes. The pipe and junction data are given in Tables 7.13 and 7.14, respectively.

1. What roughness factor yields the best results?
2. What is the calculated pressure at J-5 using this factor?
3. Other than the pipe roughnesses, what other factors could cause the model to disagree with field-recorded values for flow and pressure?

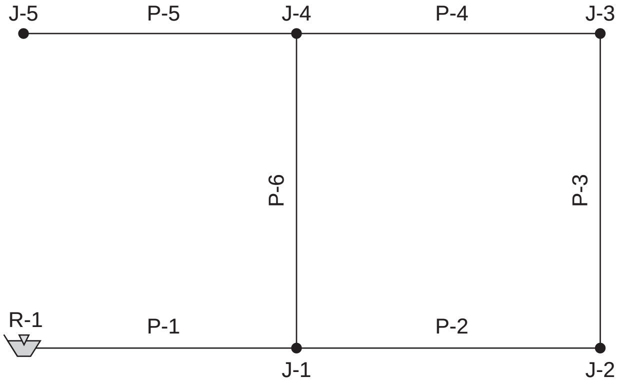

Figure 7.13 Schematic for Problem 7.2.

Table 7.13 Pipe information for Problem 7.2

Pipe	Diameter (mm)	Length (m)
P-1	250	1,525
P-2	150	300
P-3	150	240
P-4	150	275
P-5	150	245
P-6	200	230

Table 7.14 Junction information for Problem 7.2

Junction	Elevation (m)	Demand (L/min)
J-1	55	950
J-2	49	1,060
J-3	58	1,440
J-4	46	1,175
J-5	44	980

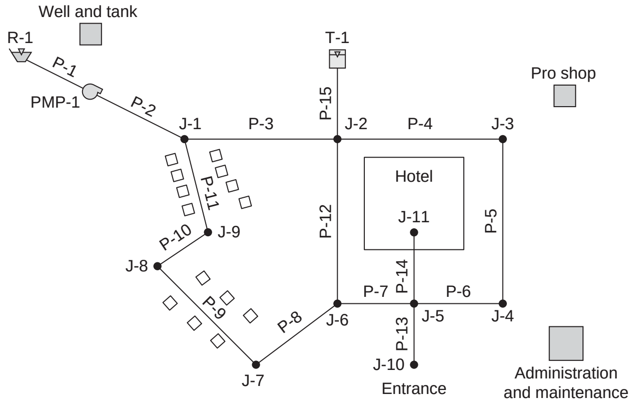

Figure 7.14 Schematic for Problem 7.3.

7.3 A distribution system is needed to supply water to a resort development for normal usage and emergency purposes (such as fighting a fire). The proposed system layout is shown in Fig. 7.14. The source of water for the system is a pumped well. The water is treated and placed in a ground-level tank (shown in the figure as a reservoir because of its plentiful supply), which is maintained at a water surface elevation of 210 ft (64 m). The water is then pumped from this tank into the rest of the system.

The well system alone cannot efficiently provide the amount of water needed for fire protection, so an elevated storage tank is also needed. The bottom of the tank is at 376 ft (114.6 m) (high enough to produce 35 psi or 243 kPa at the highest node), and the top is approximately 20 ft (6.1 m) higher. To avoid the cost of an elevated tank, this 80-ft-diameter (24.4-m-diameter) tank is located on a hillside, 2,000 ft (610 m) away from the main system. Assume that the tank starts with a water surface elevation of 380 ft (115.8 m). The pump was originally sized to deliver 300 gpm (1,135 L/min) with enough head to pump against the tank when it is full. Three defining points on the pump curve are as follows: 0 gpm at 200 ft of head; 300 gpm at 180 ft of head; and 600 gpm at 150 ft of head (0 L/min at 610 m of head; 1,135 L/min at 54.9 m of head; and 2271 L/min at 45.7 m of head). The pump elevation is assumed to be the same as the elevation at J-1, although the precise pump elevation is not crucial to the analysis.

The system is to be analyzed under several demand conditions with minimum and maximum pressure constraints. During normal operations, the junction pressures should be between 35 and 80 psi (243 and 555 kPa). Under fire flow conditions, however, the minimum pressure is allowed to drop to 20 psi (139 kPa). Fire protection is being considered both with and without a sprinkler system.

Demand Alternatives: WaterGEMS enables you to store multiple demand alternatives corresponding to various conditions (such as average day and peak hour). This feature allows you to run different scenarios that incorporate various demand conditions within a single project file without losing any input data. For an introduction and more information about scenarios and alternatives, see WaterGEMS's online help system.

Pipe Network: The pipe network consists of the pipes listed in Table 7.15. The diameters shown are based on the preliminary design and may not be adequate for the final design. For all pipes, use ductile iron as the material and a Hazen–Williams *C* factor of 130. The junction information for this problem is given in Table 7.16.

To help keep track of important system characteristics (like maximum velocity and lowest pressure), you may find it helpful to keep a table such as Table 7.17.

Table 7.15 Pipe information for Problem 7.3

	Diameter		Length	
Pipe	(in)	(mm)	(ft)	(m)
P-1	8	200	20	6.1
P-2	8	200	300	91.4
P-3	8	200	600	182.9
P-4	6	150	450	137.2
P-5	6	150	500	152.4
P-6	6	150	300	91.4
P-7	8	200	250	76.2
P-8	6	150	400	121.9
P-9	6	150	400	121.9
P-10	6	150	200	61.0
P-11	6	150	500	152.4
P-12	8	200	500	152.4
P-13	6	150	400	121.9
P-14	6	150	200	61.0
P-15	10	250	2,000	609.6

Table 7.16 Junction information for Problem 7.3

	Elevation		Average day		Peak hour		Minimum hour		Fire with sprinkler		Fire without sprinkler	
Junction	(ft)	(m)	(gpm)	(L/min)	(gpm)	(L/min)	(gpm)	(L/min)	(gpm)	(L/min)	(gpm)	(L/min)
J-1	250	76.2	0	0	0	0	0	0	0	0	0	0
J-2	260	79.2	0	0	0	0	0	0	0	0	0	0
J-3	262	79.9	20	75.7	50	189.3	2	7.6	520	1,968.2	800	3,028
J-4	262	79.9	20	75.7	50	189.3	2	7.6	520	1,968.2	800	3,028
J-5	270	82.3	0	0	0	0	0	0	0	0	800	3,028
J-6	280	85.3	0	0	0	0	0	0	0	0	800	3,028
J-7	295	89.9	40	151.4	100	378.5	2	7.6	40	151.4	40	151.4
J-8	290	88.4	40	151.4	100	378.5	2	7.6	40	151.4	40	151.4
J-9	285	86.9	0	0	0	0	0	0	0	0	0	0
J-10	280	85.3	0	0	0	0	0	0	360	1,362.6	160	605.6
J-11	270	82.3	160	605.6	400	1,514.0	30	113.6	160	605.6	160	605.6

Table 7.17 Results summary for Problem 7.3

Variable	Average day	Peak hour	Minimum hour	Fire with sprinkler	Fire without sprinkler
Node w/ low pressure					
Low pressure (psi)					
Node w/ high pressure					
High pressure (psi)					
Pipe w/ max. velocity					
Max. velocity (ft/s)					
Tank in/out flow (gpm)					
Pump discharge (gpm)					

Conversion factors: 1 ft/s = 0.3048 m/s; 1 gpm = 3.785 L/min; 1 psi = 6.94 kPa.

Table 7.18 Color-coding range for Problem 7.3

Max. velocity		
(ft/s)	(m/s)	Color
0.5	0.15	Magenta
2.5	0.76	Blue
5.0	1.52	Green
8.0	2.44	Yellow
20.0	6.10	Red

Another way to quickly determine the performance of the system is to color-code the pipes according to some indicator.

In hydraulic design, a good performance indicator is often the velocity in the pipes. Pipes consistently flowing below 0.5 ft/s (0.15 m/s) may be oversized. Pipes with velocities over 5 ft/s (1.5 m/s) are fairly heavily stressed, and those with velocities above 8 ft/s (2.4 m/s) are usually bottlenecks in the system under that flow pattern. Color-code the system using the ranges given in Table 7.18. After you define the color-coding, place a legend in the drawing (see Table 7.18).

1. Fill in or reproduce the Results Summary table after each run to get a feel for some of the key indicators during various scenarios.
2. For the average day run, what is the pump discharge?
3. If the pump has a best efficiency point at 300 gpm (1,135.5 L/min), what can you say about its performance on an average day?
4. For the peak hour run, the velocities are fairly low. Does this mean you have oversized the pipes? Explain.
5. For the minimum hour run, what was the highest pressure in the system? Why would you expect the highest pressure to occur during the minimum hour demand?
6. Was the system (as currently designed) acceptable for the fire flow case with the sprinkled building? On what did you base this decision?
7. Was the system (as currently designed) acceptable for the fire flow case with all the flow provided by hose streams (no sprinklers)? If not, how would you modify the system so that it will work?

7.4 A ductile-iron pipe network ($C = 130$) is shown in Fig. 7.15. Use the Hazen–Williams equation to calculate friction losses in the system. The junctions and pump are at an elevation of 5 ft (1.52 m) and all pipes are 6 in. (150 mm) in diameter. (*Note:* Use a standard, three-point pump curve. The data for the pump, junctions, and pipes are in Tables 7.19, 7.20, and 7.21.) The water surface of the reservoir is at an elevation of 30 ft (9.14 m).

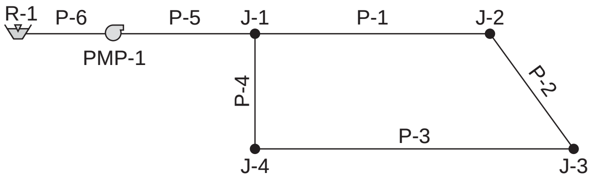

Figure 7.15 Schematic for Problem 7.4.

Table 7.19 Pump information for Problem 7.4

Head (ft)	Head (m)	Flow (gpm)	Flow (L/min)
200	60.96	0	0
175	53.34	1,000	3,785
100	30.48	2,000	7,570

Table 7.20 Junction information for Problem 7.4

Junction Label	Demand (L/min)	Demand (gpm)
J-1	1,514	400
J-2	2,082	550
J-3	2,082	550
J-4	1,325	350

Table 7.21 Pipe information for Problem 7.4

Pipe label	Length (m)	Length (ft)
P-1	23.77	78
P-2	12.19	40
P-3	27.43	90
P-4	11.89	39
P-5	3.05	10
P-6	3.05	10

1. What are the resulting flows and velocities in the pipes?
2. What are the resulting pressures at the junction nodes?
3. Place a check valve on pipe P-3 such that the valve only allows flow from J-3 to J-4. What happens to the flow in pipe P-3? Why does this occur?
4. When the check valve is placed on pipe P-3, what happens to the pressures throughout the system?
5. Remove the check valve on pipe P-3. Place a 6 in. (150 mm) flow control valve (FCV) node at an elevation of 5 ft (1.52 m) on pipe P-3. The FCV should be set so that it only allows a flow of 100 gpm (378.5 L/min) from J-4 to J-3. (*Hint:* A check valve is a pipe property.) What is the resulting difference in flows in the network? How are the pressures affected?
6. Why does not the pressure at J-1 change when the FCV is added?
7. What happens if you increase the FCV's allowable flow to 2,000 gpm (7,570 L/min). What happens if you reduce the allowable flow to zero?

7.5 A local country club has hired you to design a sprinkler system that will water the greens of their nine-hole golf course. The system must be able to water all nine holes at once. The water supply has a water surface elevation of 10 ft (3.05 m). All pipes are PVC (C= 150; use the Hazen–Williams equation to determine friction losses). Use a standard, three-point pump curve for the pump, which is at an elevation of 5 ft (1.52 m). The flow at the sprinkler is modeled using an emitter coefficient. The data for the junctions, pipes, and pump curve are given in Tables 7.22, 7.23, and 7.24. The initial network layout is shown in Fig. 7.16.

1. Determine the discharge at each hole.
2. What is the operating point of the pump?

Table 7.22 Junction information for Problem 7.5

Junction label	Emitter coefficient		Elevation	
	(gpm/psi$^{0.5}$)	(L/min/kPa$^{0.5}$)	(ft)	(m)
J-1	—	—	10	3.05
Hole 1	8	11.49	7	2.13
Hole 2	10	14.37	7	2.13
Hole 3	15	21.55	40	12.19
Hole 4	12	17.24	5	1.52
Hole 5	8	11.49	5	1.52
Hole 6	8	11.49	15	4.57
Hole 7	10	14.37	20	6.10
Hole 8	15	21.55	10	3.05
Hole 9	8	11.69	12	3.66

Table 7.23 Pipe information for Problem 7.5

Pipe label	Diameter (mm)	Diameter (in)	Length (m)	Length (ft)
P-1	100	4	3.05	10
P-2	100	4	304.8	1,000
P-3	100	4	243.8	800
P-4	76	3	228.6	750
P-5	76	3	152.4	500
P-6	76	3	213.4	700
P-7	50	2	121.9	400
P-8	100	4	243.8	800
P-9	76	3	152.4	500
P-10	50	2	121.9	400
P-11	50	2	152.4	500

Table 7.24 Pump information for Problem 7.5

Head (ft)	Head (m)	Flow (gpm)	Flow (L/min)
170	51.8	0	0
135	41.1	300	1,135.5
100	30.5	450	1,703.3

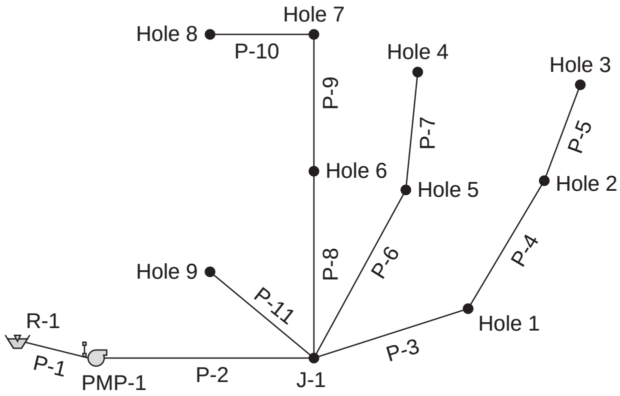

Figure 7.16 Schematic for Problem 7.5.

7.6 A subdivision of 36 homes is being constructed in a new area of town. Each home will require 1.7 L/s during peak periods. All junction nodes are 192 m in elevation. All pipes are ductile iron (*C*= 130; use the Hazen–Williams equation to determine the friction losses in the pipe). The current lot and network layout are shown Fig. 7.17. Junction and pipe information are given in Tables 7.25 and 7.26.

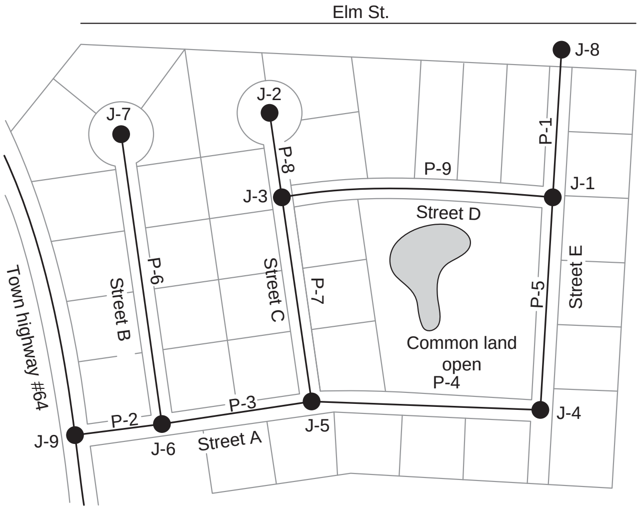

Figure 7.17 Schematic for Problem 7.6.

Table 7.25 Junction information for Problem 7.6

Junction label	Number of lots serviced
J-1	5
J-2	4
J-3	4
J-4	5
J-5	6
J-6	6
J-7	6

Table 7.26 Pipe information for Problem 7.6

Pipe label	Length (m)	Diameter (mm)
P-1	60.0	150
P-2	60.0	150
P-3	110.5	150
P-4	164.0	150
P-5	152.5	150
P-6	204.0	100
P-7	148.0	150
P-8	61.0	100
P-9	194.0	150

Currently, a model of the entire water system does not exist. However, hydrant tests were conducted using hydrants located on two water mains, one in Town Highway #64 and the other in Elm Street. The following data were obtained:

Town Highway #64 Hydrant Test

Static pressure:	310.3 kPa
Residual pressure:	98.5 kPa at 32 L/s
Elevation of pressure gauge:	190 m

Elm Street Hydrant Test

Static pressure:	413.7 kPa
Residual pressure:	319.3 kPa at 40 L/s
Elevation of pressure gauge:	191.5 m

The subdivision will connect to existing system mains in these streets at nodes J-8 and J-9. (*Hint:* Model the connection to an existing water main with a reservoir and a pump.)

1. What are the demands at each of the junction nodes? What is the total demand?
2. Does the present water distribution system have enough capacity to supply the new subdivision?
3. Which connection to the existing main is supplying more water to the subdivision? Why?
4. Are the proposed pipe sizes adequate to maintain velocities between 0.15 and 2.44 m/s, and pressures of at least 140 kPa?
5. Would the subdivision have enough water if only one connection were used? If so, which one?
6. What do you think are some possible pitfalls of modeling two connections to existing mains within the same system, as opposed to modeling back to the water source?

7.7 Use the pipe sizes given in Table 7.27 for the subdivision in Problem 7.6. City ordinances require that the pressure at the fire flow discharge and at other points in the distribution system cannot fall below 125 kPa during a fire flow of 34 L/s. (*Hint:* The total flow at the fire flow node does not need to include the baseline demand.)

Table 7.27 Pipe information for Problem 7.7

Pipe label	Diameter (mm)
P-1	200
P-2	150
P-3	150
P-4	150
P-5	150
P-6	150
P-7	150
P-8	150
P-9	150

1. If a residential fire occurs at J-7, would the current system be able to meet the fire flow requirements set by the city?
2. If not, what can be done to increase the available flow to provide adequate fire flow to that hydrant?
3. If a fire flow is placed at J-4, does the system meet the requirements with the proposed improvements? Without the proposed improvements?

7.8 A local water company is concerned with the water quality within its water distribution network (Fig. 7.18). They want to determine the age and the chlorine concentration of the water as it exits the system at different junctions. The water surface at the reservoir is 70 m.

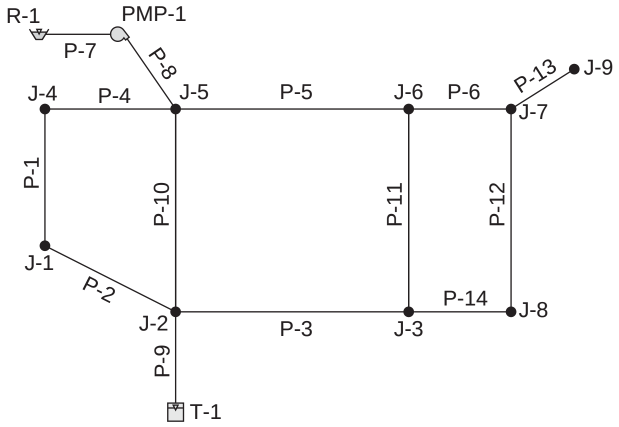

Figure 7.18 Schematic for Problem 7.8.

Chlorine is injected into the system at the source of flow, R-1, at a concentration of 1 mg/L. It has been determined through a series of bottle tests that the average bulk reaction rate of the chlorine in the system (including all pipes and tanks) is approximately –0.5 per day. Pump information and stepwise water demand pattern data are as shown in Tables 7.28 and 7.29. Junction and pipe information are given in Tables 7.30 and 7.31.

Table 7.28 Pump information for Problem 7.8

Head (m)	Discharge (L/min)	Controls
40	0	Off if node T-1 above 103.5 m
35	3,000	On if node T-1 below 100.5 m
24	6,000	

Table 7.29 Stepwise demand pattern data for Problem 7.8

Time from start (h)	Multiplier	Time from start (h)	Multiplier
0	0.80	13	1.30
1	0.60	14	1.40
2	0.50	15	1.50
3	0.50	16	1.60
4	0.55	17	1.80
5	0.60	18	1.80
6	0.80	19	1.40
7	1.10	20	1.20
8	1.50	21	1.00
9	1.40	22	0.90
10	1.30	23	0.80
11	1.40	24	0.80
12	1.40		

Table 7.30 Junction data for Problem 7.8

Junction	Elevation (m)	Demand (L/min)
J-1	73	151
J-2	67	227
J-3	81	229
J-4	56	219
J-5	67	215
J-6	73	219
J-7	55	215
J-8	84	180
J-9	88	151

Table 7.31 Pipe data for Problem 7.8

Pipe	Length (m)	Diameter (mm)	Roughness
P-1	300	200	130
P-2	305	200	130
P-3	300	200	130
P-4	200	200	130
P-5	300	300	130
P-6	200	200	130
P-7	1	300	130
P-8	5,000	300	130
P-9	300	300	130
P-10	500	200	130
P-11	500	200	130
P-12	500	200	130
P-13	150	150	130
P-14	200	200	130

The cylindrical tank has a diameter of 15 m. The base and minimum elevations are 99 m. The maximum elevation is 104 m and the initial elevation is 103.4 m.

1. Perform an age analysis on the system using a duration of 300 h and a time step of 2 h. Fill in the results in Table 7.32, indicating the maximum water age at each junction and tank after the system reaches equilibrium (a pattern of average water age vs. time becomes evident). What point in the system generally has the oldest water? Explain why the water is oldest at this location.
2. Perform a constituent analysis using the same duration and time step as in part 1. Fill in the results in Table 7.32, indicating the minimum chlorine concentration for each junction and tank after the system has reached equilibrium (a pattern of concentration vs. time becomes evident). What point in the system has the lowest chlorine concentration? Explain why the chlorine residual is lowest at this location.

Table 7.32 Results table for Problem 7.8

Junction	J-1	J-2	J-3	J-4	J-5	J-6	J-7	J-8	J-9	T-1
Age (h)										
Chlorine concentration (mg/L)										

3. From the above table and graphs of demand, age, and concentration versus time generated within WaterGEMS, determine the following correlations:
   (a) Age and chlorine concentration;
   (b) Demand and chlorine concentration at a junction;
   (c) Demand and water age at a junction.
4. Why is it necessary to run the model for such a long time? Do you feel that 300 h is too long or too short a time period for testing the model? Why?

7.9 A planning commission has indicated a new industry may be connected to the water system described in Problem 7.8. You are to determine the pipe diameters in the network to minimize the installation cost assuming all the pipes are ductile iron. Use the Darwin Designer module to determine the total cost and size each pipe for each of the following conditions. Use the pipe cost information from Problem 7.4 for the ductile-iron pipe.

I. Size the pipes using a demand multiplier (peaking factor) of 3.2. The pressure must remain between 170 and 550 kPa during peak demand. Exclude pipes P-7 and P-8 in your analysis when determining the pipe sizes. (*Hint:* You will need to specify an additional demand of zero (0 L/min) with the default pressure constraints at the junctions or a fatal error will occur.)

II. It is expected that a new industry with an expected additional demand of 2,000 L/min with a required minimum pressure of 260 kPa will be added to the system. It could be tapped into the network at either junction 6, 7, or 8. Size the pipes for the conditions in part I along with the industry added to all proposed junctions. You will need to analyze the network three times, once for the industry at J-6, again with the industry at J-7, then finally with the industry at J-8.

1. Indicate which option(s) would work best.
2. Which junction should the industry be tapped into to be the least costly and what is the expected cost?
3. What is Table 7.32 Results Table for Problem 7.8 the size of each pipe for the best solution at the least costly option with the industry added?
4. What is the calculated minimum pressure at the industry for the best solution?

7.10 Find and help a city or town in your local area, which has at least one of the following engineering problems facing their water supply system, such as (a) low water pressure in certain locations during peak water demand periods or high fire demand; (b) unknown Hazen–William formula coefficients for some of their pipes; (c) planned new subdivision developments in the city/town; (d) proposed new pump station, valve station, and/or interconnection of major pipelines; (e) proposed elimination of dead-end pipes; (f) optimization of a new water storage tank's location; (g) determination of a new water storage tank's elevation; (h) interconnection of the city/town's water supply system with another city/town's water supply system during an emergency situation; (i) frequent pipe breaks due to excessive water pressures in certain areas; (j) the planned lining of old pipes for structural and hydraulic (*C* value) improvements; (k) planned by-passing of major water facilities (such as water reservoir, water storage tower, and transmission pipeline) for renovation, replacement, or elimination; (l) planned partial empty (1/2) of water storage tower in order to reduce chlorination detention time for reducing DBP concentrations; (m) incorporation of SCADA system's real-time hydraulic and water quality data into the computer-aided water system analysis, and (n) planned addition or elimination of major water fittings. The use of computer analyses for optimization of your engineering solutions is required.

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Chapter 8

Pumping, Storage, and Dual Water Systems

8.1 PUMPS AND PUMPING STATIONS

Pumps and pumping stations (Figs. 8.1 and 8.2) serve the following purposes in water systems:

1. Lifting water from its source (surface or ground), either immediately to the community through *high-lift* installations, or by *low-lift* systems to purification works
2. Boosting water from low-service to high-service areas, to separate fire supplies, and to the upper floors of many-storied buildings
3. Transporting water through treatment works, backwashing filters, draining component settling tanks, and other treatment units; withdrawing deposited solids; and supplying water (especially pressure water) to operating equipment

In addition to *centrifugal* and *propeller pumps*, water systems may include (a) *displacement pumps*, ranging in size from hand-operated pitcher pumps to the huge pumping engines of the last century built as steam-driven units; (b) *rotary pumps* equipped with two or more rotors (varying in shape from meshing lobes to gears and often used as small fire pumps); (c) *hydraulic rams* utilizing the impulse of large masses of low-pressure water to drive much smaller masses of water (one-half to one-sixth of the driving water) through the delivery pipe to higher elevations, in synchronization with the pressure waves and sequences induced by water hammer; (d) *jet pumps* or jet ejectors, used in wells and dewatering operations, introducing a high-speed jet of air through a nozzle into a constricted section of pipe; (e) *air lifts* in which air bubbles, released from an upward-directed air pipe, lift water from a well or sump through an *eductor pipe*; and (f) *displacement ejectors* housed in a pressure vessel in which water (especially wastewater) accumulates and from which it is displaced through an eductor pipe when a float-operated valve is tripped by the rising water and admits compressed air to the vessel.

Today most water pumping is done by either centrifugal pumps or propeller pumps. These are usually driven by electric motors, less often by steam turbines, internal combustion engines, or hydraulic turbines. How the water is directed through the impeller determines the type of pump. There are (1) *radial flow* in open- or closed-*impeller pumps*, with volute or turbine casings, and single or double suction through the eye of the impeller; (2) *axial flow* in *propeller pumps*; and (3) *diagonal flow* in *mixed-flow*, *open-impeller pumps*. Propeller pumps are not centrifugal pumps. Both centrifugal pumps and propeller pumps can be referred to as *rotodynamic* pumps.

Single-stage pumps have but *one impeller*, and *multistage pumps* have *two or more*, each feeding into the next higher stage. Multistage turbine well pumps may have their motors submerged, or they may be driven by a shaft from the prime mover situated on the floor of the pumping station.

8.2 PUMP CHARACTERISTICS

A centrifugal pump is defined by its *characteristic curve*, which relates the *pump head* (head added to the system) to the flow rate. Pumping units are chosen in accordance with system heads and pump characteristics. As shown in Fig. 8.3, the system head is the sum of the static and dynamic heads against the pump. As such, it varies with required flows and with changes in storage and *suction levels*. When a distribution system lies between pump and distribution reservoir, the system head also responds to fluctuations in demand. Pump characteristics depend on pump size, speed, and design. For a given speed N in revolutions/min, they are determined by the relationships between the rate of discharge Q, usually in gpm (or L/m or m^3/s) and the head H in ft (or m), the efficiency E in %, and the power input P in horsepower (or

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Water Engineering: Hydraulics, Distribution and Treatment, First Edition. Nazih K. Shammas and Lawrence K. Wang.

Figure 8.1 Pumping station Van Sasse in Grave, the Netherlands (*Source:* http://en.wikipedia.org/wiki/Image:Gemaal_van_sasse_interieur.jpg).

Figure 8.2 General view of the pumping station Van Sasse in Grave, the Netherlands (*Source:* http://en.wikipedia.org/wiki/Image:Gemaal_van_sasse.jpg).

kilowatt). For purposes of comparison, pumps of given geometrical design are also characterized by their *specific speed* N_s, the hypothetical speed of a homologous (geometrically similar) pump with an impeller diameter D such that it will discharge 1 gpm (3.78 L/m) against a 1 ft (0.30 m) head. Because discharge varies as the product of area and velocity, and velocity varies as $H^{1/2}$, Q varies as $D^2H^{1/2}$. But velocity

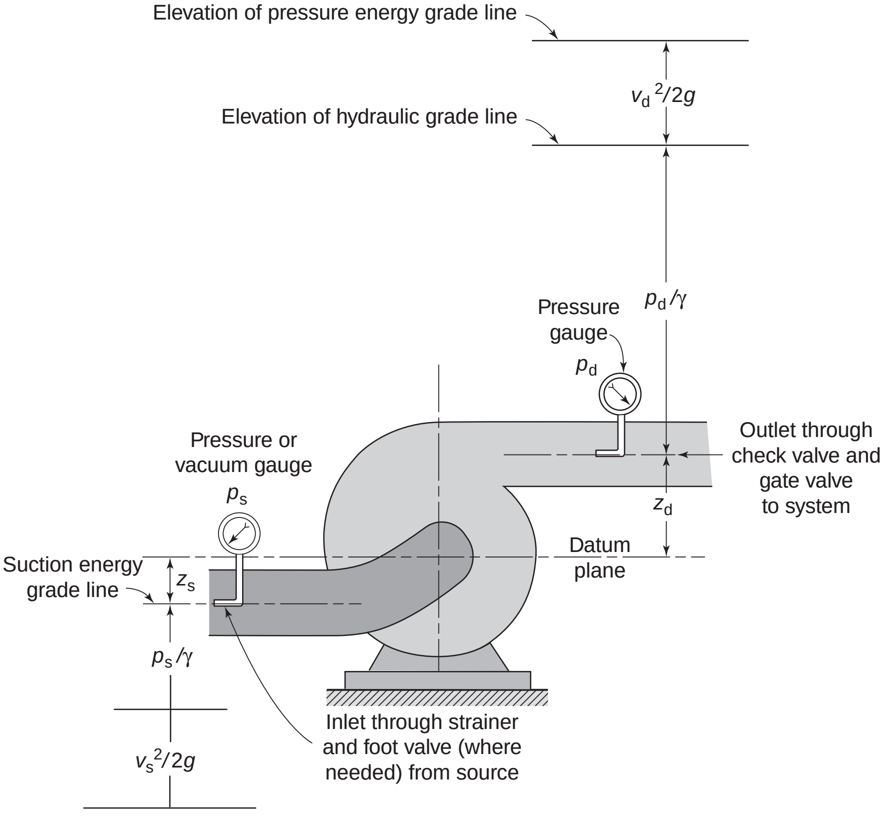

Figure 8.3 Head relationships in pumping systems. System head II $= (z_d - z_s) + (p_d/\gamma - p_s/\gamma) + (v_d^2/2g - v_s^2/2g)$. For suction lift, p_s/γ and z_s are negative, and $v_s^2/2g$ is positive. For suction pressure, p_s/γ and $v_s^2/2g$ are positive, and z_s is negative.

also varies as $\pi DN/60$. Hence, $H^{1/2}$ varies as DN, or N varies as $H^{3/4}/Q^{1/2}$, and the specific speed becomes

$$N_s = NQ^{1/2}/H^{3/4} \tag{8.1}$$

where

N_s = specific speed, rpm
N = speed, rpm
Q = capacity, gpm (m^3/s)
H = head, ft (m)

To obtain the specific speed based on US customary units of head and capacity, multiply the specific speed based on metric units of head and capacity by 52.

Generally speaking, pump efficiencies increase with pump size and capacity. Below specific speeds of 1000 units, efficiencies drop off rapidly. Radial-flow pumps perform well between specific speeds of 1,000 and 3,500 units; mixed-flow pumps in the range of 3,500–7,500 units; and axial-flow pumps after that up to 12,000 units. As shown in Eq. (8.1), for a given N, high-capacity, low-head pumps have the highest specific speeds. For double-suction pumps, the specific speed is computed for half the capacity. For multistage pumps, the head is distributed between the stages. In accordance with Eq. (8.1), this keeps the specific speed high and with it, the efficiency.

Changing the speed of a centrifugal pump will change its operating characteristics including the water flow. If the speed of a pump (rpm) is changed, its water flow will change as follows:

$$Q_2 = (Q_1)(N_2/N_1) \tag{8.2}$$

where

Q_2 = water flow, gpm (m^3/s)
Q_1 = rated water flow, gpm (m^3/s)
N_2 = pump speed now, rpm
N_1 = rated pump speed, rpm

Changing the speed of a centrifugal pump will also change its head. If the speed of a pump (rpm) is changed, its head will change as follows:

$$H_2 = (H_1)(N_2/N_1)^2 \tag{8.3}$$

where

H_2 = head now, ft (m)
H_1 = rated head, ft (m)
N_2 = pump speed now, rpm
N_1 = rated pump speed, rpm

On changing the speed of a centrifugal pump its power requirement will change as follows:

$$P_2 = (P_1)(N_2/N_1)^3 \tag{8.4}$$

where

P_2 = power requirement now, hp (kW)
P_1 = rated power requirement, hp (kW)
N_2 = pump speed now, rpm
N_1 = rated pump speed, rpm

EXAMPLE 8.1 DETERMINATION OF PUMP'S NEW POWER REQUIREMENT

Changing the speed of a centrifugal pump will change its operating characteristics, including the power requirement.

Determine the new power requirement, if

P_1 = rated head = 20 hp = 14.914 kW
N_2 = pump speed now = 1,200 rpm
N_1 = rated pump speed = 1,425 rpm

Solution 1 (US Customary System):

$$\begin{aligned} P_2 &= (P_1)(N_2/N_1)^3 \\ &= (20\text{ hp})(1{,}200/1{,}425)^3 = \mathbf{12\ hp}. \end{aligned}$$

Solution 2 (SI System):

$$\begin{aligned} P_2 &= (P_1)(N_2/N_1)^3 \\ &= (14.914\text{ kW})(1{,}200/1{,}425)^3 = \mathbf{8.948\ kW}. \end{aligned}$$

8.2.1 Power Requirements and Efficiencies of Pumps

Work can be expressed as lifting a weight a certain vertical distance. It is usually defined in terms of ft-lb of work. Power is work per unit of time. These statements are expressed mathematically by the following relationships:

$$\text{Work} = WH \tag{8.5}$$

$$\text{Power} = \text{Work}/t \tag{8.6}$$

where

Work = work, ft-lb (m-kg)
W = weight, lb (kg)
H = height, ft (m)
Power = power, ft-lb/s (m-kg/s)
t = time, s

If the water flow from a pump is converted to weight of water and multiplied by the vertical distance it is lifted, the amount of work or power done can be represented by the following equation:

$$\text{Power} = \text{Work}/t = WH/t$$

$$\text{HP} = (Q,\ \text{gpm})(8.34\ \text{lb/gal})(H,\ \text{ft})(\text{min}/60\ \text{s}) \times(\text{horsepower}/550\ \text{ft-lb/s})$$

$$\text{HP} = (Q)(H)/3{,}957 \tag{8.7a}$$

where Q = water flow, gpm; H = head or lift, ft; and HP = power in US customary unit, hp.

In the SI or metric system:

$$\text{MP} = (Q, \text{m}^3/\text{s})(1{,}000\ \text{kg/m}^3)(H, \text{m})(\text{kW}/101.97\ \text{m-kg/s})$$

$$\text{MP} = 9.8066(Q)(H) \tag{8.7b}$$

where Q = water flow, m^3/s; H = head or lift, m; and MP = power in metric unit, kW. Here 1 kW = 101.97 m-kg/s = 1,000 W; 1 W = 0.102 m-kg/s = 1 J/s; and 1 hp = 0.746 kW = 550 ft-lb/s.

The total power input can be determined using the following equation:

$$\text{INHP} = \text{input power in US customary unit} = (V)(A)/746 \tag{8.8a}$$

$$\text{INMP} = \text{input power in metric unit} = 0.001(V)(A) \tag{8.8b}$$

where V = voltage, V; and A = current, amp.

The wire-to-water efficiency is the efficiency of the total power input to produce water horsepower:

$$E_{\text{ww}} = (E_{\text{pump}})(E_{\text{motor}})(E_{\text{o}}) \tag{8.9}$$

$$E_{\text{ww}} = \text{water horsepower}/\text{input horsepower} \tag{8.10}$$

where

E_{ww} = wire-to-water efficiency
E_{pump} = efficiency of the pump
E_{motor} = efficiency of the motor driving the pump
E_{o} = efficiency of any other parts in the entire motor-pump-wiring system

EXAMPLE 8.2 DETERMINATION OF PUMP POWER

Determine the water horsepower, break horsepower, and motor horsepower for a pump operating under the following conditions: water flow of 620 gpm (0.039 m^3/s = 39 L/s) is to be pumped against a total head of 135 ft (41.15 m); the pump efficiency is 80%; and the motor driving the pump has an efficiency of 90%.

Solution 1 (US Customary System):

$$\text{WHP} = \text{water horsepower in US customary unit} = (Q)(H)/3{,}957$$

$$= (620)(135)/3{,}957 = \mathbf{21.2\ hp} \tag{8.11}$$

(= 8.7a)

$$\text{BHP} = \text{break horsepower in US customary unit} = \text{WHP}/E_{\text{pump}}$$

$$= 21.2/0.80 = \mathbf{26.5\ hp} \tag{8.12}$$

$$\text{MHP} = \text{motor horsepower in US customary unit} = \text{BHP}/E_{\text{motor}}$$

$$= 26.5/0.90 = \mathbf{29.4\ hp} \tag{8.13}$$

where Q = water flow, gpm; and H = head or lift, ft.

Solution 2 (SI System):

$$\begin{aligned} \text{WMP} &= \text{water power in metric unit} = 9.8066(Q)(H) \\ &= 9.8066(0.0391)(41.15) = 15.7866\ \text{kW} \end{aligned} \qquad \textbf{(8.14)}$$

(= 8.7b)

$$\begin{aligned} \text{BMP} &= \text{break power in metric unit} = \text{WMP}/E_{\text{pump}} \\ &= 15.7866/0.80 = 19.733\ \text{kW} \end{aligned} \qquad \textbf{(8.15)}$$

$$\begin{aligned} \text{MMP} &= \text{motor power in metric unit} = \text{BMP}/E_{\text{motor}} \\ &= 19.733/0.90 = 21.92\ \text{kW} \end{aligned} \qquad \textbf{(8.16)}$$

where Q = water flow, m^3/s; and H = head or lift, m.

EXAMPLE 8.3 COMPUTATION OF TOTAL POWER INPUT

Determine the total power input if the electrical input to a motor-pump system is 220 V and 25 amp.

Solution 1 (US Customary System):

$$\begin{aligned} \text{INHP} &= \text{input power in US customary unit} = (V)(A)/746 \\ &= (220)(25)/746 = 7.4\ \text{hp.} \end{aligned} \qquad \textbf{(8.8a)}$$

Solution 2 (SI System):

$$\begin{aligned} \text{INMP} &= \text{input power in metric unit} = 0.001(V)(A) \\ &= 0.001(220)(25)/746 = 5.5\ \text{kW.} \end{aligned} \qquad \textbf{(8.8b)}$$

8.2.2 Cavitation

Specific speed is an important criterion, too, of safety against *cavitation*, a phenomenon accompanied by vibration, noise, and rapid destruction of pump impellers. Cavitation occurs when enough potential energy is converted to kinetic energy to reduce the absolute pressure at the impeller surface below the vapor pressure of water at the ambient temperature. Water then vaporizes and forms pockets of vapor that collapse suddenly as they are swept into regions of high pressure. Cavitation occurs when inlet pressures are too low or when pump capacity or speed of rotation is increased without a compensating rise in inlet pressure. Lowering a pump in relation to its water source, therefore, reduces cavitation.

If we replace the head H in Eq. (8.1) by H_{sv}, the *net positive inlet* or *suction head*, the difference between the total inlet head (the absolute head plus the velocity head in the inlet pipe) and the head corresponding to the vapor pressure of the water pumped (table in Appendix 4), we obtain the suction specific speed (S):

$$S = NQ^{0.5}/H_{\text{sv}}^{0.75} \quad \text{(US customary units)} \qquad \textbf{(8.17a)}$$

where S is the specific speed, dimensionless; N is the rotative speed, rpm; Q is the capacity, gpm; and H_{sv} is the net positive inlet or suction head, ft.

Equation (8.17b) is a specific speed equation using the SI or metric units:

$$S = 51.7\ NQ^{0.5}/H_{\text{sv}}^{0.75} \quad \text{(SI units)} \qquad \textbf{(8.17b)}$$

where S is the specific speed, dimensionless; N is the rotative speed, rpm; Q is the capacity, m^3/s; and H_{sv} is the net positive inlet or suction head, m.

The specific speed, S, is a number that can be used to compare the performance of specific pump impellers under various conditions of rotative speed, capacity, and head per stage.

The units used to determine specific speed must be consistent within the numerical system for S to become meaningful. Certain general safe limits have been established for S by experiment. The following are examples:

Single-suction pumps with overhung impellers
$S <$ 8,000–12,000

Single-stage pumps with shaft through eye of impeller
$S \leq$ 7,000–11,000

High-pressure, multistage pumps (single suction)
$S <$ 5,500–7,500

High-pressure, multistage pumps with special first-stage
impeller (single suction) $S \leq$ 7,500–10,000

$H_{sv} = p_s/\gamma + v_s^2/2g - p_w/\gamma$, where p_s/γ is the absolute pressure, v_s is the velocity of the water in the inlet pipe, and p_w is the vapor pressure of the water pumped, with γ being the specific weight of water and g the gravity constant. The energy grade line is at a distance $h_s = p_a/\gamma - p_s/\gamma + v_s^2/2g$ from the eye of the impeller to the head delivered by the pump, where p_a is the atmospheric pressure. The ratio H_{sv}/h_s, where $h = p_a/\gamma - h_s$, is called the *cavitation parameter*.

8.2.3 Performance Characteristics

Common performance characteristics of a centrifugal pump operating at constant speed are illustrated in Fig. 8.4. Note that the *shut-off head* is a fixed limit and that power consumption is minimum at shut-off. For this reason, centrifugal pumps, after being *primed* or filled with water, are often started with the pump discharge valve closed. As the head falls past the point of maximum efficiency, normal discharge, or rated capacity of the pump (point 1 in Fig. 8.4), the power continues to rise. If a centrifugal pump is operated against too low a head, a motor selected to operate the pump in the head range around maximum efficiency may be overloaded. Pump delivery can be regulated (a) by a valve on the discharge line, (b) by varying the pump speed mechanically or electrically, or (c) by throwing two or more pumps in and out of service to best advantage.

What happens when more than one pumping unit is placed in service is shown in Fig. 8.4 with the help of a curve for the system head. Obviously, pumping units can operate only at the point of intersection of their own head curves with the system head curve. In practice, the system head varies over a considerable range at a given discharge (Fig. 8.5). For example, where a distributing reservoir is part of a system and both the reservoir and the source of water fluctuate in elevation, (a) a *lower* curve identifies head requirements when

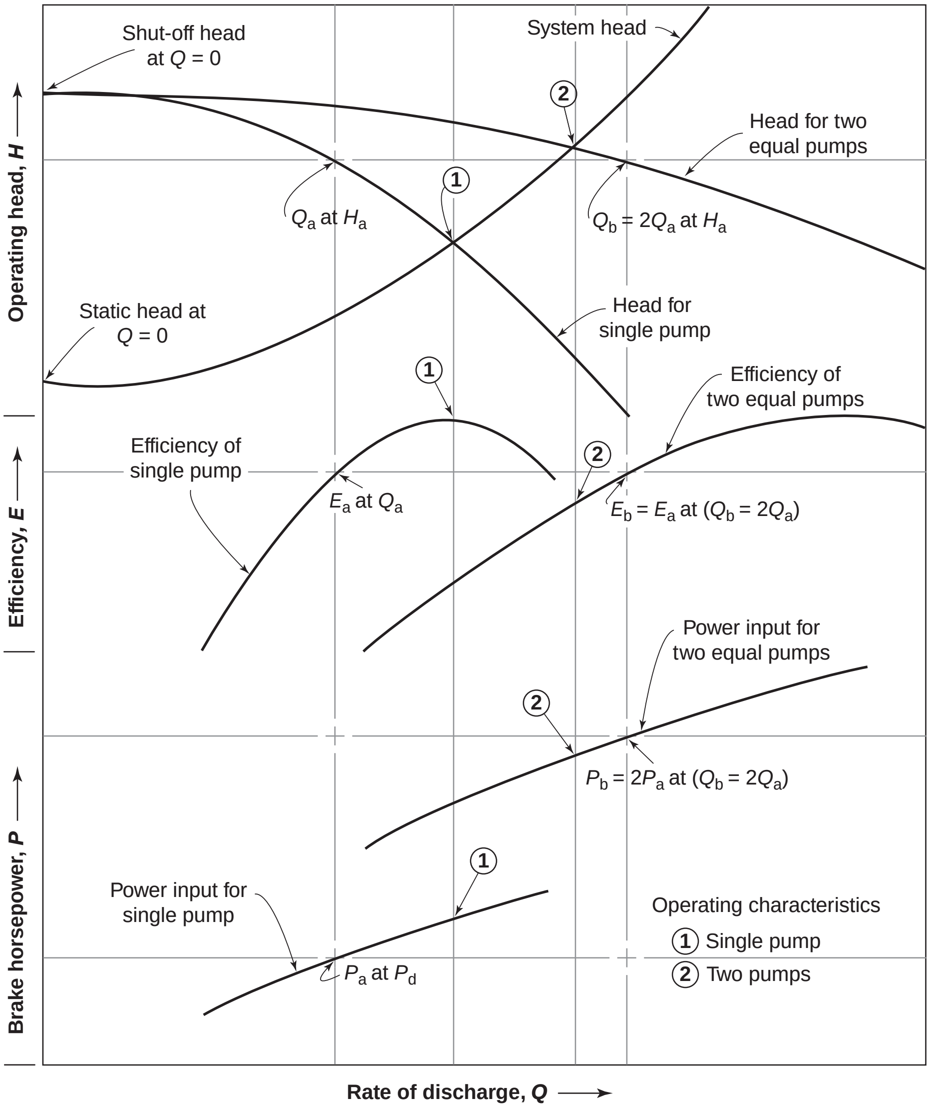

Figure 8.4 Performance characteristics of single and twin centrifugal pumps operating at constant speed.

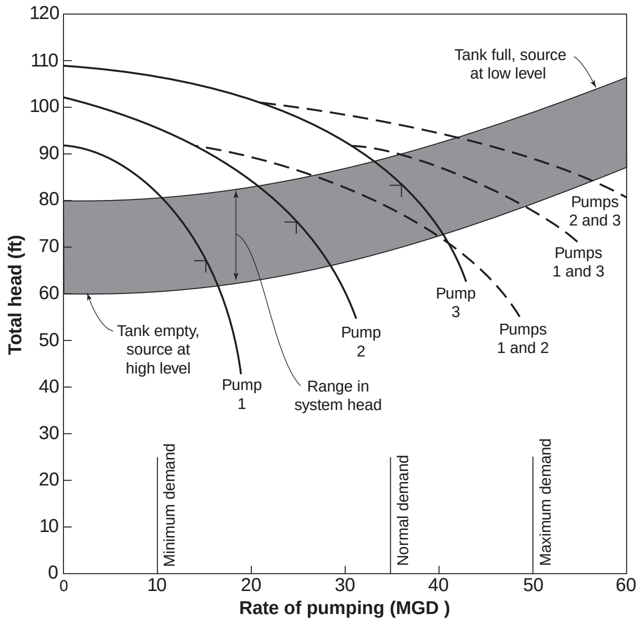

Figure 8.5 Pump selection in Example 8.4. Conversion factors: 1 ft = 0.3048 m; 1 MGD = 3.785 MLD = 0.0438 m^3/s = 43.8 L/s.

the reservoir is empty and the water surface of the source is high and (b) an *upper* curve establishes the system head for a full reservoir and a low water level at the source. The location and magnitude of drafts also influence system heads. Nighttime pressure distributions may be very different from those during the day. How the characteristic curves for twin-unit operation are developed is indicated in Fig. 8.4. Note that the two identical pumping units have not been selected with an eye to highest efficiency of operation in parallel. Characteristic curves for other multiple units are developed in the same way from the known curves of individual units.

Where most of the operating head is static lift—when the water is pumped through relatively short lengths of suction and discharge piping, for example—there is little change in the system head at different rates of flow. In these circumstances, the head curve is nearly horizontal, and the discharge of parallel pumps is substantially additive. This is common in wastewater pumping stations in which the flow is lifted from a lower to an immediately adjacent higher level. Examples are pumping stations along interceptors or at outfalls.

By contrast, friction may control the head on pumps discharging through long force mains, and it may not be feasible to subdivide flows between pumping units with reasonable efficiency. Multispeed motors or different combinations of pumps and motors may then be required.

Because flows from a number of pumps may have to be fed through a different piping system than flows from any single unit, it may be necessary to develop "modified" characteristic curves that account for losses in different combinations of piping.

Centrifugal pumps are normally operated with discharge velocities of 5–15 ft/s (1.5–4.6 m/s). The resulting average outlet diameter of the pump, called the pump size, can be determined by the following two equations:

$$D_{\text{podin}} = 0.2\sqrt{Q} \qquad \text{(US customary units)} \qquad \textbf{(8.18a)}$$

where D_{podin} is the pump outlet diameter, in.; and Q is the capacity of the pump, gpm; and

$$D_{\text{podcm}} = 63.95\sqrt{Q} \qquad \text{(SI units)} \qquad \textbf{(8.18b)}$$

where Q is the capacity of the pump, m^3/s; and D_{podcm} is the pump outlet diameter, cm. For instance, when the capacity of the pump is 200 gpm (0.01262 m^3/s = 12.62 L/s), the outlet diameter of the pump should be 2.8284 in. (7.1841 cm). An engineer may select 3 in. (75 mm) as the outlet diameter.

EXAMPLE 8.4 SELECTION OF PUMPS COMBINATION TO SATISFY WATER DEMAND

A mill supply drawing relatively large quantities of water from a river is to deliver them at a fairly low head. The minimum demand is 10 MGD (0.438 m^3/s), the normal 35 MGD (1.53 m^3/s), and the maximum 50 MGD (2.19 m^3/s). The river fluctuates in level by 5 ft (1.52 m), and the working range of a balancing tank is 15 ft (4.57 m). The vertical distance between the bottom of the tank and the surface of the river at its high stage is 60 ft (18.3 m). The friction head in the pumping station and a 54 in. (76.2 cm) force main rises from a minimum of 1 ft (0.30 m) at the 10 MGD (0.438 m^3/s) rate to a maximum of nearly 20 ft (6.1 m) at the 50 MGD (2.19 m^3/s) rate. Make a study of suitable pumping units, knowing that 1 MGD = 3.785 MLD = 0.0438 m^3/s = 43.8 L/s.

Solution:

The solution to this problem is shown in Fig. 8.5. Three pumps are provided: no. 1 with a capacity of 15 MGD (0.657 m^3/s = 657 L/s) at 66 ft (20.117 m) head; no. 2 with 25 MGD (1.059 m^3/s = 1059 L/s) at 78 ft (23.774 m) head; and no. 3 with 37 MGD (1.62 m^3/s = 1620 L/s) at 84 ft (25.6 m) head. Each pump has an efficiency of 89% at the design point. The efficiencies at the top and bottom of the working range are listed in Table 8.1.

Table 8.1 Pumping characteristics of system in Example 8.4

Pumps in service, no.	1	2	3	1 and 2	1 and 3	2 and 3
Rate of pumping MGD (m^3/s)	10 (0.44)	21 (0.92)	33.5 (1.47)	27 (1.18)	36 (1.58)	42 (1.84)
Head ft (m)	81 (24.7)	83 (25.3)	88 (26.8)	85 (25.9)	90 (27.4)	93 (28.3)
Efficiency (%)	80 (80)	88 (88)	88 (88)	71, 86 (71, 86)	35, 87 (35, 87)	68, 64 (68, 64)
Rate of pumping MGD (m^3/s)	15 (0.66)	25 (1.10)	37 (1.62)	34 (1.49)	43.5 (1.91)	49.5 (2.17)
Head ft (m)	66 (20.1)	78 (23.8)	84 (25.6)	80 (24.4)	85 (25.9)	89 (27.1)
Efficiency (%)	89 (89)	89 (89)	89 (89)	82, 88 (82, 88)	71, 89 (71, 89)	79, 87 (79, 87)
Rate of pumping MGD (m^3/s)	16.5 (0.72)	28.5 (1.25)	40.5 (1.77)	40 (1.75)	49.5 (2.17)	56.5 (2.47)
Head ft (m)	62 (18.9)	66 (20.1)	73 (22.3)	73 (22.3)	79 (24.1)	84 (25.6)
Efficiency (%)	88 (88)	84 (84)	86 (86)	88, 88 (88, 88)	83, 88 (83, 88)	79, 89 (79, 89)

Conversion factors: 1 MGD = 0.0438 m^3/s = 43.8 L/s; 1 ft = 0.3048 m.

8.3 SERVICE STORAGE

The three major components of service storage are as follows:

1. Equalizing, or operating, storage
2. Fire reserve
3. Emergency reserve

8.3.1 Equalizing, or Operating, Storage

Required *equalizing*, or *operating*, *storage* can be read from a *demand rate curve* or, more satisfactorily, from a *mass diagram*. As shown in Fig. 8.6 for the simple conditions of steady inflow, during 12 and 24 h, respectively, the amount of equalizing, or operating, storage is the sum of the maximum ordinates between the demand and supply lines. To construct such a mass diagram, proceed as follows:

1. From past measurements of flow, determine the *draft* during each hour of the day and night for typical days (maximum, average, and minimum).
2. Calculate the amounts of water drawn up to certain times, that is, the *cumulative draft*.
3. Plot the cumulative draft against time.
4. For steady supply during 24 h, draw a straight line diagonally across the diagram, as in Fig. 8.6a. Read the storage required as the sum of the two maximum ordinates between the draft and the supply line.
5. For steady supply during 12 h (e.g., by pumping) draw a straight line diagonally from the beginning of the pumping period to its end—for example, from 6 a.m. to 6 p.m., as in Fig. 8.6b. Again read the storage required as the sum of the two maximum ordinates.

Achieving a steady supply at the rate of maximum daily use will ordinarily require an equalizing storage between 15% and 20% of the day's consumption. Limitation of supply to 12 h may raise the operating storage to an amount between 30% and 50% of the day's consumption.

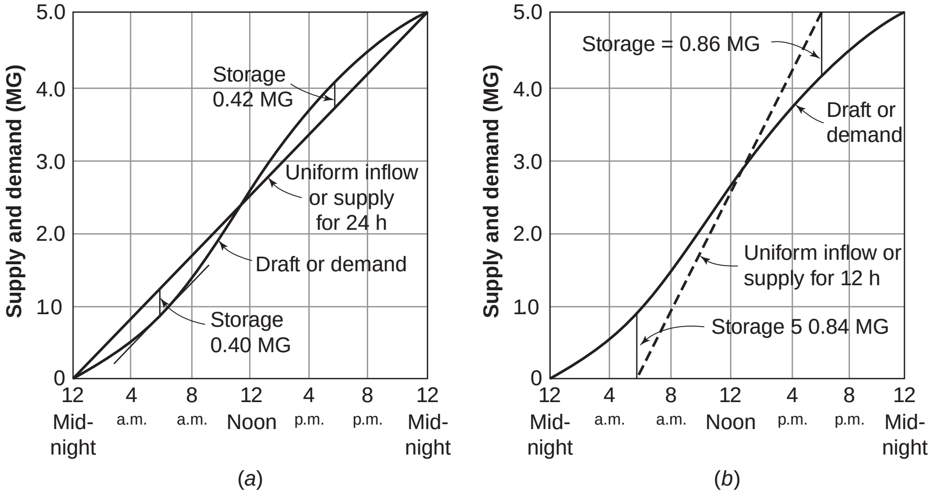

Figure 8.6 Determination of equalizing, or operating, storage by mass diagram; see Example 8.5. Uniform inflow, or supply, (a) extending over 24 h; (b) confined to 12 h. Total storage = 0.40 + 0.42 = 0.82 MG; (b) total storage = 0.84 + 0.86 = 1.70 MG. Conversion factor: 1 MG = 1 million gallons = 3.785 ML = 3,785 m^3.

EXAMPLE 8.5 DETERMINATION OF EQUALIZING STORAGE

Determine the equalizing, or operating, storage for the drafts of water shown in Table 8.2 (a) when inflow is uniform during 24 h and (b) when flow is confined to 12 h from 6 a.m. to 6 p.m.

Table 8.2 Observed drafts for Example 8.5

(*a*) Time	4 a.m.	8 a.m.	Noon	4 p.m.	8 p.m.	Midnight
(*b*) Draft MG (m^3)	0.484 (1,832)	0.874 (3,308)	1.216 (4,603)	1.102 (4,171)	0.818 (3,096)	0.506 (1,915)
(*c*) Cumulative draft MG (m^3)	0.484 (1,832)	1.358 (5,140)	2.574 (9.743)	3.676 (13,914)	4.494 (17,010)	5.000 (18,925)

Conversion factors: 1 MG = 1 million gallons = 3.785 ML = 3,785 m^3.

Solution 1 (US Customary System):

a. For steady supply during 24 h, the draft plotted in Fig. 8.6a exceeds the demand by 0.40 MG by 6 a.m. If this excess is stored, it is used up by 11 a.m. In the afternoon, the demand exceeds the supply by 0.42 MG by 6 p.m. and must be drawn from storage that is replenished by midnight. Hence, the required storage is the sum of the morning excess and afternoon deficiency, or **0.82 MG**. This storage volume as a percentage of total draft equals (0.82 MG/5 MG) × 100 = 16.4%.

b. For steady supply during the 12-h period from 6 a.m. to 6 p.m., the draft plotted in Fig. 8.6b exceeds the supply by 0.84 MG between midnight and 6 a.m. and must be drawn from storage. In the afternoon, the supply exceeds the demand by 0.86 MG by 6 p.m., but this excess is required to furnish water from storage between 6 p.m. and midnight. Therefore, total storage equals 0.84 + 0.86 = **1.70 MG**, or (1.7 MG/5 MG) × 100 = 34% of the day's consumption.

Solution 2 (SI System):

a. For steady supply during 24 h, the draft plotted in Fig. 8.6a exceeds the demand by 0.40 MG (1,514 m^3) by 6 a.m. If this excess is stored, it is used up by 11 a.m. In the afternoon, the demand exceeds the supply by 0.42 MG (1,590 m^3) by 6 p.m. and must be drawn from storage that is replenished by midnight. Hence, the required storage is the sum of the morning excess and afternoon deficiency, or 0.82 MG **(3,104 m^3)**. This storage volume as a percentage of total draft equals $[(3{,}104\ m^3)/(5 \times 3{,}785\ m^3)] \times 100 = 16.4\%$, where $5 \times 3{,}785\ m^3 = 5$ MG.

b. For steady supply during the 12-h period from 6 a.m. to 6 p.m., the draft plotted in Fig. 8.6b exceeds the supply by 0.84 MG (3,180 m^3) between midnight and 6 a.m. and must be drawn from storage. In the afternoon, the supply exceeds the demand by 0.86 MG (3,255 m^3) by 6 p.m., but this excess is required to furnish water from storage between 6 p.m. and midnight. Therefore, total storage equals 3,180 + 3,255 = **6,435 m^3**, or $[(6{,}435)/(5 \times 3{,}785)] \times 100 = 34\%$ of the day's consumption.

8.3.2 Fire Reserve

Based on IFC recommendations on observed durations of serious conflagrations, it is recommended that distributing reservoirs be made large enough to supply water for fighting a serious conflagration for (a) 4 h for fire flows of more than 4,000 gpm (252 L/s), (b) 3 h for fire flows of 3,000–3,750 gpm (189–237 L/s), and (c) 2 h for fire flows of 2,750 gpm (174 L/s) and less (see Table 4.13). The resulting fire reserve may not always be economically attainable, and design values may have to be adjusted downward to meet local financial abilities. Changing community patterns, moreover, may make for changing requirements in the future.

8.3.3 Emergency Reserve

The magnitude of this storage component depends on (a) the danger of interruption of reservoir inflow by failure of supply works and (b) the time needed to make repairs. If shutdown of the supply is confined to the time necessary for routine inspections during the hours of minimum draft, the emergency reserve is sometimes made no more than 25% of the total storage capacity, that is, the reservoir is assumed to be drawn down by one-fourth its average depth. If supply lines or equipment are expected to be out of operation for longer times, higher allowances must be made.

8.3.4 Total Storage

The desirable total amount of storage is equal to the sum of the component requirements. In each instance, economic considerations dictate the final choice. In pumped supplies, cost of storage must be balanced against cost of pumping, and attention must be paid to economies affected by operating pumps more uniformly and restricting pumping to a portion of the day only. In all supplies, cost of storage must be balanced against cost of supply lines, increased fire protection, and more uniform pressures in the distribution system.

Storage facilities should have sufficient capacity, as determined from engineering studies, to meet domestic demands and, where fire protection is provided, fire flow demands. The recommended standards for sizing a water storage tank or reservoir as stated in the *Recommended Standards for Water Works, 2007 Edition* (Health Research, 2007; often referred to as the Ten-States Standards), are as follows:

1. Fire flow requirements established by the appropriate state insurance services office should be satisfied where fire protection is provided.
2. The minimum storage capacity (or equivalent capacity) for systems not providing fire protection shall be equal to the average daily consumption. This requirement may be reduced when the source and treatment facilities have sufficient capacity with standby power to supplement peak demands of the system.
3. Excessive storage capacity should be avoided to prevent potential water quality deterioration problems.

EXAMPLE 8.6 TOTAL WATER STORAGE VOLUME

For a steady gravity supply equal to the maximum daily demand, a 4-h fire supply at 8,000 gpm ($0.505\ \text{m}^3/\text{s} = 505\ \text{L/s}$), and no particular hazard to the supply works, find the storage to be provided for a city using an average of 7.5 MGD ($0.328\ \text{m}^3/\text{s} = 328\ \text{L/s}$) of water.

Solution 1 (US Customary System):

The equalizing storage is 15% of average daily consumption:

$$0.15 \times 7.5\ \text{MG} = 1.13\ \text{MG}.$$

The fire reserve is 8,000 gpm for 4 h:

$$(8{,}000\ \text{gal/min} \times 4\ \text{h} \times 60\ \text{min/h})/10^6 = 1.92\ \text{MG}.$$

The resulting subtotal is 1.13 + 1.92 = 3.05 MG.

Because the emergency reserve is one-fourth of the total storage, the subtotal is three-fourths (0.75) of the total storage. Therefore,

$$\text{Total storage} = 3.05/0.75 = \mathbf{4.1\ MG}.$$

Solution 2 (SI System):

$$0.328\ \text{m}^3/\text{s} = 0.328 \times 24 \times 60 \times 60\ \text{m}^3/\text{d} = 28{,}387\ \text{m}^3/\text{d}.$$

The equalizing storage is 15% of average daily consumption:

$$0.15 \times 28{,}387\ \text{m}^3 = 4{,}258\ \text{m}^3.$$

The fire reserve is $0.505\ \text{m}^3/\text{s}$ for 4 h:

$$(0.505\ \text{m}^3/\text{s})(4 \times 60 \times 60\ \text{s}) = 7{,}269\ \text{m}^3.$$

The resulting subtotal is $4{,}258 + 7{,}269 = 11{,}527\ \text{m}^3$.

Because the emergency reserve is one-fourth of the total storage, the subtotal is three-fourths (0.75) of the total storage. Therefore,

$$\text{Total storage} = 11{,}527/0.75 = \mathbf{15{,}369\ m^3}.$$

EXAMPLE 8.7 TOTAL WATER STORAGE VOLUME USING THE TEN-STATES STANDARDS

For a steady gravity supply equal to the maximum daily demand, a 4-h fire supply at 8,000 gpm ($0.505\ \text{m}^3/\text{s} = 505\ \text{L/s}$), and no particular hazard to the supply works, find the storage to be provided for a city using an average of 7.5 MGD ($0.328\ \text{m}^3/\text{s} = 328\ \text{L/s}$) of water using the Ten-States Standards.

(*Note:* This example uses the same parameters as Example 8.6.)

Solution 1 (US Customary System):

Storage volume for domestic consumption = average daily consumption (Ten-States Standards):

$$= 7.5\ \text{MGD}$$

$$= 7.5\ \text{MG daily}.$$

The fire reserve is 8,000 gpm for 4 h (similar to Example 8.6):

$$= (8{,}000\ \text{gal/min}\ \times 4\ \text{h} \times 60\ \text{min/h})/10^6$$
$$= 1.92\ \text{MG}.$$

$$\textbf{Total Storage} = \text{domestic storage} + \text{fire strage}$$
$$= 7.5 + 1.92 = \mathbf{9.42\ MG}.$$

Solution 2 (SI System):

Storage volume for domestic consumption = average daily consumption (Ten-States Standards):

$$= 0.328\ \text{m}^3/\text{s} = 328\ \text{L/s}$$
$$= 28{,}387\ \text{m}^3\ \text{daily}.$$

The fire reserve is 0.505 m^3/s for 4 h (similar to Example 8.6):

$$= (0.505\ \text{m}^3/\text{s})(4 \times 60 \times 60\ \text{s})$$
$$= 7{,}269\ \text{m}^3.$$

$$\textbf{Total Storage} = \text{domestic storage} + \text{fire strage}$$
$$= 28{,}387 + 7{,}269 = \mathbf{35{,}656\ m^3}.$$

8.4 LOCATION OF STORAGE

In addition to capacity of service storage, location is an important factor in the control of distribution systems. For example, 1 MGD (3.785 MLD) of elevated fire reserve, suitably sited in reference to the area to be protected, is equivalent to the addition of a 12 in. (300 mm) supply main. The underlying reasoning is that, when drawing this volume of water in a 4-h fire, flow is provided at a rate of $(24/4) \times 1$ MGD = 6 MGD or (24/4)(3.785 MLD) = 22.7 MLD. This is the amount of water an 18 in. (450 mm) pipe can carry at a velocity of less than 5 ft/s (1.5 m/s). Why this must be neighborhood storage is explained by the high frictional resistance of more than 8% accompanying such use.

The engineering considerations for deciding the location of water supply storage tanks or reservoirs are as follows:

1. Consideration should be given to maintaining water quality when locating water storage facilities.
2. The bottom of ground-level reservoirs and standpipes should be placed at the normal ground surface and shall be above the 100-year flood level or the highest flood of record.
3. If the bottom elevation of a storage reservoir must be below normal ground surface, it shall be placed above the groundwater table. At least 50% of the water depth should be above grade. Sewers, drains, standing water, and similar sources of possible contamination must be kept at least 50 ft (15 m) from the reservoir. Gravity sewers constructed of water main quality pipe, pressure tested in place without leakage, may be used at distances greater than 20 ft (6 m) but less than 50 ft (15 m).
4. The top of a partially buried storage structure shall not be less than 2 ft (0.6 m) above normal ground surface. Clearwells constructed under filters may be exempted from this requirement when the design provides adequate protection from contamination.

8.5 ELEVATION OF STORAGE

Storage reservoirs and tanks operate as integral parts of the system of pumps, pipes, and connected loads. In operation all the parts respond to pressure changes as the system follows the diurnal and seasonal demands. Ideally the storage elevation should be such that the reservoir "floats" on the system, neither emptying nor standing continuously full. In systems with inadequate pipes or pumps, or having a storage reservoir that is too high, the hydraulic gradient may at times of peak demand fall below the bottom of the reservoir. When this occurs, the full load falls on the pumps, and system pressures deteriorate suddenly.

8.6 TYPES OF DISTRIBUTING RESERVOIRS

Where topography and geology permit, service reservoirs are formed by impoundage, balanced excavation and

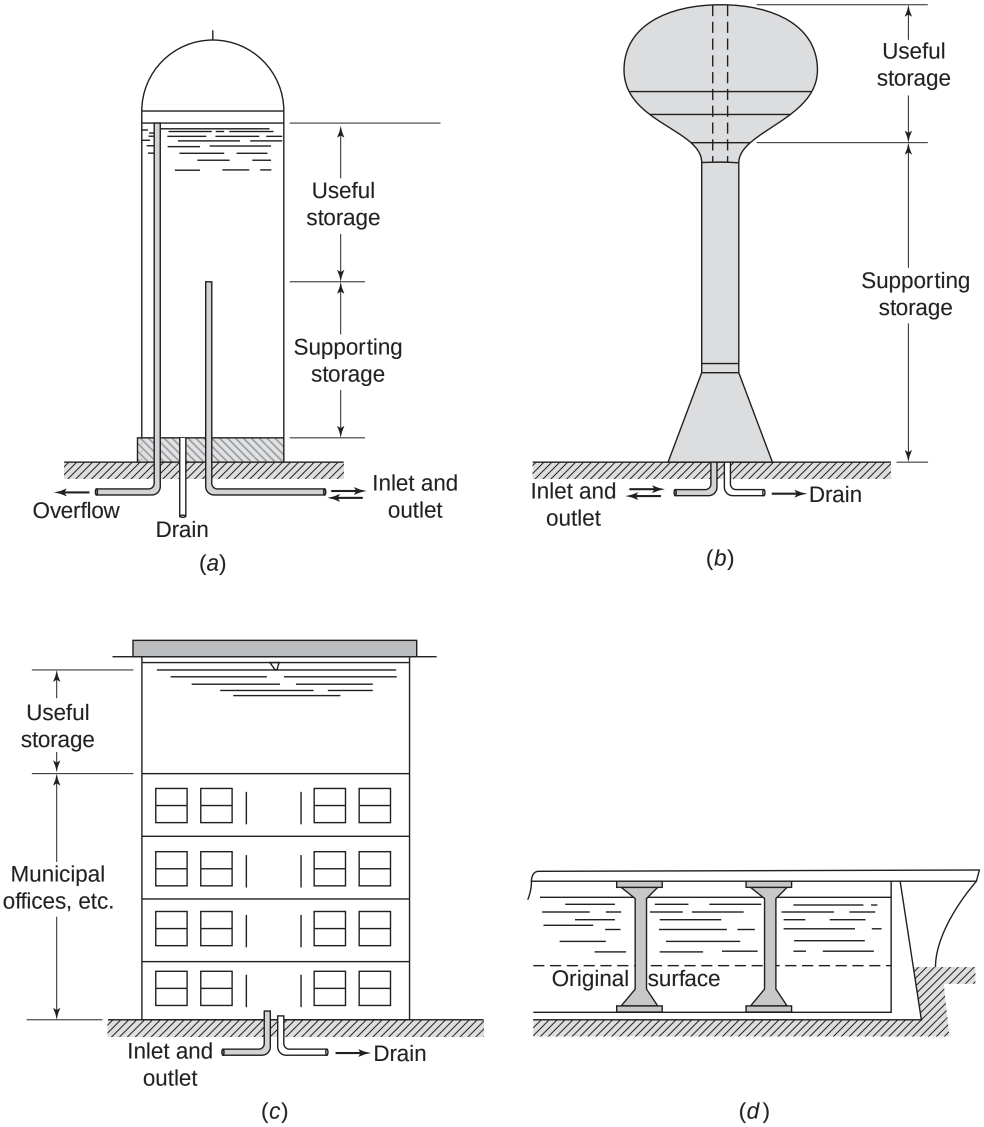

Figure 8.7 Types of service reservoirs: (a) standpipe; (b) and (c) elevated tanks; (d) ground-level service reservoir.

embankment, or masonry construction (Fig. 8.7). To protect the water against chance contamination and against deterioration by algal growths stimulated by sunlight, distributing reservoirs should be covered. Roofs need not be watertight if the reservoir is fenced. Open reservoirs should always be fenced. Where surface runoff might drain into them, they should have a marginal intercepting conduit.

Earthen reservoirs, their bottom sealed by a blanket of clay or rubble masonry and their sides by core walls, were widely employed at one time. Today, lining with concrete slabs is more common. Gunite, a sand–cement–water mixture, discharged from a nozzle or gun through and onto a mat of reinforcing steel, has also been employed to line or reline them. Plastic sheets protected by a layer of earth have also been used to build inexpensive but watertight storage basins. Roofs are made of wood or concrete. Beam and girder, flat-slab, arch, and groined-arch construction have been used. Where concrete roofs can be covered with earth, both roof and water will be protected against extremes of temperature.

Inlets, outlets, and overflows are generally placed in a gate house or two. Circulation to ensure more or less continuous displacement of the water and to provide proper detention of water after chlorination may be controlled by *baffles* or subdivisions between inlet and outlet.

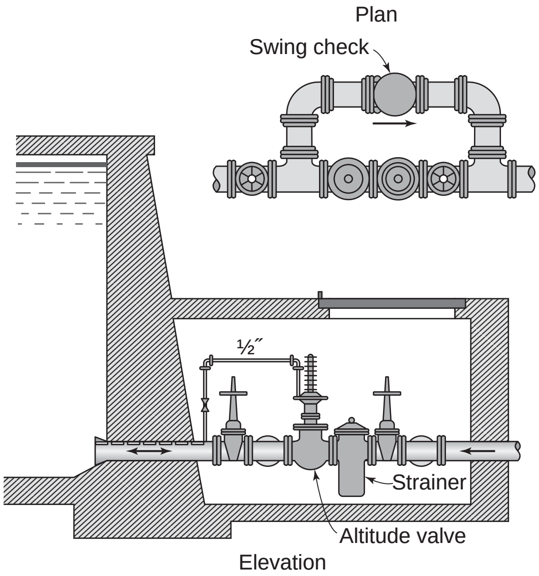

Figure 8.8 Altitude valve on supply to distribution reservoir.

Overflow capacity should equal the maximum rate of inflow. *Altitude-control valves* on reservoir inlets (Fig. 8.8) will automatically shut off inflow when the maximum water level is reached. An arrangement that does not interfere with draft from the reservoir includes a bypass with a *swing check valve* seating against the inflow.

Where natural elevation is not high enough, water is stored in concrete or steel *standpipes* and *elevated tanks*. In cold climates, steel is most suitable. Unless the steel in reinforced concrete tanks is prestressed, vertical cracks, leakage, and freezing will cause rapid deterioration of the structure. Ground-level storage in reinforced concrete or steel tanks in advance of automatic pumping stations is an alternative.

The useful capacity of standpipes and elevated tanks is confined to the volume of water stored above the level of wanted distribution pressure. In elevated tanks, this level generally coincides with the tank bottom; in standpipes, it may lie much higher. Steel tanks are welded or riveted. Their structural design and erection have become a specialized activity of certain tank manufacturers.

The function of elevated tanks and *spheroidal tanks* can be expressed to aesthetic advantage in their architecture without resorting to ornamentation (Figs. 8.9 and 8.10). *Standpipes* are simple cylinders. A veneer or outer shell of concrete or masonry may make them attractive. They may be designed as parts of multipurpose structures. The lower level may serve for offices, warehouses, or other functions. At the top, sightseeing or restaurant facilities may convert a potential eyesore into a center of attraction.

Figure 8.9 Water storage tank (*Source*: http://en.wikipedia.org/wiki/Image:Louisville_water_tower.jpg).

Figure 8.10 The mushroom-shaped concrete water tower in Helsinki, Finland. It is 52 m high and can hold 12,000 m^3 of water (*Source*: http://en.wikipedia.org/wiki/Water_supply_network).

EXAMPLE 8.8 PUMPING AND STORAGE SYSTEM

Draw a sketch, showing a system (including the raw water reservoir, pump station, water transmission lines, elevated water storage tank, hydraulic grade line, and elevation in ft or m) based on the following information: the raw water reservoir of a city is at an elevation of 500 ft (152.4 m). An automatic booster pumping station is proposed, having for its control point a 550,000 gal (2,082 m^3) water storage tank. The design tank water level is at an elevation of 490 ft (149.4 m).

Measurement of the water storage tank level is to be transmitted to the pumping station and is to be the only variable used for the control of the pumps in the pumping station. The head losses of the water transmission line between the raw water reservoir and the elevated water storage tank (including the miscellaneous head losses in the proposed pumping station) computed for the rate of flow of 1,600 gpm (101 L/s) are 40 ft (12.2 m), which is to be overcome by the pumps. A chlorination system is used for treating the water entering the elevated water storage tank. The chlorinated water from the tank is transmitted to the city's water filtration plant for further treatment prior to domestic consumption.

Solution:

Figure 8.11 shows a sketch of the required water supply system.

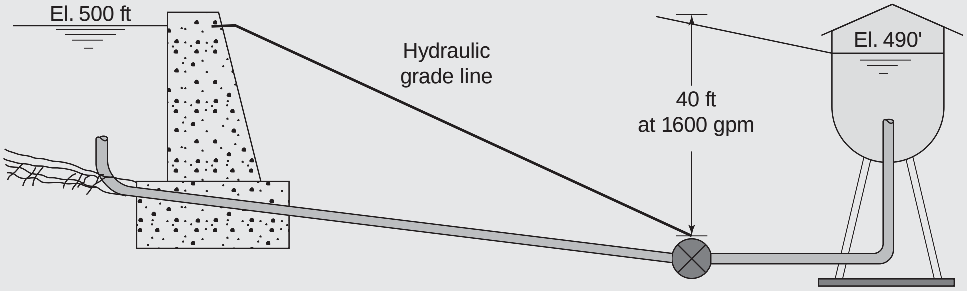

Figure 8.11 Sketch of water supply system for Example 8.8.
Conversion factors: 1′ = ft = 0.3048 m; 1 gpm = 3.785 L/min = 0.0000631 m^3/s = 0.0631 L/s.

EXAMPLE 8.9 PUMPS AND WATER STORAGE TANKS

The water supply system shown in Example 8.8 provides water to a city at the rates of 2.0 MGD (88 L/s) for average daily water demand, 3.4 MGD (150 L/s) for maximum daily water demand, and 4.8 MGD (210 L/s) for peak hourly water demand. The total head losses are 40 ft (12.2 m) at a flow of 1,600 gpm (101 L/s).

What are the head losses of the transmission line between the raw water reservoir and the elevated water storage tank (including all miscellaneous head losses), which must be overcome by the pumps at 2, 3.4, and 4.8 MGD (88, 150, and 210 L/s; or 7.6, 12.9, and 18.2 MLD)? Explain which water demand is used for the design of a pumping station between the raw water reservoir and the water storage tank, and why. Is the size of the existing water storage tank sufficient for the city in accordance with the current Ten-States Standards for Water Works?

The following equation shows the relationship between the head loss and the water flow:

$$H_2 = H_1(Q_2/Q_1)^2$$

where

Q_1 = the first water flow, gpm (m^3/s)
Q_2 = the second water flow, gpm (m^3/s)
H_1 = head loss at flow of Q_1, ft (m)
H_2 = head loss at flow of Q_2, ft (m)

Solution 1 (US Customary System):

Head loss at average daily water demand of 2 MGD (or 1388 gpm):

$$H_2 = (40)(1{,}388/1{,}600)^2 = \mathbf{30\ ft}.$$

Head loss at maximum daily water demand of 3.4 MGD (or 2,360 gpm):

$$H_2 = (40)(2{,}360/1{,}600)^2 = \mathbf{87\ ft}.$$

Head loss at maximum hourly water demand of 4.8 MGD (or 3,331 gpm):

$$H_2 = (40)(3{,}331/1{,}600)^2 = \mathbf{173\ ft}.$$

Solution 2 (SI System):

Head loss at average daily water demand of 7.6 MLD (or 0.088 m^3/s, or 88 L/s):

$$H_2 = (12.2)(0.088/0.101)^2 = \mathbf{9.36\ m}.$$

Head loss at maximum daily water demand of 12.9 MLD (or 0.150 m^3/s, or 150 L/s):

$$H_2 = (12.2)(0.150/0.101)^2 = \mathbf{27.00\ m}.$$

Head loss at maximum hourly water demand of 18.2 MLD (or 0.210 m^3/s, or 210 L/s):

$$H_2 = (12.2)(0.210/0.101)^2 = \mathbf{53.80\ m}.$$

The system will not be economically feasible if the maximum hourly water demand is used for design of a pump station due to its related high head loss (173 ft or 52.73 m). Normally the **maximum daily water demand is used for designing a pump station**. The peak hourly water demand of the city will be provided by the elevated water storage tank.

According to the latest edition of the Ten-States Standards, the minimum water storage capacity for water systems not providing fire protection shall be equal to the average daily consumption, or 2 MG (7,570 m^3). **A new water storage tank (2 MG or 7,570 m^3 + fire demand) is needed** because the existing water storage tank (550,000 gal or 2,082 m^3) is not big enough.

EXAMPLE 8.10 DESIGN OF A PUMPING STATION

Design a pump station for the water system examined in Examples 8.8 and 8.9. Double-suction, horizontal centrifugal pumps driven by AC electric motors are considered the best application for this project. Select pumps having rated speeds of 1,750, 1,150, or 870 rpm for this pump station using assumed pump characteristics.

Recommend the pump capacity, the number of pumps, and pumping mode (parallel operation or series operation). Calculate the effective head of the selected pump, its water horsepower, brake horsepower (horsepower input to each pump), and motor horsepower assuming the pump efficiency is 80% and the motor efficiency is 90%. Show the assumed capacity–efficiency curve, the head–capacity curve, and the BHP–capacity curve (or the BMP–capacity curve) on a sketch, and then indicate on the sketch the rated points of the pumps. Write brief engineering conclusions stating the following: (a) the number of installed pump units and the type of motor driving each pump unit; (b) the method, if any, of capacity control that an engineer would propose and the probable overall pump station efficiency at the average daily water consumption; and (c) the probable horsepower of the motor driving each unit.

Solution 1 (US Customary System):

Average daily consumption = 2 MGD = 1,388 gpm.

Maximum daily consumption = 3.4 MGD = 2,360 gpm.

Peak hourly demand = 4.8 MGD = 3,331 gpm.

With sufficient water storage capacity available in the future, two pumps of equal capacity should be operating in parallel to supply the maximum daily consumption of 2,360 gpm. Each pump is to supply 1,215 gpm, and both pumps 2,430 gpm.

Head loss at 2,360 gpm, $H = 40\,(2{,}360/1{,}600)^2 = 87$ ft.

The effective **head of each pump** should be 87 ft less the difference in water surface elevations.

Difference in water surface elevation = 500 ft − 490 ft = 10 ft.

Effective head of each pump, $H = 87\text{ ft} - 10\text{ ft} = \mathbf{77\ ft}$.

Water horsepower (WHP) = $(QH)/3{,}957$
$= (1{,}215)(77)/3{,}957 = \mathbf{23.6\ hp}$.

Brake horsepower (BHP) = horsepower input to each pump = $\text{WHP}/E_{\text{pump}}$
$= 23.6/0.8 = \mathbf{29.5\ hp}$.

Motor horsepower (MHP) = horsepower input to motor = $\text{BHP}/E_{\text{motor}}$
$= 29.5\text{ hp}/0.9 = \mathbf{32.8\ hp}$.

Solution 2 (SI System):

Average daily consumption = 7.57 MLD = 0.088 m^3/s = 88 L/s.

Maximum daily consumption = 12.87 MLD = 0.150 m^3/s = 150 L/s.

Peak hourly demand = 18.17 MLD = 0.210 m^3/s.

With sufficient water storage capacity available in the future, two pumps of equal capacity should be operating in parallel to supply the maximum daily consumption of = 12.87 MLD = 0.150 m^3/s = 150 L/s.

Each pump is to supply 0.077 m^3/s, and both pumps 0.153 m^3/s.

Head loss, at 12.87 MLD or 0.150 m^3/s, $H = (12.19 \text{ m})(0.150 \text{ m}^3/\text{s}/0.101 \text{ m}^3/\text{s})^2 = 26.5$ m.

The effective **head of each pump** should be 26.5 m (87 ft) less the difference in water surface elevations.

Difference in water surface elevation = 152.4 m − 149.3 m = 3.1 m.

Effective head of each pump, H = 26.5 m − 3.1 m = 23.4 m.

Water power (WMP) = 9.8066(QH)
= 9.8066(0.077)(23.5) = **17.7 kW**.

Brake power (BMP) = power input to each pump = WMP/E_{pump}
= (17.7)/0.8 = **22.2 kW**.

Motor power (MMP) = power input to motor = BMP/E_{motor}
= 22.2/0.9 = **24.6 kW**.

The sketch of the selected centrifugal pump will have the pump characteristic curves shown in Fig. 8.12.

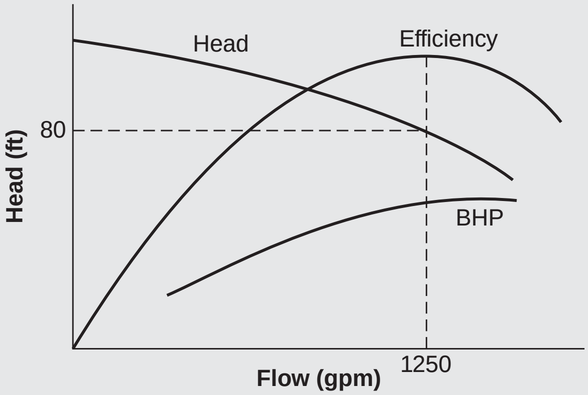

Figure 8.12 Pump characteristic curves for Example 8.10. Conversion factors: 1 ft = 0.3048 m; 1 gpm = 3.785 L/min = 0.0000631 m^3/s = 0.0631 L/s.

The engineer's recommendations are as follows:

a. Install three pump units, each with a capacity of 1,250 gpm (79 L/s) at a total developed head of 80 ft (24.38 m) at full capacity. Two of the pumps should have standard 60-cycle, AC induction-motor drives, 1,750 rpm. The third pump to be held in reserve in case of electric power failure or shutdown of one electric-drive pump for repairs may be driven with a gasoline engine, in case an emergency power generator is not available. If an emergency power generator is available, the third pump can be identical to the selected two pumps.

b. The storage tank capacity (550,000 gal or 2,082 m^3) is normally sufficient to meet the maximum hourly demand and fluctuation during maximum daily demand. However, the existing storage tank capacity is insufficient according to the latest edition (2007) of the Ten-States Standards for water works. Expansion of the water storage tank is recommended if construction funds are available. The quantity of water in the water storage tank should be controlled by a floating control system. If the float drops below a specified level, the control system actuates the starting mechanism of the second pump to put it in parallel with the other pump. The pump station efficiency at average daily consumption will be approximately 50–60%.

c. Each pump must be driven by a 35 hp (26.1 kW), 1,750 rpm motor.

EXAMPLE 8.11 LEAKAGE LOSSES VERSUS SPECIFIC SPEED FOR DOUBLE-SUCTION PUMPS

Specific speed *S* is a dimensionless quantity used to characterize turbomachinery, and is of significant importance in selecting a pump for a specific application, since there is a relationship and internal leakage within the pump as shown in Table 8.3 for double-suction split case centrifugal pumps.

Table 8.3 Specific speed versus leakage losses for double-suction pumps in Example 8.11

Specific speed *S*	Leakage losses in % of power input (%)
500	9.5
1,000	4.3
1,500	2.6
2,000	1.8
2,500	1.4
3,000	1.2
3,500	1.0

Determine and discuss the specific speeds and the expected inherent losses in % of power input for two pumps:

1. Pump A: it has a 356-mm-(14-in.)-diameter impeller with its optimum efficiency at 0.2145 m^3/s (3,400 gpm) and 54.9 m (180 ft.) head when operating at 1,900 rpm.
2. Pump B: it has a 356-mm-(14-in.)-diameter impeller with its optimum efficiency at 0.0126 m^3/s (200 gpm.) and 79.2 m (260 ft.) head when operating at 3600 rpm.

Solution 1 (US Customary System):

For pump A:

$$S = NQ^{0.5}/H_{\text{sv}}^{0.75} = 1{,}900 \times (3{,}400)^{0.5}/180^{0.75} = \mathbf{2{,}255}.$$

Pump A would have an inherent loss of about **1.6%.**

For pump B:

$$S = 3{,}600 \times (200)^{0.5}/260^{0.75} = \mathbf{786}.$$

Pump B would have an inherent loss of about **6%.**

Solution 2 (SI System):

For pump A:

$$S = 51.7\ NQ^{0.5}/H_{\text{sv}}^{0.75} = 51.7 \times 1{,}900 \times (0.2145)^{0.5}/54.9^{0.75} = \mathbf{2{,}255}.$$

Pump A's expected inherent loss would be about **1.6%**.

For pump B:

$$S = 51.7 \times 3600 \times (0.0126)^{0.5}/79.2^{0.75} = \mathbf{786}.$$

Pump B's expected inherent loss would be about **6%**.

8.7 DUAL WATER SUPPLY SYSTEMS[1]

The term *dual water supply systems* refers to two water supply distribution systems in a city. One is a freshwater system for potable use; the other system is for lower quality water (seawater or untreated raw freshwater, or treated/reclaimed wastewater) for firefighting and toilet flushing purposes. In Hong Kong and Saudi Arabia, dual water supply systems have been in use for more than 40 years.

Water supply networks are sized so that they can provide the large flows needed for firefighting, but this creates potable water quality problems due to the long time water

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[1] Section 8.7 is based on a timely article written by Dr. Daniel A. Okun a few months before he passed away in 2007. The article appeared in the February 2007 issue of *Water21*, a magazine of the International Water Association, pages 47–49; reproduced courtesy of IWA Publishing.

can spend in the network. What is needed, argues Okun (2007), is a switch to dual systems, with potable supplies provided through a smaller bore network using a material such as stainless steel, allowing reclaimed water to be used in existing networks.

8.7.1 Background

In the eighteenth and early nineteenth centuries, the demand for protection against fire and the great conflagrations and loss of life that they brought predicated the provision of water distribution systems designed for fire protection. Only later were these distribution systems put into service for commercial and then residential use, which led to the subsequent development of the water closet and sewerage systems.

That our present distribution systems are delivering water of exceedingly poor quality today, almost without regard to the water's source, treatment, or distribution, has been made manifest by the vast literature emerging from every corner of the water supply scene. The American Water Works Association (AWWA) has shown considerable awareness of the problems. Its annual water quality technology conferences have each had more than 100 papers identifying the problems. Some 40 classes of problems are set out by AWWA, with recommendations for individual utilities to assess their own particular problems and find their own answers. But relatively few utilities have the appropriate staff or financial resources to undertake the required studies and address each of the many problems.

The recommended practice of frequent *flushing* of the distribution systems has been widely adopted, but it hardly addresses the problems. Flushing is costly in personnel and extremely wasteful of treated drinking water, which is discharged to stormwater sewerage systems. In addition, frequent flushing is not very effective in keeping the pipes free from *biofilm* growths on pipe walls and maintaining hydraulic capacity.

8.7.2 The Nature of the Problems with Drinking Water Quality

The critical problem is that fire protection requires there to be many hydrants throughout a city, which have to have the capacity to deliver relatively high flow rates at all times and at all locations throughout the area. Pipe sizes were initially a minimum of 6 in. (150 mm), but today this has increased in many communities to a minimum of 8 in. (200 mm).

Because fires are infrequent, the velocity of the water in the network is almost always slow, resulting in residence times of months between when the water is treated and when it arrives at the taps of consumers in the outer regions of the service area. Recent tracer studies by the University of North Carolina in two of the larger cities in the state revealed residence times of more than 10 days. Such times make adequate *chlorine residuals* at the tap unlikely.

The inadequacy of *disinfection,* with the resulting risk of microbial exposure at the tap, is not the most troublesome problem arising from ineffective disinfection. In attempting to provide adequate disinfection despite the poor conditions in the pipelines, providers considerably increase the *chlorine dose*, resulting in increased levels of *disinfection by-products (DBPs)* through reactions with both chemical and microbial contaminants in the water.

Trihalomethanes (THMs) and *haloacetic acids* (HAAS) are the only two DBPs that are being regulated by the US Environmental Protection Agency (U.S. EPA), but then with great difficulty. Their *maximum contaminant levels* (MCLs) were epidemiologically uncertain, as indicated by the arbitrary adoption of a THM level of a "round" 0.10 mg/L in 1979. This figure was reduced recently, based in large part on the ability of utilities to reach a lower level.

The DBP problem is much more difficult to manage than is evident from recent research. As shown later, many more contaminants are present in drinking water networks than are recognized today as potential reactants with the chlorine present in the water and therefore there are many more other DBPs that need to be regulated.

Even more concern for the health effects of THMs has arisen because of a study carried out on 50 women in two very different locations (Cobb County, GA, and Corpus Christi, TX), which have water supplies with very different *THM bromide* concentrations and disinfectant types: *chloroform* in the former and *brominated THMs* in the latter.

Blood samples were taken from women and water samples were taken from their showers in the early morning. It was shown that the THMs in their blood samples rose significantly after showering. The types of THMs in their blood samples matched the type of THM in the water. THM standards are based on lifetime exposures, but recent studies have suggested that THMs pose possible reproductive problems for women that would dictate more rigorous MCLs for DBPs in the future.

8.7.3 The Pipes in the Distribution Systems

Because the pipes in all urban water distribution systems currently need to be a minimum of 6 in. (150 mm) in diameter or larger, they are generally heavy cement-lined ductile iron pipes, each section 16 ft (5.3 m) in length. These require some 350 joints per mile of pipe, including those needed for fire hydrants. The pipes are laid on soil in trenches, and in time the joints leak and lose water. If the pipes are below the water table, any infiltration of contaminated groundwater would pose a health risk.

Because the pipes are always under pressure, it had been believed that contamination from surrounding groundwater would not be a problem. Recent studies, however, have revealed that sudden changes in the velocity of the water are created by the opening and closing of valves in the lines and the starting and stopping of pumps, events that occur several times a day. This causes negative pressure transients that result in the infiltration of groundwater from the soil in the vicinity of the pipes.

Such transients would, of course, also occur in small pipes, but stainless steel pipes used for distribution systems that only carry drinking water would not leak because these pipes do not have open joints. In the very small sizes, they can be laid from spools and in the larger sizes the pipe can be welded. Stainless steel is already widely used in Japan for water distribution systems. These pipes have an added advantage over cement-lined pipes because they are not prone to heavy growths of biofilms due to their smooth interior walls.

The fact that leakage from distribution systems increases with time is well recognized, but these losses are generally not considered important. However, recent studies show that leaks have much more serious consequences. The negative pressures that have been found to occur regularly in pipelines encourage infiltration of the water in the soil surrounding the pipes. This is possible because heavy pipes laid on soil tend to subside over time, which opens the joints sufficiently to create two-way leakage.

8.7.4 Biofilms and the Problems They Cause

The poor water quality found in distribution systems today results above all from the considerable growth of biofilms that are attracted to the insides of the pipes because of the pipe materials chosen and the long residence times of the water. The many joints, hydrants, valves, and other appurtenances, along with the cement and other conventional linings, present attractive surfaces for the growth of biota, which deplete the disinfectants.

The heavy growths in turn shield the disinfectant from the biota. The biofilms grow thick because of the very slow velocities of the water and the long residence times in the pipes. They build up to the point that, with products of corrosion and tuberculation, they significantly reduce the hydraulic capacity of the pipes. If water consumers were able to see the insides of most distribution systems, they would be encouraged to buy bottled water, now a fast-growing trend.

Studies in Britain have compared conventional cement-lined pipe with stainless steel pipes of various compositions to assess their comparative rates of growth of biofilms. All of the stainless steel pipes were found to be far better than the cement-lined pipe in terms of the rates of biofilm growth. The stainless steel pipes that are available in very small sizes are provided from spools that carry long lengths of product, sufficient to serve the outlying residential areas of larger cities without the problems that are now encountered. Larger stainless steel pipes are available in considerable lengths that can be welded, thus avoiding joints.

8.7.5 The Proposed System

The professionals engaged in providing drinking water to the public make great efforts to ensure water of high quality, which requires considerable investments in treating the water. However, in the last step, distribution of the water to the consumer, the water is permitted to be seriously degraded.

We should be embarrassed by the tremendous efforts and funds we invest in providing high-quality water only to allow it to be subjected to the many problems enumerated above, which result in increasing health risks to the public. In our efforts to address each of the problems, we oblige the water utilities to undertake studies and remedies far beyond the capacities of all but the very large utilities.

Just because we have inherited distribution systems created for fighting fires over two centuries, a practice that is responsible for all of the problems, that is no reason for continuing the practice into the future, especially when the only solution offered is frequent flushing of all the pipelines. Flushing is costly in execution, relatively ineffective, and wasteful of the treated drinking water.

A reasonable solution is available: distribution systems designed for drinking water alone. This option has many advantages. The pipe diameters for most of the distribution lengths would be much smaller than the current minimum sizes. Materials such as stainless steel can largely eliminate the bane of our present systems: leaking joints.

One very great advantage is that the size of community water treatment works would be a small fraction of what they now need to be, encouraging the use of membrane treatment, which improves the drinking water quality beyond what is now available in most communities. The result would be purer water at a lower cost. Figure 8.13 illustrates what a new community might do to initiate such an approach to conserving its drinking water, affording high-quality treatment, and avoiding degradation in the distribution system.

What makes this approach reasonable today is that dual systems on a large scale began to cover the United States some 40 years ago. Some 2,000 communities in the United States and many abroad, both the largest and the smallest

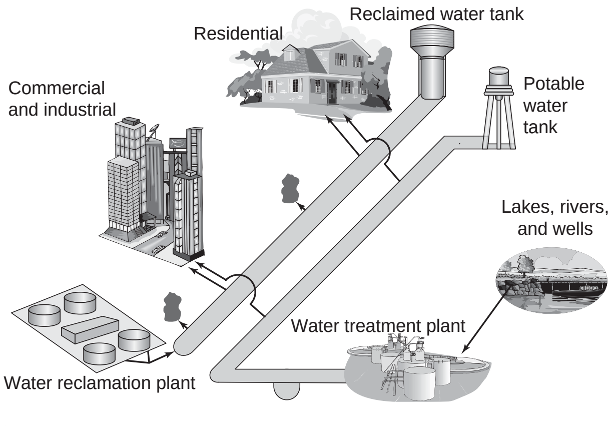


Figure 8.13 Dual water supply systems.

cities, are adopting dual systems. They began in arid areas but they are present also in Oregon and Washington, the country's wettest states. In Florida, with about 48 in. per year (1,219 mm/yr) of rainfall, some 450 communities have dual systems. Their popularity is based largely on the drinking water supply conservation they provide.

For new communities, a dual system with one system for drinking water only, and the other for all nonpotable purposes, including fire protection, would address not only drinking water supply problems, but water quality problems as well. In addition, such systems would be far less costly than conventional systems. Communities would be able to afford membrane treatment because the amount of drinking water to be treated to high quality would be considerably smaller than overall rates of water to be treated.

One such system was adopted 10 years ago for a new suburb of Sydney, Australia, called Rouse Hill, involving 250,000 people with a first stage for 100,000 residents. It has been operating successfully since. This approach is particularly beneficial for systems that are obliged to take water from sources of poor quality.

The problem is much more difficult to address in existing communities that are growing or retrofitting new systems. All cities are site specific. Where new distribution systems are being added, installing the stainless steel pipes gradually would be beneficial but costs would need to be assessed to determine whether they are feasible. The high costs of the conventional pipes and future problems with water quality may justify the higher initial costs.

From a public health perspective we cannot afford to continue our current practices. Dual systems and the water supply conservation they provide, along with distribution systems carrying only well-treated drinking water, dictate the need to study new approaches.

8.8 RAW WATER INTAKE STRUCTURES AND RAW WATER PUMPING WELLS

In accordance with the Ten-States Standards (*Recommended Standards for Water Works,* 2007), design of an intake structure shall provide for (a) withdrawal of water from more than one level if water quality varies with depth; (b) separate facilities for release of less desirable water held in storage; (c) where frazil ice may be a problem, holding the velocity of flow into the intake structure to minimum, generally not to exceed 0.5 ft/s (0.1524 m/s); (d) inspection of manholes every 1,000 ft (304.8 m) for pipe sizes large enough to permit visual inspection; (e) occasional cleaning of the inlet line; (f) adequate protection against rupture by dragging anchors, ice, and so on; (g) ports located above the bottom of the stream, lake, or impoundment, but at sufficient depth to be kept submerged at low water levels; (h) where shore wells are not provided, a diversion device capable of keeping large quantities of fish or debris from entering an intake structure; and (i) when buried surface water collectors are used, sufficient intake opening area to minimize inlet headloss. Particular attention should be given to the selection of backfill material in relation to the collector pipe slot size and gradation of the native material over the collector system.

A raw water pumping well shall meet the following requirements: (a) it shall have motors and electrical controls located above grade, and protected from flooding as required by the reviewing authority; (b) it shall be accessible; (c) it shall be designed against flotation; (d) it shall be equipped with removable or traveling screens before the pump suction well; (e) it shall provide for introduction of chlorine or other chemicals in the raw water transmission main if necessary for water control; (f) it shall have intake valves and provisions for backflushing or cleaning by a mechanical device and testing for leaks, where practical; and (g) it shall have provisions for withstanding surges where necessary.

An off-stream raw water storage reservoir is a facility into which raw water is pumped during periods of good water quality and high stream flow for future release to water treatment facilities. These off-stream raw water storage reservoirs shall be constructed to assure that (a) water quality is protected by controlling surface runoff into the reservoir; (b) dikes are structurally sound and protected against wave action and erosion; (c) intake structures and devices meet requirements stated above; (d) point of influent flow is separated from the point of withdrawal; and (e) separate pipes are provided for influent to and effluent from the raw water storage reservoir.

EXAMPLE 8.12 DESIGN OF RAW WATER INTAKE STRUCTURE

Design a concrete raw water intake structure with two equal-size square gates under the following assumed engineering conditions:

a. Elevation of the intake structure top (maximum) = 87 m (msl = mean sea level).
b. Elevation of maximum reservoir water level = 82 m (msl).
c. Elevation of normal reservoir water level = 81 m (msl).
d. Elevation of minimum reservoir water level = 80 m (msl).
e. Elevation of the intake structure bottom (planned) = 60 m (msl).

f. Design flow (maximum daily demand) = 37.85 MLD.

g. Design water velocity through the gate = 0.1524 m/s (maximum; the Ten-States Standards).

h. One of the two square gates can be closed for screen repair or replacement.

Solution (SI System):

Design flow Q = 37.85 MLD = 0.438 m^3/s.

Selected velocity for design v = 0.09144 m/s (below 0.1524 m/s, okay).

Total area of two gates $A = Q/v$ = (0.438 m^3/s)/(0.09144 m/s) = 4.79 m^2.

Select two equal-size square gates, $L = W = (4.79/2)^{0.5}$ = 1.5476 m.

Select the next larger standard-size gate from a manufacturer's catalog.

Length L = width W = 1.6764 m.

Check velocity when both gates are open:

$$v = 0.438/[(1.6764 \times 1.6764) \times 2] = 0.078 \text{ m/s (less than 0.1524 m/s, okay).}$$

Check velocity when only one gate is open:

$$v = 0.438/[(1.6764 \times 1.6764)] = 0.1558 \text{ m/s (close to 0.1524 m/s, okay).}$$

Elevation of the top of the gate = 70.83 m (msl).

Elevation of the bottom of the gate = 69.16 m (msl).

The above are the final design results for the two gates. The following are the tentative design data for the concrete intake structure that will hold the two gates. The following tentative design data require (a) a hydraulic stability analysis in accordance with Example 8.13 and (b) a reinforced concrete structural analysis which is beyond the scope of this water engineering class.

Outside dimensions of the raw water concrete intake structure = 8.5 m × 8.5 m (tentative design).

Inside dimensions of the raw water concrete intake structure = 6.5 m × 6.5 m (tentative design).

Height of the raw water concrete intake structure = 87 m – 60 m = 17 m (tentative design).

Base-reinforced concrete slab selected to sit at the reservoir bottom = 14 m × 14 m × 2.5 m (thickness) (tentative design).

EXAMPLE 8.13 HYDRAULIC STABILITY ANALYSIS OF RAW WATER INTAKE STRUCTURE

Conduct a hydraulic stability analysis for the raw water RC intake structure tentatively designed in Example 8.12. The intake structure displaces a large amount of water. The weight of the intake structure must be greater than the weight of water displaced in order to ensure hydraulic stability. In case the tentatively designed RC intake structure (from Example 8.12) is concluded to be hydraulically unstable, please suggest how to make corrections. The following engineering conditions are assumed:

a. Outside dimensions of the raw water concrete intake structure = 8.5 m × 8.5 m; inside dimensions of the raw water concrete intake structure = 6.5 m × 6.5 m; height of the raw water concrete intake structure = 87 m – 60 m = 27 m; base-reinforced concrete slab selected = 14 m × 14 m × 2.5 m (thickness)

b. Elevation of the intake structure top (maximum) = 87 m (msl = mean sea level); elevation of maximum reservoir water level = 82 m (msl); elevation of normal reservoir water level = 81 m (msl); elevation of minimum reservoir water level = 80 m (msl); elevation of the intake structure bottom (planned) = 60 m (msl)

c. Weight of reinforced concrete = 2,308 kg/m^3; weight of water = 1,000 kg/m^3

d. Required safety factor = 1.5–2.0

Solution (SI System):

The worst condition for the hydraulic stability of an intake structure will occur when (a) the two gates are closed, (b) the intake structure is empty, and (c) the reservoir is at its maximum water level of 82 m elevation.

Although the outside volume of the intake structure is (8.5 m)(8.5 m)(87 – 60)m + (14 m)(14 m)(2.5 m) = 2,440.75 m^3, actual volume of water displaced by the intake structure is (8.5 m)(8.5 m)(82 – 60)m + (14 m)(14 m)(2.5 m) = 2,079.5 m^3.

Total weight of water displaced = (1,000 kg/m^3)(2,079.5 m^3) = 2,079,500 kg.

The weight of the intake structure must be greater than 2,079,500 kg in order to provide stability.

Weight of the side walls = [8.5 × 8.5 (87 – 60) – 6.5 × 6.5 (87 – 60)] m^3 (2,308 kg/m^3) = 1,869,580 kg.

Weight of foundation slab = (14 × 14 × 2.5) m^3 (2,308 kg/m^3) = 1,130,920 kg.

Total weight of the concrete intake structure = 1,869,480 + 1,130,920 = 3,000,400 kg.

The safety factor = (weight of intake structure)/(weight of water displaced) = 3,000,400 kg/ 2,079,500 kg = 1.44.

Discussion: Since the required safety factor is 1.5–2.0, it is concluded that the tentatively designed concrete intake structure (safety factor = 1.44) should only be slightly corrected. The safety factor of this intake structure can be increased by adding additional weight to the RC structure or by anchoring the concrete footing.

PROBLEMS/QUESTIONS

8.1 Determine the water horsepower, break horsepower, and motor horsepower for a pump operating under the following conditions: water flow of 490 gpm (31 L/s) is to be pumped against a total head of 110 ft (33.53 m); the pump efficiency is 75%; and the motor driving the pump has an efficiency of 85%.

8.2 A pump is to be located 6 ft (1.83 m) above a wet well and must lift 600 gpm (38 L/s) of water another 52 ft (15.85 m) to a storage reservoir through a piping system consisting of 1,250 ft (381 m) of 6 in. (150 mm) DIP pipe ($C = 110$), two globe valves (open), and two medium sweep elbows. Determine the total dynamic head (TDH) for this water pumping system.

8.3 Using the data from Problem 8.2 assume the following additional data: efficiency of pump = 80% efficiency of motor = 85%. Determine (a) the motor horsepower in hp and kWh/day and (b) the daily power cost if the unit power cost is $0.1 per kWh.

8.4 Determine the total power input if the electrical input to a motor-pump system is 220 V and 36 amp.

8.5 Determine the wire-to-water efficiency (%) if $Q =$ 510 gpm (32 L/s), TDH = 53.65 ft (16.35 m), $V = 220$ V, and $A = 36$ amp.

8.6 Changing the speed of a centrifugal pump will change its operating characteristics, including the water flow. Determine the new flow rate or capacity, Q_2, if

Q_1 = rated water flow = 620 gpm = 39 L/s
N_2 = pump speed now = 1,320 rpm
N_1 = rated pump speed = 1,650 rpm

8.7 Changing the speed of a centrifugal pump will change its operating characteristics, including the head. Determine the new head, if

H_1 = rated head = 120 ft = 36.58 m
N_2 = pump speed now = 1,320 rpm
N_1 = rated pump speed = 1,650 rpm

8.8 Changing the speed of a centrifugal pump will change its operating characteristics, including the power requirement. Determine the new power requirement, if

P_1 = rated head = 16 hp = 12 kW
N_2 = pump speed now = 1,320 rpm
N_1 = rated pump speed = 1,650 rpm

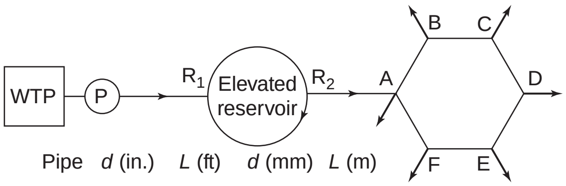
8.9 A water pumping station is designed to raise water from a lake at an elevation of 50 ft (15.24 m) to a reservoir located at an elevation of 140 ft (42.67 m). Water is pumped through a 16 in. (40.64 cm) cast-iron pipe that develops a head loss of 19 ft (5.79 m) of water at a discharge of 1000 gpm (63 L/s). The pumping station has two pumps, each of which possesses the following characteristics:

Flow (gpm)	Head (ft)
500	195
1,000	180
1,500	150
2,000	110
2,500	50

Flow (L/s)	Head (m)
32	59.44
63	54.86
95	45.72
127	33.53
158	15.24
Shut-off head = 200 ft (60.96 m)	

What will be the discharge of the two pumps when operating in parallel?

8.10 The water supply system shown in Fig. 8.14 is designed to serve city ABCDEFA. Water is treated and collected in a water tank in the treatment plant (WTP). A pump delivers the water through the main PR1 to an elevated reservoir (water level 2,700 ft, or 822.96 m) at the top of a hill. Pumping is done at a constant rate and only for a period of 16 h per day from 4 a.m. to 8 p.m. Water flows from the elevated reservoir into the distribution main network ABCDEF through a 24 in. main R_2A.



Pipe	*d* (in.)	*L* (ft)	*d* (mm)	*L* (m)
PR_1	24	35,000	600	10,668
R_2A	24	15,000	600	4,572
AB	14	8,000	350	2,438
BC	12	10,000	300	3,048
CD	8	5,000	200	1,524
DE	8	5,000	200	1,524
EF	12	10,000	300	3,048
AF	14	8,000	350	2,438

Figure 8.14 Water system for Problem 8.10.

The water consumption during the maximum day demand in the city is as follows:

Period	Flow rate (gpm)
Midnight to 4 a.m.	2,000
4 a.m. to 8 a.m.	4,000
8 a.m. to noon	6,000
Noon to 4 p.m.	5,000
4 p.m. to 8 p.m.	4,000
8 p.m. to midnight	3,000

Period	Flow rate (L/s)
Midnight to 4 a.m.	126
4 a.m. to 8 a.m.	252
8 a.m. to noon	379
Noon to 4 p.m.	315
4 p.m. to 8 p.m.	252
8 p.m. to midnight	189

Assume that the water withdrawal from the network is equally distributed among the points A, B, C, D, E, and F, which are at the same elevation of 2,300 ft (701.04 m) and that all pipes are ductile iron with $C = 100$. Also consider that the pump possesses the following characteristics:

Flow (gpm)	0	2,000	4,000	6,000	8,000	12,000
Head (ft)	400	390	370	345	310	225
Flow (L/s)	0	126	252	379	505	757
Head (m)	121.92	118.87	112.78	105.16	94.49	68.58

(a) Calculate the volume of the elevated reservoir needed to balance supply and demand.

(b) Compute the maximum flow in each pipe of the network.

(c) Determine the elevation of the water tank at the treatment plant.

(d) At what rate of flow can you pump the water to the elevated reservoir if another identical pump is installed in parallel with the existing pump?

8.11 The water supply system shown in Fig. 8.15 is planned to serve a small village. Treated water from the treatment plant (WTP) flowing at a uniform rate throughout the day is collected in a ground tank. Water is then pumped to an elevated reservoir at the top of a hill through the main AB. Water flows from the elevated reservoir into the distribution network through the main CD.

Water consumption during the maximum day in the village is as follows:

Period	Flow rate (L/s)
Midnight to 6 a.m.	60
6 a.m. to noon	240
Noon to 6 p.m.	120
6 p.m. to midnight	90

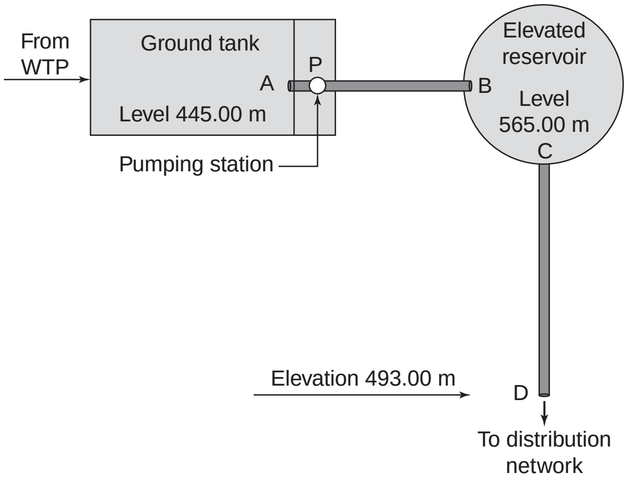


Figure 8.15 Water system for Problem 8.11.

Period	Flow rate (gpm)
Midnight to 6 a.m.	951
6 a.m. to noon	3,804
Noon to 6 p.m.	1,902
6 p.m. to midnight	1,427

The pumping station has three pumps, two of which operate in parallel and the third pump is a standby. Pumping is done at a constant rate and only for a period of 12 hours from 6 a.m. to 6 p.m. Each pump possesses the following characteristics:

Flow (gpm)	0	951	1,427	1,902	2,378	2,853
Head (ft)	54.86	50.29	46.63	41.15	32.00	19.20
Flow (L/s)	0	60	90	120	150	180
Head (m)	180	165	153	135	105	63

The Hazen–Williams coefficient C for all mains is 100. The total equivalent length of main AB is 8,202 ft (2,500 m) and main CD is 29,527 ft (9,000 m).

(a) Determine the volumes needed for the ground tank and elevated reservoir (each separately) to balance supply and demand. (Storage for firefighting and emergencies is not required.)

(b) Determine the minimum standard size needed for the water main AB so that the two operating pumps can deliver the required flow from the ground tank to the elevated reservoir.

8.12 A water pumping station is designed to raise water from a treatment plant ground reservoir at an elevation of 1,968 ft (600 m) to an elevated reservoir located on the opposite side of town at an elevation of 2,165 ft (660 m). The trunk line and the water distribution network connecting the two reservoirs is equivalent to a pipe line ($C = 100$) 16 in. (400 mm) in diameter and 82,020 ft (25,000 m) long.

The pumping station has three pumps, two of which operate in series and the third pump is a standby. Each pump possesses the following characteristics:

Flow (gpm)	0	634	1,268	1,902	2,536	3,170	3,804
Head (ft)	328	321	308	285	259	230	197
Flow (L/s)	0	40	80	120	160	200	240
Head (m)	100	98	94	87	79	70	60

During night hours, when consumption is low, the pumped water is greater than demand and excess water is stored in the elevated reservoir. Considering this mode of operation, determine the flow rate that can be delivered through the system under the following pumping conditions:

(a) Two pumps are operational.

(b) One pump is operational, that is, one pump failed and the standby is under maintenance.

8.13 A water pumping station is designed to raise water from a treatment plant ground reservoir at an elevation of 190 ft (57.91 m) to a downtown elevated reservoir located at an elevation of 415 ft (126.49 m). Water is pumped through a 20 in.(500 mm) pipe (C = 100), which is 30,000 ft (9,144 m) long. The pumping station has two pumps, each of which possesses the following characteristics:

Flow (gpm)	15,852	31,704	47,556	63,408	79,260	95,112	110,964
Head (ft)	951	886	804	689	558	410	213
Flow (L/s)	1,000	2,000	3,000	4,000	5,000	6,000	7,000
Head (m)	290	270	245	210	170	125	65

Shut-off head = 300 ft (91.44 m)

What will be the discharge of the two pumps when operating in series? Also what will be the total head against which the pumps are actually operating?

8.14 An industrial complex utilizes water at a uniform rate of 5,400 gpm (20,439 L/min) during its working hours from 8 a.m. to noon and from 4 p.m. to 8 p.m. This water demand is withdrawn from an elevated water tank located at an elevation of 40 ft (12.19 m) above ground level.

Water is pumped at a uniform rate continuously throughout the day to the tank using two pumps connected in series from a well whose water level is 460 ft (104.21 m) below ground level. The water pipe from the well to the tank is 3,000 ft (914.4 m) long when $C = 100$.

Each pump has the following characteristics:

Flow (gpm)	0	1,000	2,000	3,000	4,000
Head (ft)	350	325	275	175	50
Flow (L/s)	0	63	126	189	252
Head (m)	106.68	99.06	83.82	53.34	15.24

(a) Calculate the water tank volume required to equalize supply and demand.

(b) Determine the size of the pipe delivering water from the well to the water tank.

8.15 A summer resort complex consumes water at the following uniform rates during the indicated periods:

Period	Consumption rate (gpm)
Midnight to 8 a.m.	5.4
8 a.m. to 4 p.m.	11.4
4 p.m. to midnight	8.2

Period	Consumption rate (m^3/h)
Midnight to 8 a.m.	20
8 a.m. to 4 p.m.	50
4 p.m. to midnight	30

How large a storage tank would be required to equalize supply and demand for each of the following conditions?

(a) Water is obtained from a nearby city at a uniform rate from 8 a.m. to 4 p.m.

(b) Water is obtained from the same city at a uniform rate over the 24-h period.

8.16 For the water supply of a small town with a daily water requirement of 0.594 MG (2,250 m^3), the construction of a water reservoir has been proposed. The pattern of drawoff is as follows:

Period	% of daily demand
7 a.m. to 8 a.m.	30
8 a.m. to 5 p.m.	35
5 p.m. to 6:30 p.m.	30
8:30 p.m. to 7 a.m.	5

Water is to be supplied to the reservoir at a constant rate for a period of 8 h, 8 a.m. to 4 p.m. Determine the storage capacity of the reservoir needed to balance supply and demand. No storage for fire is required. If pumping is to be done at a constant rate over the 24 h, what will then be the required reservoir size?

8.17 An industrial plant requires 5,000 gpm (316 L/s) of water during its on-shift hours from 6 a.m. to 10 p.m. How large (in gallons or liters) a storage tank would be required to equalize the pumping rate for each of the following conditions?

(a) Water is obtained from a well at a uniform rate over the 24-h period.

(b) Water is obtained from a well during the period from 10 p.m. to 6 a.m., which is the off-peak period of electricity consumption.

8.18 A factory requires 793 gpm (50 L/s) of water during its working hours from 8 a.m. to 4 p.m. How large a storage reservoir would be required to equalize supply and demand for each of the following water supply conditions?

(a) Water is supplied from a well at a uniform rate over the 24-h period.

(b) Water is supplied at a uniform rate from the city network during the off-peak period from 4 p.m. to 8 a.m.

8.19 A residential neighborhood, population 20,000, is supplied with water from an elevated reservoir. The daily water consumption is as follows:

Period	% of daily demand
Midnight to 3 a.m.	5
3 a.m. to 6 a.m.	7
6 a.m. to 9 a.m.	13
9 a.m. to noon	20
Noon to 3 p.m.	20
3 p.m. to 6 p.m.	15
6 p.m. to 9 p.m.	10
9 p.m. to midnight	10

The average daily water consumption is 106 gpcd (400 Lpcd) and the maximum daily consumption is 125% of the daily average. Determine the storage volume (in m^3) necessary to balance supply and demand in each of the following cases:

(a) If water is supplied into the reservoir at a constant rate over the 24-h period.

(b) If water is supplied into the reservoir at a constant rate, but only for a period of 12 h from 6 a.m. to 6 p.m.

8.20 A residential complex consists of 60 duplex villas (two housing units in each). Each housing unit can accommodate a maximum of 10 persons. The expected maximum daily water consumption is 106 gal/capita (400 L/capita). Water is supplied from the municipal network at a uniform rate throughout the day and is collected in a ground storage reservoir. Water is then pumped to an elevated reservoir at a constant rate but only for a period of 9 h from 6 a.m. to 3 p.m. Water then flows from the elevated reservoir to the various villas by gravity to satisfy the following daily demands:

Period	% of daily demand
Midnight to 3 a.m.	5
3 a.m. to 6 a.m.	5
6 a.m. to 9 a.m.	25
9 a.m. to noon	15
Noon to 3 p.m.	15
3 p.m. to 6 p.m.	10
6 p.m. to 9 p.m.	15
9 p.m. to midnight	10

In order to balance supply and demand, determine

(a) Required volume of the ground reservoir.

(b) Required volume of the elevated reservoir.

8.21 A small water system in a rural area has a population of 800, and its daily per capita usage is estimated to be 100 gpcd (379 Lpcd). The required fire flow determined by the village engineer is 500 gpm (32 L/s) for a duration of 2 h. There is no particular hazard to the water supply works. Determine the required water storage to be provided for the village using the *Recommended Standards for Water Works, 2007 Edition*. Visit the website or contact the local health department for the latest edition of these Ten-States Standards for water works.

8.22 A city is planning to improve its water supply system (Fig. 8.16). At present the city has a surface water supply reservoir at an elevation of 100 ft (30.48 m) at point A, a pumping station at an elevation of 80 ft (24.38 m) at point B, water storage tank at an elevation of 200 ft (60.96 m) at point C, 500 ft (152.4 m) of suction transmission line between the reservoir and the pumping station, and 10,000 ft (3,048 m) of 10 in. (250 mm) pressure transmission line between the pumping station and the water storage tank. The water at the water storage tank site is treated by UV for disinfection and then discharged to the water distribution system.

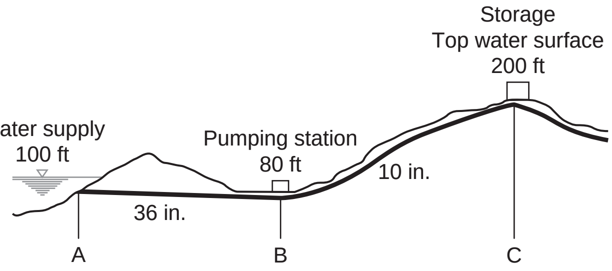


Figure 8.16 Water supply system for Problem 8.22. Conversion factors: 1 in. = 25.4 mm; 1 ft = 0.3048 m.

A proposal has been made to replace the current pumps with two centrifugal pumps, construct 10,000 ft (3,048 m) of 16 in. (400 mm) transmission main to parallel the current 10 in. (250 mm) water main and to provide additional water storage at an elevation of 200 ft (60.96 m) at the current water storage site. The following conditions are assumed: (a) *C* factor for all pipes = 120; (b) fire flow requirements in the city = 2,000 gpm (126.2 L/s) for 10 h; (c) average daily demand = 3 MGD = 131 L/s; (d) maximum daily demand = 5 MGD = 219 L/s; (e) peak hourly demand = 10 MGD = 438 L/s; (f) pumping station head losses = 500 ft of 16 in. pipe (or 152.4 m of 400 mm pipe) equivalent; and (g) the field-measured total dynamic head delivered by the pump from the raw water supply reservoir to the pump station, then to the water storage tank = 150 ft (45.72 m).

(a) What should the pump rating (gpm or L/min) be for each of the two new pumps?

(b) What should the brake horsepower (BHP or BMP) be for pump selection, and what should the motor horsepower (MHP or MMP) be for motor selection?

(c) Will the two selected pumps be connected in parallel or in series?

8.23 Using the same given data in Problem 8.22, explain why the pipe line is designed based on the maximum daily water demand, and determine the total storage capacity of a new water storage tank.

8.24 Design a concrete raw water intake structure with two equal-size square gates under the following assumed engineering conditions:

(a) Elevation of the intake structure top (maximum) = 285.43 ft (msl = mean sea level).

(b) Elevation of maximum reservoir water level = 269.03 ft (msl).

(c) Elevation of normal reservoir water level = 265.75 ft (msl).

(d) Elevation of minimum reservoir water level = 262.47 ft (msl).

(e) Elevation of the intake structure bottom (planned) = 196.85 ft (msl).

(f) Design flow (maximum daily demand) = 10 MGD.

(g) Design water velocity through the gate = 0.5 ft/s (maximum; the Ten-States Standards).

(h) One of the two square gates can be closed for screen repair or replacement.

8.25 Conduct a hydraulic stability analysis for the raw water RC intake structure tentatively designed in Problem 8.24. The intake structure displaces a large amount of water. The weight of the intake structure must be greater than the weight of water displaced in order to ensure hydraulic stability. In case the tentatively designed RC intake structure (from Problem 8.24) is concluded to be hydraulically unstable, please suggest how to make corrections. The following are the assumed engineering conditions:

(a) Outside dimensions of the raw water concrete intake structure = 27.89 ft × 27.89 ft; inside dimensions of the raw water concrete intake structure = 21.33 ft × 21.33 ft; height of the raw water concrete intake structure = 285.43 ft − 196.85 ft = 88.58 ft; base-reinforced concrete slab selected = 45.93 ft × 45.93 ft × 8.20 ft (thickness)

(b) Elevation of the intake structure top (maximum) = 285.43 ft (msl = mean sea level); elevation of maximum reservoir water level = 269.03 ft (msl); elevation of normal reservoir water level = 265.75 ft (msl); elevation of minimum reservoir water level = 262.47 ft (msl); elevation of the intake structure bottom (planned) = 196.85 ft (msl)

(c) Weight of reinforced concrete = 144 lb/ft^3; weight of water = 62.4 lb/ft^3

(d) Required safety factor = 1.5–2.0

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Chapter 9

Cross-Connection Control

9.1 INTRODUCTION

Plumbing *cross-connections*, which are defined as actual or potential connections between a potable and nonpotable water supply, constitute a serious public health hazard. There are numerous, well-documented cases where cross-connections have been responsible for contamination of drinking water and have resulted in the spread of disease. The problem is a dynamic one, because piping systems are continually being installed, altered, or extended.

Control of cross-connections is possible, but only through thorough knowledge and vigilance. Education is essential, because even those who are experienced in piping installations fail to recognize cross-connection possibilities and dangers. All municipalities with public water supply systems *should have cross-connection control programs*. Those responsible for institutional or private water supplies should also be familiar with the dangers of cross-connections and should exercise careful surveillance of their systems. The American Water Works Association (AWWA) stated the following in a policy on public water supply matters:

> AWWA recognizes water purveyors have the responsibility to supply potable water to their customers. In the exercise of this responsibility, water purveyors or other responsible authorities must implement, administer, and maintain ongoing backflow prevention and cross-connection control programs to protect public water systems from the hazards originating on the premises of their customers and from temporary connections that may impair or alter the water in the public water systems. The return of any water to the public water system after the water has been used for any purposes on the customer's premises or within the customer's piping system is unacceptable and opposed by AWWA. The water purveyor shall assure that effective backflow prevention measures commensurate with the degree of hazard are implemented to ensure continual protection of the water in the public water distribution system. Customers, together with other authorities, are responsible for preventing contamination of the private plumbing system under their control and the associated protection of the public water system. (Reprinted by permission. Copyright © 2009, American Water Works Association.)

Public health officials have long been concerned about cross-connections and backflow connections in plumbing systems and in public drinking water supply distribution systems. Such cross-connections, which make possible the contamination of potable water, are ever-present dangers. One example of what can happen is an epidemic that occurred in Chicago in 1933. Old, defective, and improperly designed plumbing and fixtures permitted the contamination of drinking water. As a result, 1,409 people contracted amebic dysentery, and 98 of them died. This epidemic, and others resulting from contamination introduced into a water supply through improper plumbing, made clear the responsibility of public health officials and water purveyors for exercising control over public water distribution systems and all plumbing systems connected to them. This responsibility includes advising and instructing plumbing installers in the recognition and elimination of cross-connections.

Cross-connections are the links through which it is possible for contaminating materials to enter a potable water supply. The contaminant enters the potable water system when the pressure of the polluted source exceeds the pressure of the potable source. The action may be called *backsiphonage* or *backflow*. Essentially it is the reversal of a hydraulic gradient, and it can be produced by a variety of circumstances.

One might assume that the steps for detecting and eliminating cross-connections would be elementary and obvious. The reality, however, is that cross-connections can appear in many subtle forms and in unsuspected places. Reversal of pressure in the water may be freakish and unpredictable. The probability of contamination of drinking water through a cross-connection occurring within a single plumbing system may seem remote, but considering the multitude of similar

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Water Engineering: Hydraulics, Distribution and Treatment, First Edition. Nazih K. Shammas and Lawrence K. Wang.

systems, the probability is great. Cross-connections exist for these reasons:

1. Plumbing is frequently installed by persons who are unaware of the inherent dangers of cross-connections.
2. Oftentimes, connections are made as a simple matter of convenience without regard to dangerous situations that might be created.
3. Connections are made with reliance on inadequate protection such as a single valve or other mechanical device.

To combat the dangers of cross-connections and backflow connections, education in their recognition and prevention is needed. First, plumbing installers must know that hydraulic and pollutional factors may combine to produce a sanitary hazard if a cross-connection is present. Second, they must realize that reliable and simple standard backflow prevention devices and methods are available that can be substituted for convenient but dangerous direct connections. Third, plumbing installers must understand that the hazards resulting from direct connections greatly outweigh the convenience gained from a quick and direct connection.

9.2 PUBLIC HEALTH SIGNIFICANCE OF CROSS-CONNECTIONS

Public health officials have long been aware of the threat to public health that cross-connections represent. Because plumbing defects are so frequent and the opportunities for contaminants to invade the public drinking water through cross-connections so general, enteric illnesses caused by drinking water may occur at most at any location and at any time.

The following documented cases of cross-connection problems illustrate and emphasize how cross-connections have compromised water quality and public health.

9.2.1 Human Blood in the Water System

Health department officials cut off the water supply to a funeral home located in a large southern city after it was determined that human blood had contaminated the freshwater supply. The chief plumbing inspector had received a telephone call advising that blood was coming from drinking fountains within the funeral home building. Plumbing and county health department inspectors went to the scene and found evidence that blood had been circulating in the water system within the building. They immediately ordered the building cut off from the water system at the meter. City water and plumbing officials said that they did not think that the blood contamination had spread beyond the building; however, inspectors were sent into the neighborhood to check for possible contamination.

Investigation revealed that the funeral home had been using a hydraulic aspirator to drain fluids from the bodies of human "remains" as part of the embalming process. The aspirator connected directly to the water supply system at a faucet located on a sink in the "preparation" (embalming) room. Water flow through the aspirator created suction that was utilized to draw body fluids through a hose and needle attached to the suction side of the aspirator.

The contamination of the funeral home potable water supply was caused by a combination of low water pressure in conjunction with the simultaneous use of the aspirator. Instead of the body fluids flowing into the sanitary drain, they were drawn in the opposite direction—into the potable water supply of the funeral home (see Fig. 9.1).

9.2.2 Sodium Hydroxide in the Water Main

A resident of a small town in Alabama jumped in the shower at 5 a.m. one morning, and when he got out his body was covered with tiny blisters. "The more I rubbed it, the worse it got," the 60-year-old resident said. "It looked like someone took a blow torch and singed me."

He and several other residents received medical treatment at the emergency room of the local hospital after the public water system was contaminated with sodium hydroxide, a strong caustic solution.

Other residents claimed that "it [the water] bubbled up and looked like Alka Seltzer. I stuck my hand under the faucet and some blisters came up." One neighbor's head was covered with blisters after she washed her hair and others complained of burned throats or mouths after drinking the water.

The incident began after an 8 in. (200 mm) water main that fed the town broke and was repaired. While repairing the water main, one workman suffered leg burns from a chemical in the water and required medical treatment. Measurements of the pH of the water were as high as 13 in some sections of the pipe.

Investigation into the cause of the problem led to a nearby chemical company that distributes chemicals such as sodium hydroxide as being the possible source of contamination. Sodium hydroxide is brought to the plant in liquid form in bulk tanker trucks, transferred to a holding tank, and then pumped into 55 gal (208 L) drums. When the water main broke, a truck driver was adding the water from the bottom of the tank truck instead of the top, and sodium hydroxide backsiphoned into the water main (see Fig. 9.2).

9.2.3 Heating System Antifreeze in Potable Water

Bangor, ME, Water Department employees discovered poisonous antifreeze in a homeowner's heating system and water supply. The incident occurred when they shut off the service line to the home to make repairs. With the flow of water to the house cut off, pressure in the lines in the house dropped

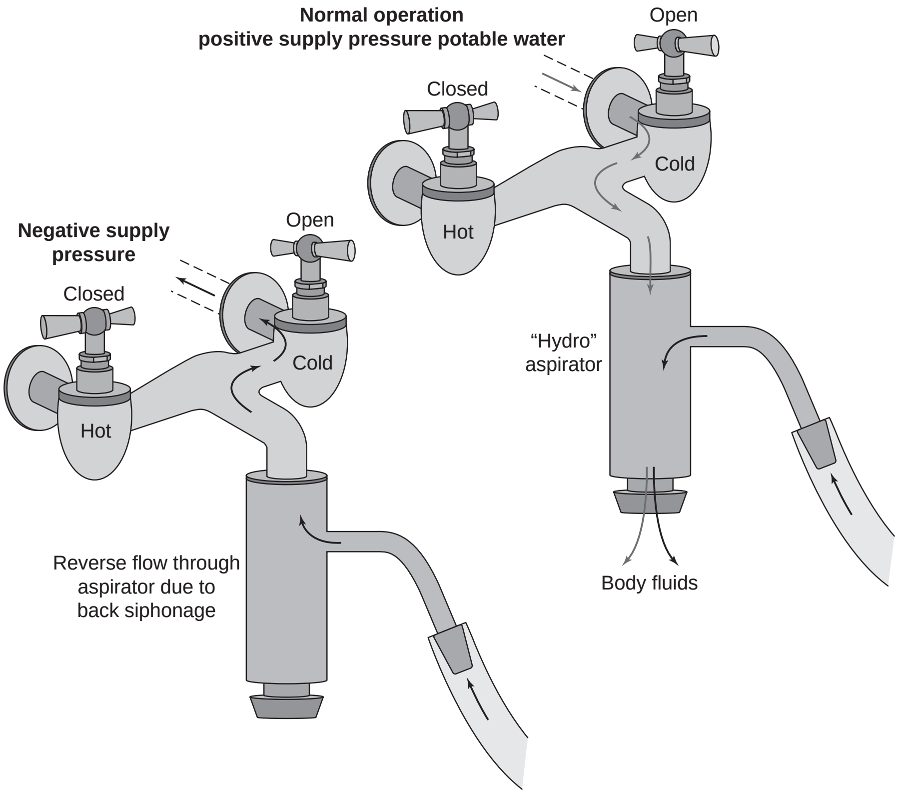


Figure 9.1 Reverse flow through aspirator due to backsiphonage.

and the antifreeze, placed in the heating system to prevent freeze-up of an unused hot water heating system, drained out of the heating system into house water lines and flowed out to the street (see Fig. 9.3). If it had not been noticed, it would have entered the homeowner's drinking water when the water pressure was restored.

9.2.4 Salt Water Pumped into Freshwater Line

A nationally known fast-food restaurant located in the southeastern United States complained to the water department that all of their soft drinks were being rejected by their customers as tasting "salty." This included soda fountain beverages, coffee, and orange juice. An investigation revealed that an adjacent water customer complained of salty water occurring simultaneously with the restaurant incident. This second complaint came from a waterfront ship repair facility that was also being served by the same water main lateral. The investigation centered on the ship repair facility and revealed the following (see Fig. 9.4):

- A backflow preventer that had been installed on the service line to the shipyard had frozen and had been replaced with a spool piece sleeve.
- The shipyard fire protection system utilized seawater that was pumped by both electric and diesel-driven pumps.
- The pumps were primed by potable city water.

With the potable priming line left open and the pumps maintaining pressure in the fire lines, raw salt water was pumped through the priming lines, through the spool piece sleeve, to the ship repair facility and the restaurant.

9.2.5 Paraquat in the Water System

"Yellow gushy stuff" poured from some of the faucets in a small town in Maryland, and the state of Maryland placed a ban on drinking the water supply. Residents were warned not to use the water for cooking, bathing, drinking, or any other purpose except for flushing toilets.

The incident drew widespread attention and made the local newspapers. In addition, it was the lead story on the ABC news affiliate in Washington, DC, and virtually all the Washington/Baltimore newspapers that evening. The news media contended that lethal pesticides may have contaminated the water supply and among the contaminants was paraquat, a powerful agricultural herbicide.

The investigation disclosed that the water pressure in the town water mains was temporarily reduced due to a water pump failure in the town water supply pumping system. Coincidentally, a gate valve between an herbicide chemical mixing tank and the town water supply piping had been left open. A lethal cross-connection had been created that permitted the herbicide to flow into the potable water supply system (see Fig. 9.5). On restoration of water pressure, the herbicides flowed into the many faucets and outlets hooked up to the town water distribution system.

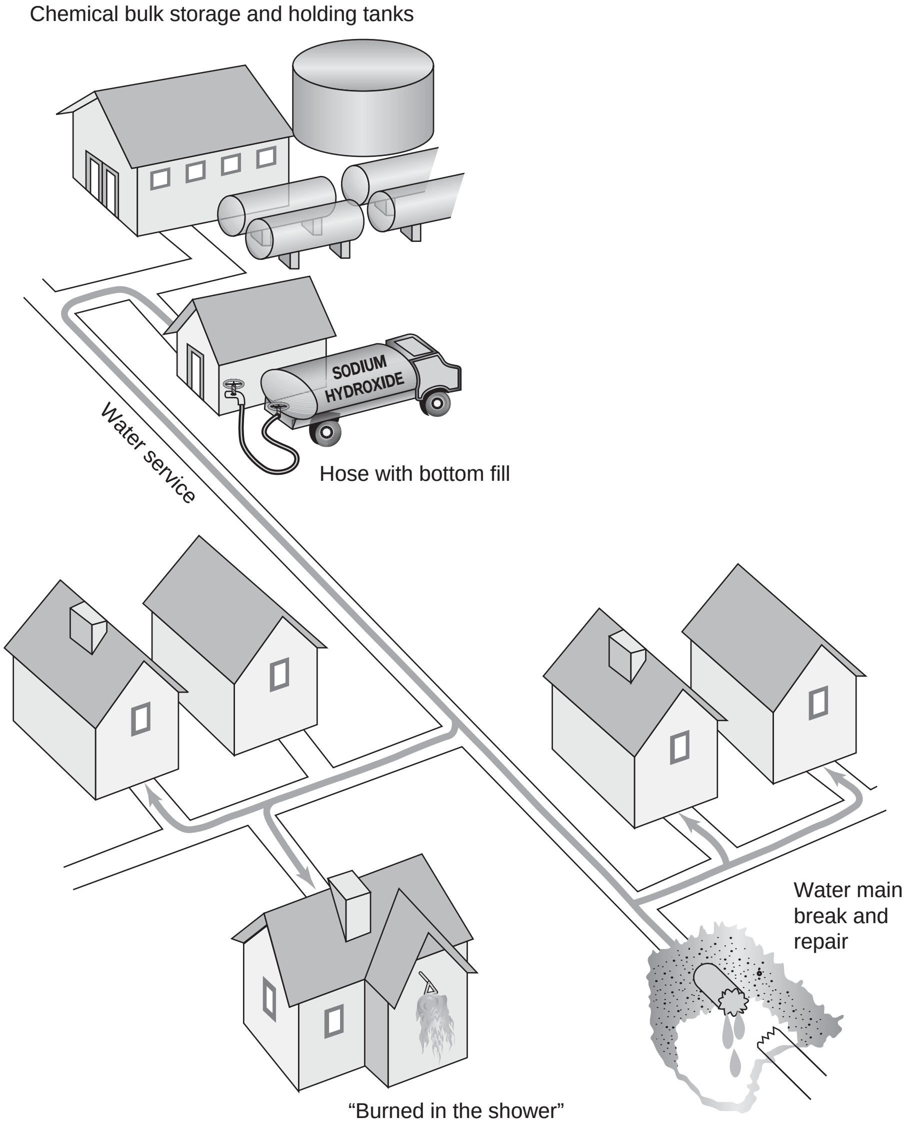


Figure 9.2 Sodium hydroxide backsiphoned into the water main.

This cross-connection, which could have been avoided by the installation of a backflow preventer, created a needless and costly event that fortunately did not result in serious illness or loss of life. Door-to-door public notification, extensive flushing, water sample analysis, and emergency arrangements to provide temporary potable water from tanker trucks all incurred expenses on top of the original problem, all of which would have been unnecessary had the proper precautions been taken.

9.2.6 Propane Gas in the Water Mains

Hundreds of people were evacuated from their homes and businesses in a town in Connecticut as a result of propane entering the city water supply system. Fires were reported in two homes and the town water supply was contaminated. One five-room residence was gutted by a blaze resulting from propane gas "bubbling and hissing" from a bathroom toilet and in another home a washing machine explosion blew a woman against a wall. Residents throughout the area reported hissing and bubbling noises, coming from washing machines, sinks, and toilets. Faucets sputtered out small streams of water mixed with gas, and residents in the area were asked to evacuate their homes.

This near-disaster occurred in one 30,000-gal (113,550-L)-capacity liquid propane tank when the gas company initiated immediate repair procedures. To start the repair, the tank was "purged" of residual propane by using water from one of two private fire hydrants located on the property. Water purging is the preferred method of purging over the use of carbon dioxide since it is more positive and will float out any sludge as well as any gas vapors. The "purging" consisted of hooking up a hose to one of the private fire hydrants located on the property and initiating flushing procedures (see Fig. 9.6).

Because the vapor pressure of the propane residual in the tank was 85–90 psi (590–625 kPa), and the water pressure

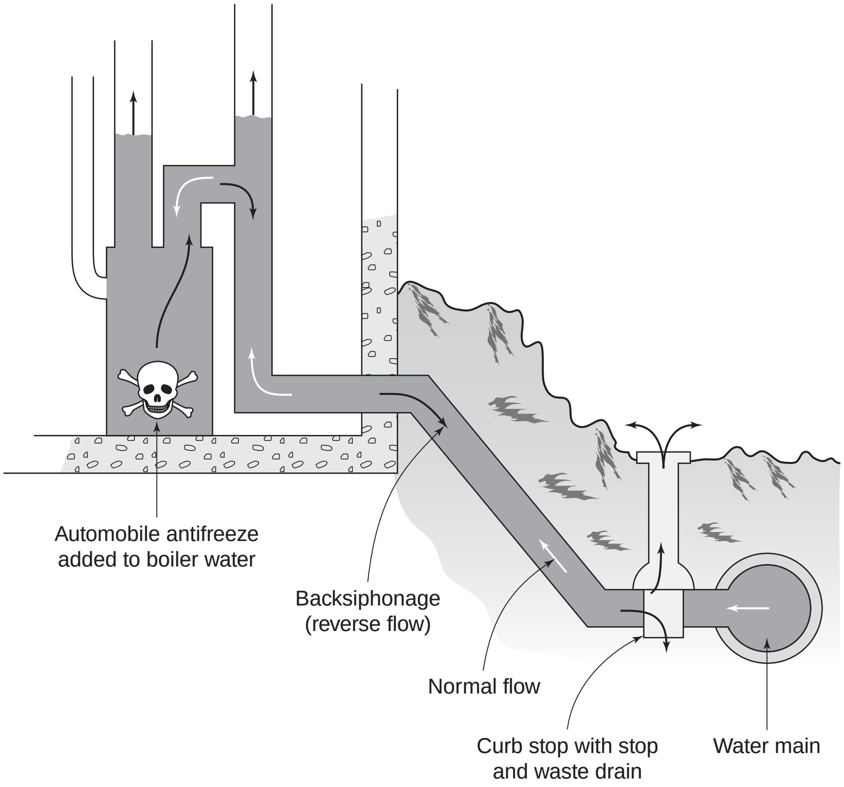


Figure 9.3 Heating system antifreeze into potable water.

was only 65–70 psi (451–486 kPa), propane gas backpressure backflowed into the water main. It was estimated that the gas flowed into the water mains for about 20 min and that about 2,000 ft^3 (57 m^3) of gas was involved. This was approximately enough gas to fill 1 mi of an 8 in. (1.61 km of a 200 mm) water main.

9.2.7 Chlordane and Heptachlor at a Housing Authority

The services to 75 apartments housing approximately 300 people were contaminated with chlordane and heptachlor in a city in Pennsylvania. The insecticides entered the water supply system while an exterminating company was applying them as a preventive measure against termites. While the pesticide contractor was mixing the chemicals in a tank truck with water from a garden hose coming from one of the apartments, a workman was cutting into a 6 in. (150 mm) main line to install a gate valve. The end of the garden hose was submerged in the tank containing the pesticides, and at the same time, the water to the area was shut off and the lines were being drained prior to the installation of the gate valve. When the workman cut the 6 in. (150 mm) line, water started to drain out of the cut, thereby setting up a backsiphonage condition. As a result, the chemicals were siphoned out of the truck, through the garden hose, and into the system, contaminating the 75 apartments (see Fig. 9.7).

Repeated efforts to clean and flush the lines were not satisfactory, and a decision was eventually made to replace the water line and all the plumbing that was affected. There were no reports of illness, but residents of the housing authority were told not to use any tap water for any purpose and they were given water that was trucked into the area by volunteer fire department personnel. They were without their normal water supply for 27 days.

9.2.8 Boiler Water Entered High School Drinking Water

A high school in New Mexico was closed for several days when a home economics teacher noticed the water in the potable system was yellow. City chemists determined that samples taken contained levels of chromium as high as 700 mg/L, “astronomically higher than the accepted levels of 0.05 mg/L.” The head chemist said that it was miraculous that no one was seriously injured or killed by the high levels of chromium. The chemical was identified as sodium dichromate, a toxic form of chromium used in heating system boilers to inhibit corrosion of the metal parts.

No students or faculty were known to have consumed any of the water; however, area physicians and hospitals advised that if anyone had consumed those high levels of chromium, the symptoms would be nausea, diarrhea, and burning of the mouth and throat. Fortunately, the home economics teacher, who first saw the discolored water before

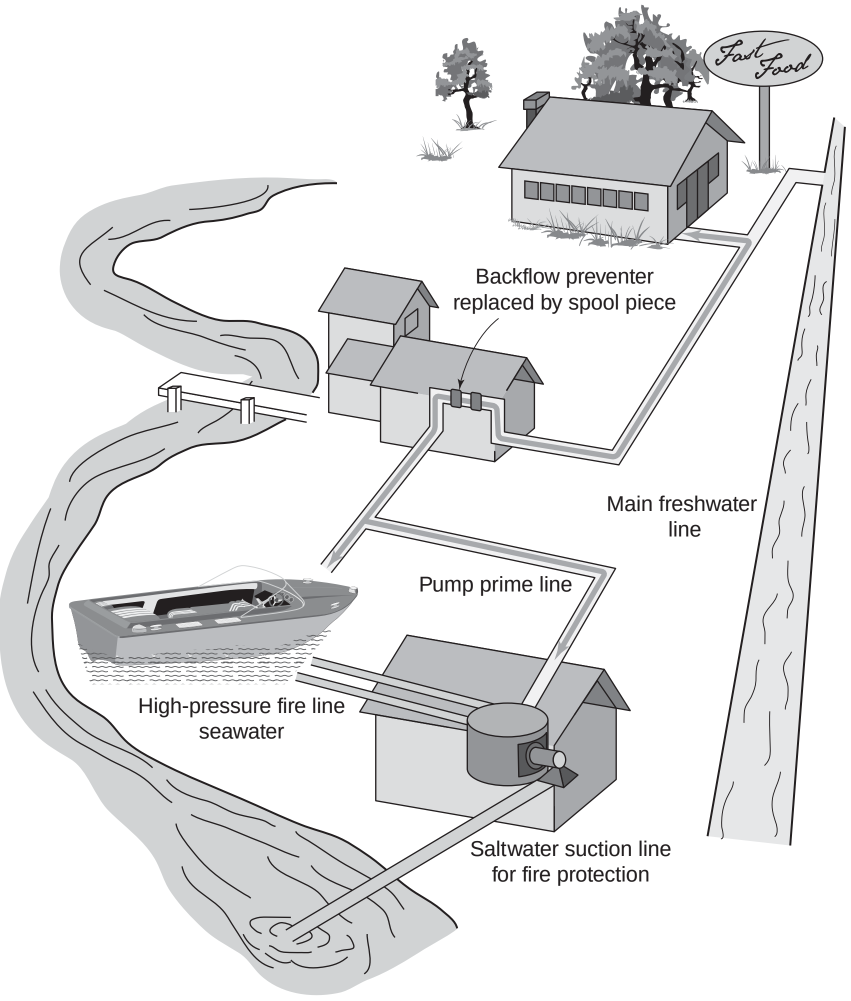


Figure 9.4 Salt water pumped into freshwater line.

school started, immediately covered all water fountains with towels so that no one would drink the water.

Investigation disclosed that chromium used in the heating system boilers to inhibit corrosion of metal parts entered the potable water supply system as a result of backflow through leaking check valves on the boiler feed lines (see Fig. 9.8).

9.2.9 Car Wash Water in the Street Water Main

This car wash cross-connection and backpressure incident, which occurred in the state of Washington, resulted in backflow chemical contamination of approximately 100 square blocks of water mains. Prompt response by the water department prevented a potentially hazardous water quality degradation problem without a recorded case of illness.

Numerous complaints of gray-green and "slippery" water were received by the water department coming from the same general area of town. A sample brought to the water department by a customer confirmed the reported problem and preliminary analysis indicated contamination with what appeared to be a detergent solution. While emergency crews initiated flushing operations, further investigation within the contaminated area signaled that the problem was probably caused by a car wash, or laundry, based on the soapy nature of the contaminant. The source was quickly narrowed down to a car wash, and the proprietor was extremely cooperative in admitting to the problem and explaining how it had occurred. The circumstances leading up to the incident were as follows:

- On Saturday, February 10, a high-pressure pump broke down at the car wash. This pump recycled reclaimed wash and rinse water and pumped it to the initial scrubbers of the car wash. No potable plumbing connection is normally made to the car wash's scrubber system.
- After the pump broke down, the car wash owner was able to continue operation by connecting a 2 in. (50 mm) hose section temporarily between the potable supply within the car wash and the scrubber cycle piping.

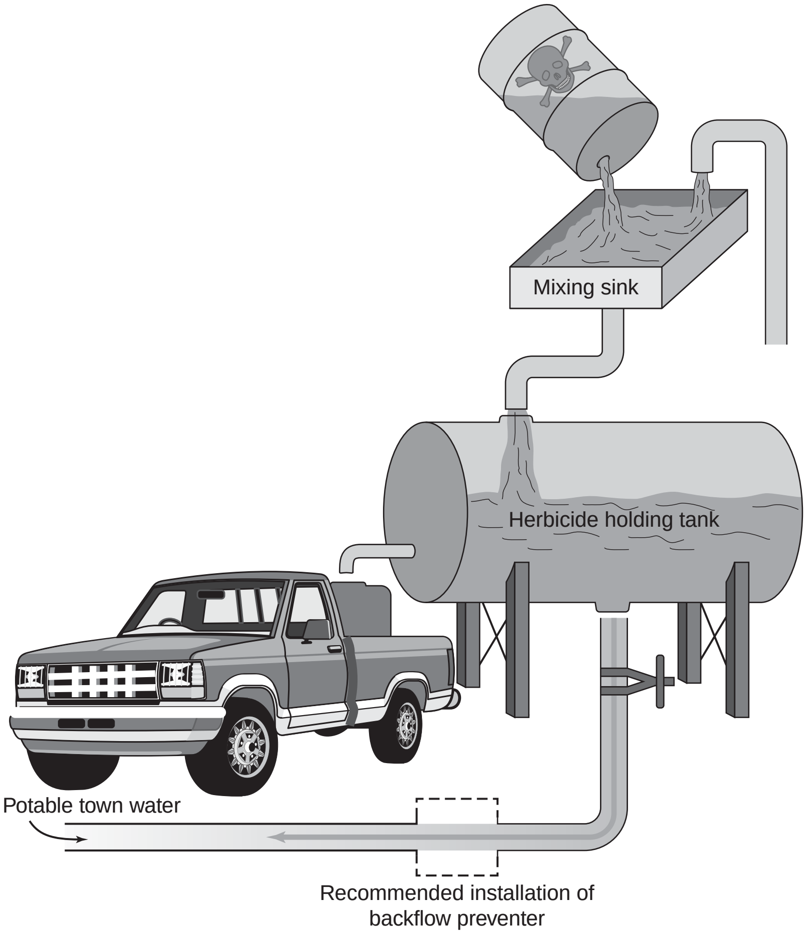


Figure 9.5 Paraquat in the water system.

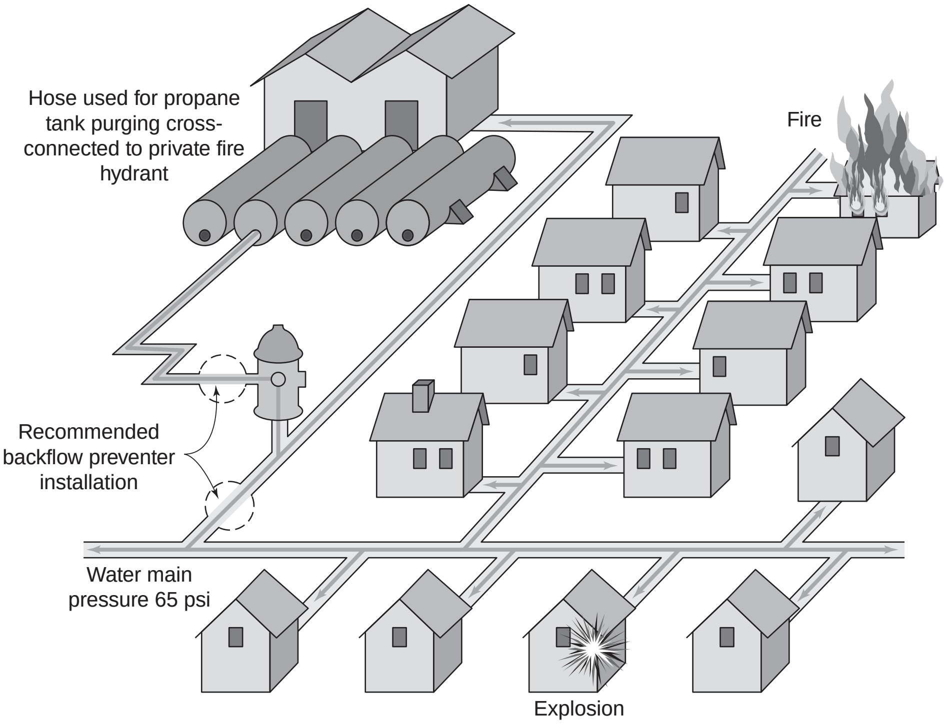


Figure 9.6 Propane gas in the water mains. Conversion factor: 1 psi = 6.94 kPa.

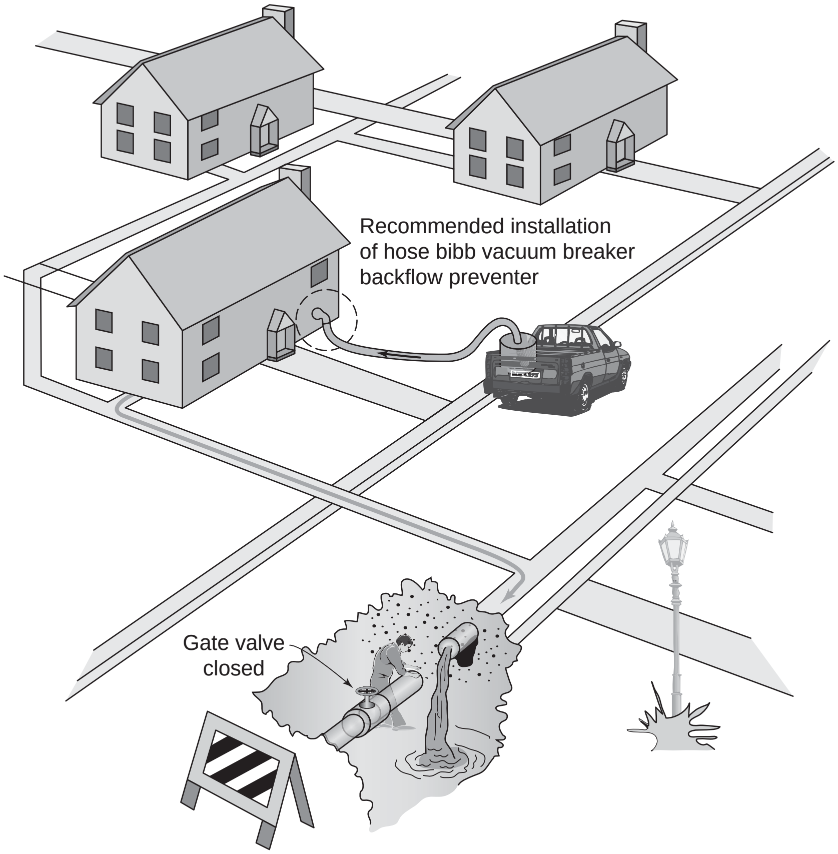


Figure 9.7 Chlordane and heptachlor in a city in Pennsylvania.

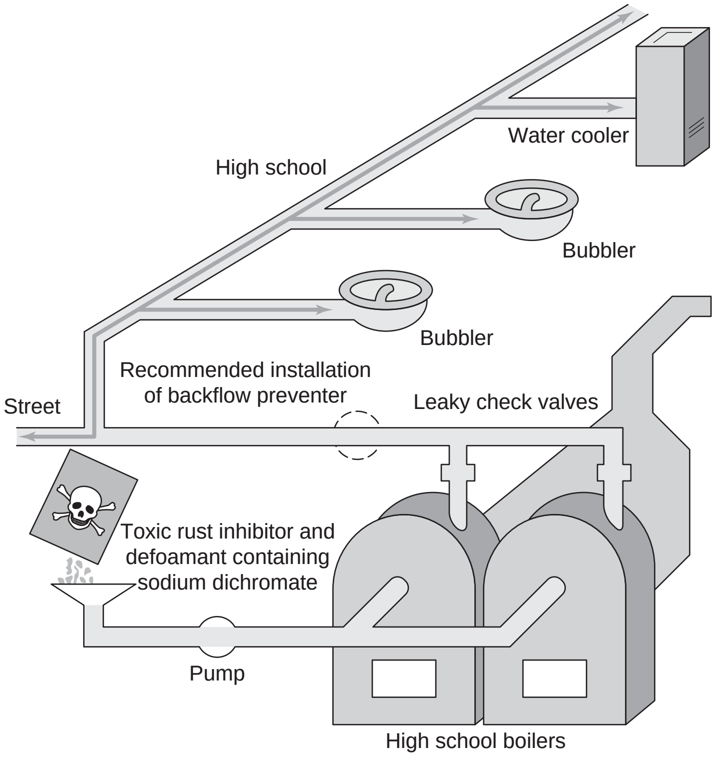


Figure 9.8 Boiler water enters high school drinking water.

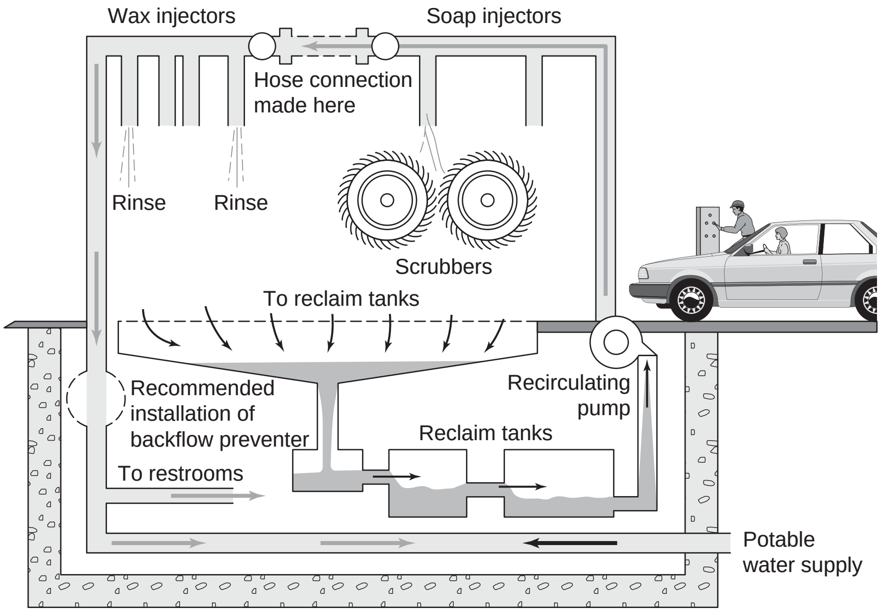


Figure 9.9 Car wash water enters the street water main.

- On Monday, February 12, the owner repaired the high-pressure pump and resumed normal car wash operations. The 2 in. (50 mm) hose connection (cross-connection) was not removed!
- Because of the cross-connection, the newly repaired high-pressure pump promptly pumped a large quantity of the reclaimed wash/rinse water out of the car wash and into a 12 in. (300 mm) water main in the street. This in turn was delivered to the many residences and commercial establishments connected to the water main (see Fig. 9.9).

Within 24 hours of the incident, the owner of the car wash had installed a 2 in. (50 mm) reduced pressure principle backflow preventer on his water service and all car wash establishments in Seattle that used a wash water reclaim system were notified of the state requirement for backflow prevention.

9.2.10 Health Problems Due to Cross-Connection in an Office Building

A cross-connection incident that occurred in a modern seven-story office building located in a large city in New Hampshire resulted in numerous cases of nausea, diarrhea, loss of work time, and employee complaints as to the poor quality of the water.

On a Saturday, a large fire occurred two blocks away from a seven-story office building in a large New Hampshire city. On Sunday, the maintenance crew of the office building arrived to perform the weekly cleaning, and after drinking the water from the drinking fountains, and sampling the coffee from the coffee machines, they noticed that the water smelled rubbery and had a strong bitter taste. On notifying the Manchester Water Company, water samples were taken and preliminary analysis disclosed that the contaminants found were not the typical contaminants associated with fire line disturbances. Investigating teams suspected that either the nearby fire could have siphoned contaminants from adjacent buildings into the water mains or the contamination could have been caused by a plumbing deficiency occurring within the seven-story building itself.

The pH levels of the building water indicated that an injection of chemicals had probably taken place within the seven-story building. Tracing of the water lines within the building pinpointed a 10,000 gal (37,850 L) hot water storage tank that was used for heat storage in the solar heating system. It did not have any backflow protection on the makeup supply line. As the storage tank pressure increased above the supply pressure, as a result of thermal expansion, the potential for backpressure backflow was present. Normally, this would not occur because a boost pump in the supply line would maintain the supply pressure to the storage tank at a greater pressure than the highest tank pressure. The addition of rust-inhibiting chemicals to this tank greatly increased the degree of hazard of the liquid. Unfortunately, at the same time that the fire took place, the pressure in the water mains was reduced to a dangerously low pressure and the low-pressure cutoff switches simultaneously shut off the storage tank booster pumps. This combination allowed the boiler water, together with its chemical contaminants, the opportunity to enter the potable water supply within the building (see Fig. 9.10). When normal pressure was reestablished in the water mains, the booster pumps kicked in, and the contaminated water was delivered throughout the building.

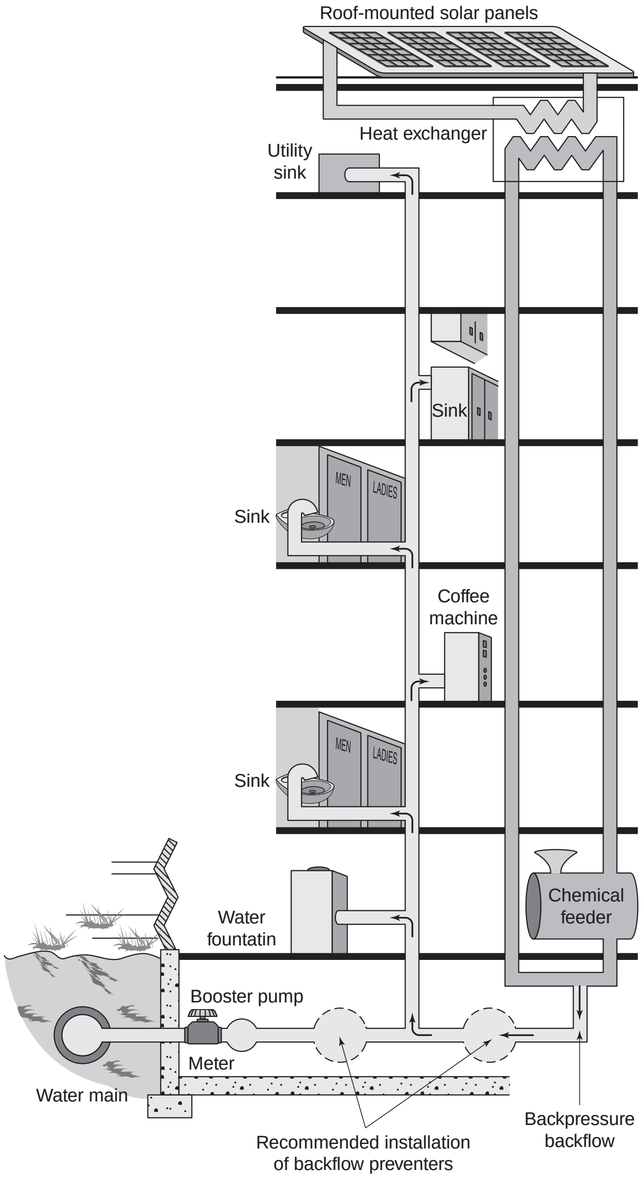


Figure 9.10 Cross-connection in a modern seven-story office building in New Hampshire.

9.3 THEORY OF BACKFLOW AND BACKSIPHONAGE

A cross-connection is the link or channel connecting a source of pollution with a potable water supply. The polluting substance, in most cases a liquid, tends to enter the potable supply if the net force acting on the liquid acts in the direction of the potable supply. Two factors are therefore essential for backflow. First, there must be a link between the two systems. Second, the resultant force must be toward the potable supply.

An understanding of the principles of backflow and backsiphonage requires an understanding of the terms frequently used in their discussion. Force, unless completely resisted, will produce motion. Weight is a type of force resulting from gravitational attraction. *Pressure* (P) is a force-per-unit area, such as lb/in.^{2} (psi) or kN/m^2 (kPa). *Atmospheric pressure* is the pressure exerted by the weight of the atmosphere above Earth.

Pressure may be referred to using an absolute scale, lb/in.^{2} absolute (psia) or kN/m^2 absolute (kPa absolute). Pressure may also be referred to using a gauge scale, lb/in.^{2} gauge (psig) or kN/m^2 gauge (kPa gauge). Absolute pressure and gauge pressure are related. Absolute pressure is equal to the gauge pressure plus the atmospheric pressure. At sea level the atmospheric pressure is 14.7 psia using the US customary units. Thus,

$$P_{\text{absolute}} = P_{\text{gauge}} + 14.7\ \text{psi} \quad \text{(US customary units)} \qquad \textbf{(9.1)}$$

or

$$P_{\text{gauge}} = P_{\text{absolute}} - 14.7\ \text{psi} \quad \text{(US customary units)} \qquad \textbf{(9.2)}$$

In the SI units where P_{absolute} and P_{gauge} are in kPa ($1\ \text{kPa} = 1000\ \text{Pa} = 1000\ \text{N/m}^2 = 1\ \text{kN/m}^2$) their relationship can be given as follows:

$$P_{\text{absolute}} = P_{\text{gauge}} + 102\ \text{kPa} \quad \text{(SI units)} \qquad \textbf{(9.3)}$$

or

$$P_{\text{gauge}} = P_{\text{absolute}} - 102\ \text{kPa} \quad \text{(SI units)} \qquad \textbf{(9.4)}$$

In essence then, absolute pressure is the total pressure. Gauge pressure is simply the pressure read on a gauge. If there is no pressure on the gauge other than atmospheric, the gauge would read zero. Then the absolute pressure would be equal to 14.7 psi (102 kPa), which is the atmospheric pressure.

The term *vacuum* indicates that the absolute pressure is less than the atmospheric pressure and that the gauge pressure is negative. A complete or total vacuum would mean a pressure of 0 psia or −14.7 psig (−102 kPa gauge). Because it is impossible to produce a total vacuum, the term *vacuum*, as used in the text, will mean all degrees of partial vacuum. In a partial vacuum, the pressure would range from slightly less than 14.7 psia (0 psig) to slightly greater than 0 psia (−14.7 psig).

Backsiphonage results in fluid flow in an undesirable or reverse direction. It is caused by atmospheric pressure exerted on a pollutant liquid such that it forces the pollutant toward a potable water supply system that is under a vacuum. Backflow, although literally meaning any type of reversed flow, refers to the flow produced by the differential pressure existing between two systems, both of which are at pressures greater than atmospheric.

9.3.1 Water Pressure

For an understanding of the nature of pressure and its relationship to water depth, consider the pressure exerted on the base of a cubic foot of water at sea level (see Fig. 9.11a).

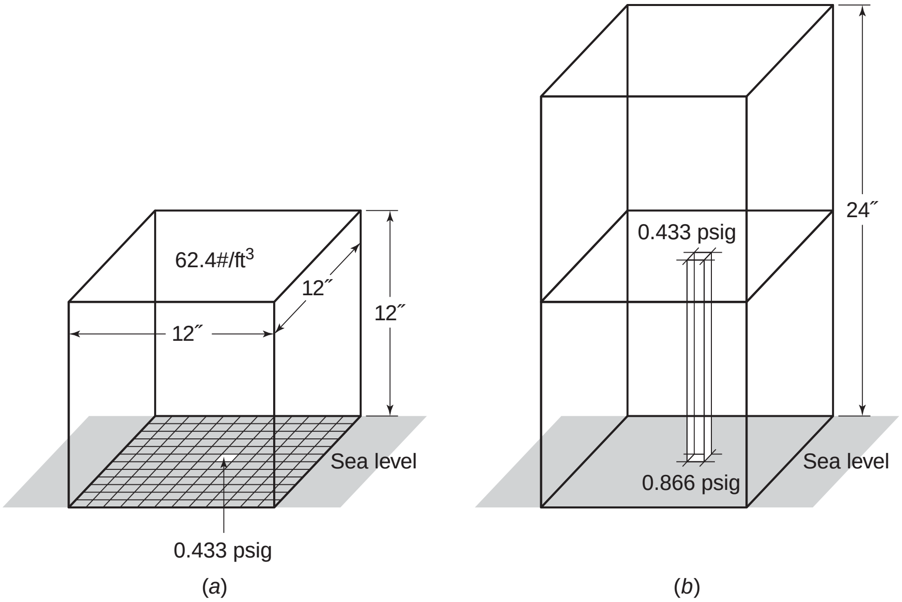


Figure 9.11 Pressures exerted by (a) 1 ft (0.3048 m) of water and (b) 2 ft (0.6096 m) of water at sea level. Conversion factors:
1# = 1 b = 0.454 kg; 1″ = 1 in. = 25.4 mm; 1 psig = 6.94 kPa gauge pressure.

The average weight of a cubic foot of water (62.4 lb/ft^3) will exert a pressure of 62.4 lb/ft^2 (430 kPa) gauge. The base may be subdivided into 144 square inches with each subdivision being subjected to a pressure of 0.433 psig ($P_{\text{gauge}} = 3\,\text{kPa}$).

Suppose another 1 ft^3 (0.0283 m^3) of water was placed directly on top of the first (see Fig. 9.11b). The pressure on the top surface of the first cube, which was originally atmospheric, or 0 psig, would now be 0.433 psig ($P_{\text{gauge}} = 3\,\text{kPa}$) as a result of the superimposed cubic foot of water. The pressure of the base of the first cube would also be increased by the same amount of 0.866 psig ($P_{\text{gauge}} = 6\,\text{kPa}$), or two times the original pressure.

If this process were repeated with a third cubic foot of water, the pressures at the base of each cube would be 1.299, 0.866, and 0.433 psig ($P_{\text{gauge}} = 9, 6$ and $3\,\text{kPa}$), respectively.

It is evident that pressure varies with depth below a free water surface; in general, each foot (0.3048 m) of elevation change, within a liquid, changes the pressure by an amount equal to the weight-per-unit area of 1 ft (0.3048 m) of the liquid. The rate of increase for water is 0.433 psi/ft (9.84 kPa/m) of depth.

Frequently water pressure is referred to using the terms *pressure head* or just *head* and is expressed in units of feet of water. One foot (0.3048 m) of head would be equivalent to the pressure produced at the base of a column of water 1 ft (0.3048 m) in depth. One foot (0.3048 m) of head or 1 ft (0.3048 m) of water is equal to 0.433 psig ($P_{\text{gauge}} = 3\,\text{kPa}$). One hundred feet (30.48 m) of head is equal to 43.3 psig ($P_{\text{gauge}} = 300\,\text{kPa}$).

9.3.2 Siphon Theory

Figure 9.12a depicts the atmospheric pressure on a water surface at sea level. An open tube is inserted vertically into the water; atmospheric pressure, which is 14.7 psia ($P_{\text{absolute}} = 102\,\text{kPa}$), acts equally on the surface of the water within the tube and on the outside of the tube.

If, as shown in Fig. 9.12b, the tube is capped and a vacuum pump is used to evacuate all the air from the sealed tube, a vacuum with a pressure of 0 psia ($P_{\text{absolute}} = 0\,\text{kPa}$) is created within the tube. Because the pressure at any point in a static fluid is dependent on the height of that point above a reference line, such as sea level, it follows that the pressure within the tube at sea level must still be 14.7 psia ($P_{\text{absolute}} = 102\,\text{kPa}$). This is equivalent to the pressure at the base of a column of water 33.9 ft (10.3 m) high. With the column open at the base, water would rise to fill the column to a depth of 33.9 ft (10.3 m). In other words, the weight of the atmosphere at sea level exactly balances the weight of a column of water 33.9 ft (10.3 m) in height. The absolute pressure within the column of water in Fig. 9.12b at a height of 11.5 ft (3.5 m) is equal to 9.7 psia ($P_{\text{absolute}} = 67.3\,\text{kPa}$). This is a partial vacuum with an equivalent gauge pressure of –5.0 psig ($P_{\text{gauge}} = -34.7\,\text{kPa}$).

As a practical example, assume the water pressure at a closed faucet on the top of a 100-ft (30.48-m)-high building to be 20 psig ($P_{\text{gauge}} = 138.8\,\text{kpa}$)); the pressure on the ground floor would then be 63.3 psig ($P_{\text{gauge}} = 439.3\,\text{kPa}$). If the pressure at the ground were to drop suddenly to 33.3 psig ($P_{\text{gauge}} = 231.1\,\text{kPa}$) due to a heavy fire demand in the area, the pressure at the top would be reduced to −10 psig ($P_{\text{gauge}} = -69.4\,\text{kPa}$). If the building water system were airtight, the water would remain at the level of the faucet because of the partial vacuum created by the drop in pressure. If the faucet were opened, however, the vacuum would be broken and the water level would drop to a height of 77 ft (23.47 m) above the ground. Thus, the atmosphere was supporting a column of water 23 ft (7 m) high.

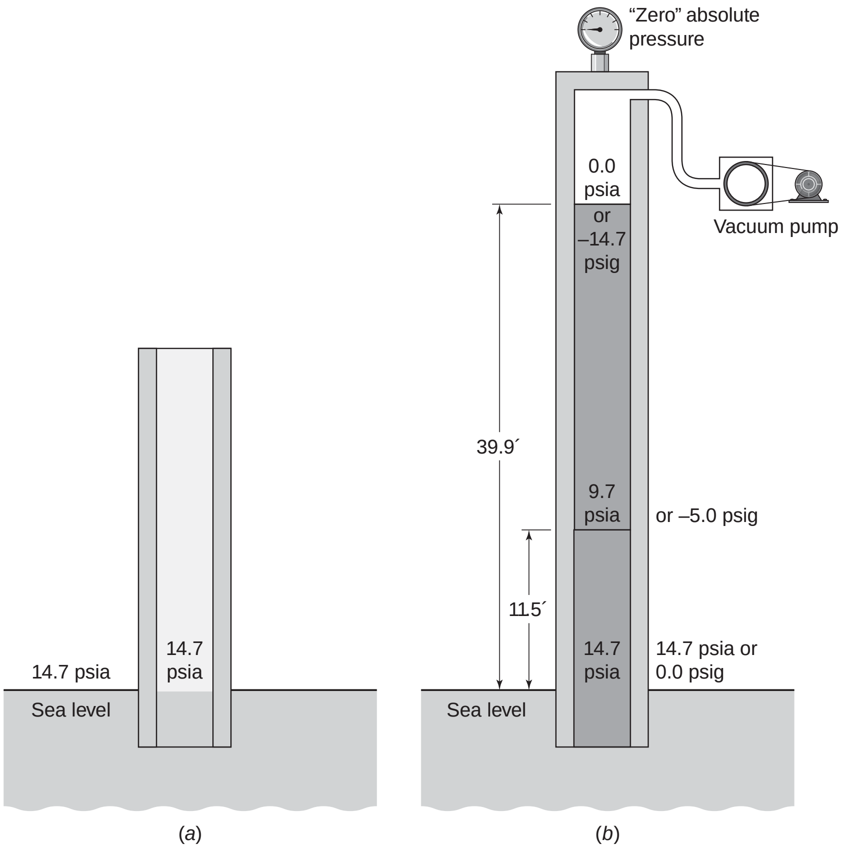


Figure 9.12 (a) Pressure on the free surface of a liquid at sea level; (b) effect of evacuating air from a column. Conversion factors: 1′ = 1 ft = 0.3048 m; 1 psia = 6.94 kPa absolute pressure; 1 psig = 6.94 kPa gauge pressure.

Figure 9.13a is a diagram of an inverted U-tube that has been filled with water and placed in two open containers at sea level. If the open containers are placed so that the liquid levels in each container are at the same height, a static state will exist; and the pressure at any specified level in either leg of the U-tube will be the same. The equilibrium condition is altered by raising one of the containers so that the liquid level in one container is 5 ft (1.52 m) above the level of the other (see Fig. 9.13b). Because both containers are open to the atmosphere, the pressure on the liquid surfaces in each container will remain at 14.7 psia ($P_{\text{absolute}} = 102$ kPa).

If it is assumed that a static state exists, momentarily, within the system shown in Fig. 9.13b, the pressure in the left tube at any height above the free surface in the left container can be calculated. The pressure at the corresponding level in the right tube above the free surface in the right container can also be calculated.

As shown in Fig. 9.13b, the pressure at all levels in the left tube would be less than at corresponding levels in the right tube. In this case, a static condition cannot exist because fluid will flow from the higher pressure to the lower pressure; the flow would be from the right tank to the left tank. This arrangement is referred to as a *siphon*. The crest of a siphon cannot be higher than 33.9 ft (10.3 m) above the upper liquid level, because the atmosphere cannot support a column of water greater in height than 33.9 ft (10.3 m).

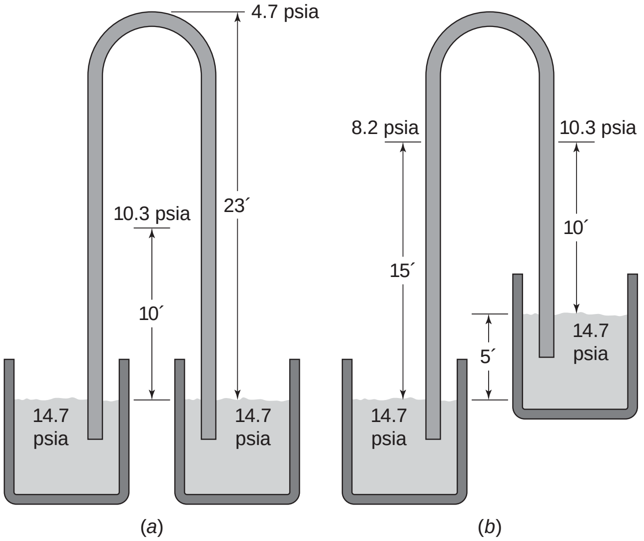


Figure 9.13 Pressure relationships in a continuous fluid system (a) at the same elevation; (b) at different elevations. Conversion factors: 1′ = 1 ft = 0.3048 m; 1 psia = 6.94 kPa absolute pressure.

Figure 9.14 illustrates how this siphon principle can be hazardous in a plumbing system. If the supply valve is closed, the pressure in the line supplying the faucet is less than the pressure in the supply line to the bathtub. Flow will occur, therefore, through siphonage, from the bathtub to the open

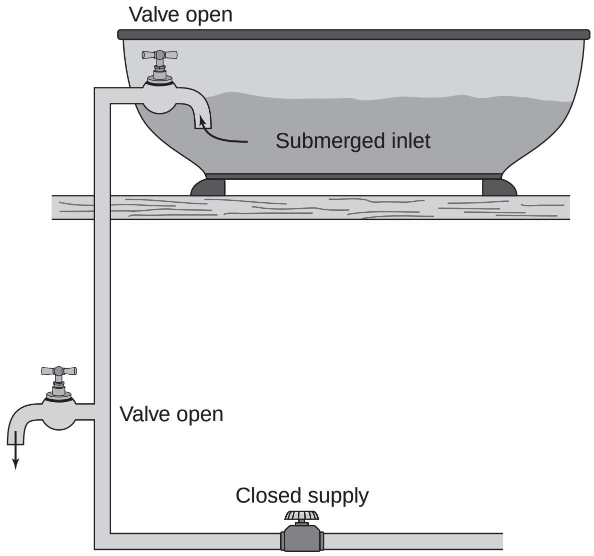


Figure 9.14 Backsiphonage in a plumbing system.

faucet. This siphon action has been produced by reduced pressures resulting from a difference in the water levels at two separated points within a continuous fluid system.

Reduced pressure may also be created within a fluid system as a result of fluid motion. One of the basic principles of fluid mechanics is the *principle of conservation of energy*. Based on this principle, it can be shown that as a fluid accelerates (velocity head increases), as shown in Fig. 9.15 and the following expression, the pressure head (P/γ) is reduced to maintain the same total head:

$$H = z + \frac{P}{\gamma} + \frac{v^2}{2g} \tag{9.5}$$

where

H = total head, ft (m)
Z = elevation, ft (m)
$\frac{P}{\gamma}$ = pressure head, ft (m)
$\frac{v^2}{2g}$ = velocity head, ft (m)

Conversely, it can be shown that as water flows through a constriction ($A_2 < A_1$) such as a converging section of pipe, the velocity of the water increases ($v_2 < v_1$):

$$Q = v_1 A_1 = v_2 A_2 \tag{9.6}$$

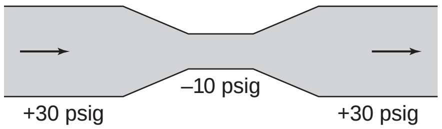


Figure 9.15 Negative pressure created by constricted flow. Conversion factor: 1 psig = 6.94 kPa gauge pressure.

where

Q = flow rate, ft^3/s (m^3/s)
v_1 = water velocity at section 1, ft/s (m/s)
v_2 = water velocity at section 2, ft/s (m/s)
A_1 = area of section 1, ft^2 (m^2)
A_2 = area of section 2, ft^2 (m^2)

As a result, the pressure is reduced. Under such conditions, *negative pressures* can develop in a pipe. The simple aspirator is based on this principle. If this point of reduced pressure is linked to a source of pollution, backsiphonage of the pollutant can occur.

One of the common occurrences of dynamically reduced pipe pressures is found on the suction side of a pump. In many cases similar to the one illustrated in Fig. 9.16, the line supplying the booster pump is undersized or does not have sufficient pressure to deliver water at the rate at which the pump normally operates. The rate of flow in the pipe may be increased by a further reduction in pressure at the pump intake. This often results in the creation of negative pressure at the pump intake. This negative pressure may become low enough in some cases to cause vaporization of the water in the line. Actually, in Fig. 9.16 illustration, flow from the source of pollution would occur when pressure on the suction side of the pump is less than pressure of the pollution source, but this is backflow, which will be discussed below.

The preceding discussion has described some of the means by which negative pressures may be created and which frequently occur to produce backsiphonage. In addition to the negative pressure or reversed force necessary to cause backsiphonage and backflow, there must also be the cross-connection or connecting link between the potable water

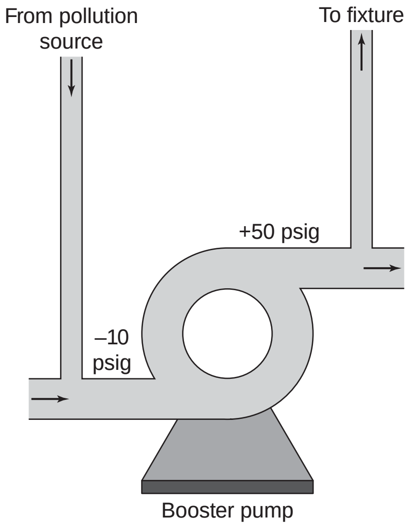


Figure 9.16 Dynamically reduced pipe pressure. Conversion factor: 1 psig = 6.94 kPa gauge pressure.

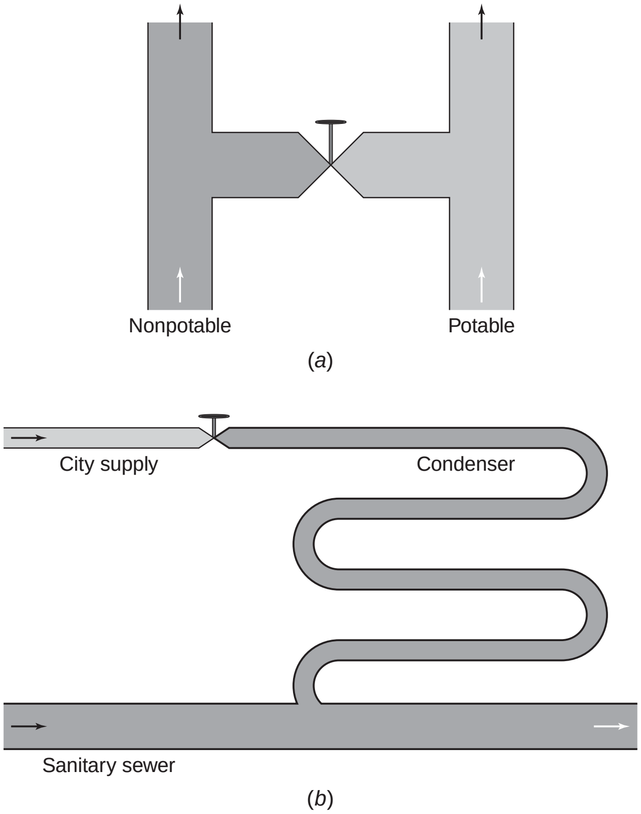


Figure 9.17 Valved connections between potable water and (a) nonpotable fluid; (b) sanitary sewer.

supply and the source of pollution. Two basic types of connections are used in piping systems:

1. The solid pipe with a valved connection
2. The submerged inlet

Figures 9.17a and 9.17b illustrate solid connections. This type of connection is often installed where it is necessary to lay an auxiliary piping system from the potable source. It is a direct connection of one pipe to another pipe or receptacle. Solid pipe connections are often made to continuous or intermittent waste lines where it is assumed that the flow will be in one direction only. An example of this would be used cooling water from a water jacket or condenser as shown in Fig. 9.17b. This type of connection is usually detectable, but creating a concern on the part of the installer about the possibility of reversed flow is often more difficult.

Submerged inlets are found on many common plumbing fixtures and are sometimes necessary features of the fixtures if they are to function properly. Examples of this type of design are siphon-jet urinals or water closets, flushing rim slop sinks, and dental cuspidors. Old-style bathtubs and lavatories had supply inlets below the flood-level rims, but modern sanitary design has minimized or eliminated this hazard in new fixtures. Chemical and industrial process vats sometimes have submerged inlets where the water pressure is used as an aid in diffusion, dispersion, and agitation of the vat contents. Even though the supply pipe may come from the floor above the vat, backsiphonage can occur because the siphon action can raise a liquid such as water almost 34 ft (10.4 m). Some submerged inlets that are difficult to control are those that are not apparent until a significant change in water level occurs or where a supply may be conveniently extended below the liquid surface by means of a hose or auxiliary piping. A submerged inlet may be created in numerous ways, and its detection in some of these subtle forms may be difficult.

9.3.3 Backflow

Backflow refers to reversed flow due to backpressure other than siphonic action. Any interconnected fluid systems in which the pressure of one exceeds the pressure of the other may have flow from one to the other as a result of the pressure differential. The flow will occur from the zone of higher pressure to the zone of lower pressure. This type of backflow is of concern in buildings where two or more piping systems are maintained. The potable water supply is usually under pressure directly from the city water main. Occasionally, a booster pump is used. The auxiliary system is often pressurized by a centrifugal pump, although backpressure may be caused by gas or steam pressure from a boiler. A reversal in differential pressure may occur when pressure in the potable system drops, for some reason, to a pressure lower than that in the system to which the potable water is connected.

The most positive method of avoiding this type of backflow is the total or complete separation of the two systems. Other methods used involve the installation of mechanical devices. All methods require routine inspection and maintenance.

Dual piping systems are often installed for extra protection in the event of an emergency or possible mechanical failure of one of the systems. Fire protection systems are an example. Another example is the use of dual water connections to boilers. These installations are sometimes interconnected, thus creating a health hazard.

9.4 METHODS AND DEVICES FOR THE PREVENTION OF BACKFLOW AND BACKSIPHONAGE

A wide variety of devices are available that can be used to prevent backsiphonage and backpressure from allowing contaminated fluids or gases into a potable water supply system. Generally, the selection of the proper device to use is based on the degree of hazard posed by the cross-connection. Additional considerations are based on piping size, location, and the potential need to periodically test the devices to insure proper operation.

The six basic types of devices that can be used to correct cross-connections are as follows:

1. Air gaps
2. Barometric loops
3. Vacuum breakers—both atmospheric and pressure type
4. Double check valves with an intermediate atmospheric vent
5. Double check valve assemblies
6. Reduced pressure principle devices

In general, all manufacturers of these devices, with the exception of the barometric loop, produce them to one or more of three basic standards, thus ensuring that dependable devices are being utilized and marketed. The major standards in the industry are devised by the American Society of Sanitary Engineers (ASSE), the AWWA, and the University of California Foundation for Cross-Connection Control and Hydraulic Research.

9.4.1 Air Gap

Air gaps are nonmechanical backflow preventers that are very effective devices for use where either backsiphonage or backpressure conditions may exist. Their use is as old as piping and plumbing itself, although their design was standardized relatively recently. In general, the air gap must be twice the supply pipe diameter but never less than 1 in. (see Fig. 9.18a).

An air gap, although an extremely effective backflow preventer when used to prevent backsiphonage and backpressure conditions, does interrupt the piping flow with corresponding loss of pressure for subsequent use. Consequently, air gaps are primarily used at the end of the line service where reservoirs or storage tanks are desired. When contemplating the use of an air gap, here are some other considerations:

1. In a continuous piping system, each air gap requires the added expense of reservoirs and secondary pumping systems.
2. The air gap can be easily defeated in the event that the two-diameter (2D) requirement was purposely or inadvertently compromised. Excessive splash may be encountered in the event that higher than anticipated pressures or flows occur. The splash may be a cosmetic or true potential hazard—the simple solution being to reduce the 2D dimension by thrusting the supply pipe into the receiving funnel. By so doing, the air gap is defeated.
3. At an air gap, we expose the water to the surrounding air with its inherent bacteria, dust particles, and other airborne pollutants or contaminants. In addition, the aspiration effect of the flowing water can drag down surrounding pollutants into the reservoir or holding tank.
4. Free chlorine can come out of treated water as a result of the air gap and the resulting splash and churning effect as the water enters the holding tanks. This reduces the ability of the water to withstand bacteria contamination during long-term storage.
5. For these reasons, air gaps must be inspected as frequently as mechanical backflow preventers. They are not exempt from an in-depth cross-connection control program requiring periodic inspection of all backflow devices.

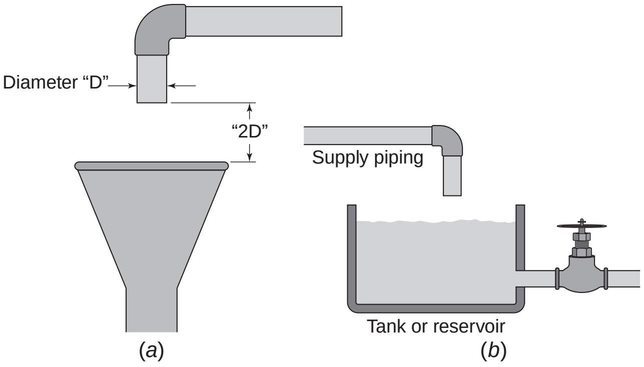


Figure 9.18 Air gaps.

Air gaps can be fabricated from commercially available plumbing components or purchased as separate units and integrated into plumbing and piping systems. An example of the use of an air gap is shown in Fig. 9.18b.

9.4.2 Barometric Loops

The barometric loop consists of a continuous section of supply piping that abruptly rises to a height of approximately 35 ft (10.6 m) and then returns back down to the originating level. It is a loop in the piping system that effectively protects against backsiphonage. It cannot be used to protect against backpressure.

Its operation, in the protection against backsiphonage, is based on the principle that a water column, at sea level pressure, cannot rise above 33.9 ft (10.3 m) (refer to Fig. 9.12b). In general, barometric loops are locally fabricated and are 35 ft (10.6 m) high (see Fig. 9.19).

9.4.3 Atmospheric Vacuum Breakers

These devices are among the simplest and least expensive mechanical types of backflow preventers and, when installed properly, can provide excellent protection against backsiphonage. They must not, however, be utilized to protect against backpressure conditions.

Construction usually consists of a polyethylene float that is free to travel on a shaft and seal in the uppermost

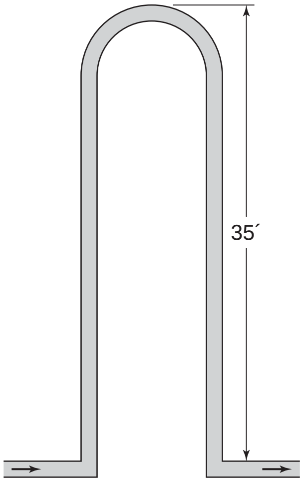


Figure 9.19 Barometric loop. Conversion factor: 1′ = 1 ft = 0.3048 m.

position against the atmosphere with an elastomeric disk. Water flow lifts the float, which then causes the disk to seal. Water pressure keeps the float in the upward sealed position. Termination of the water supply will cause the disk to drop down, venting the unit to the atmosphere and thereby opening downstream piping to atmospheric pressure, thus preventing backsiphonage. Figure 9.20a shows a typical atmospheric breaker.

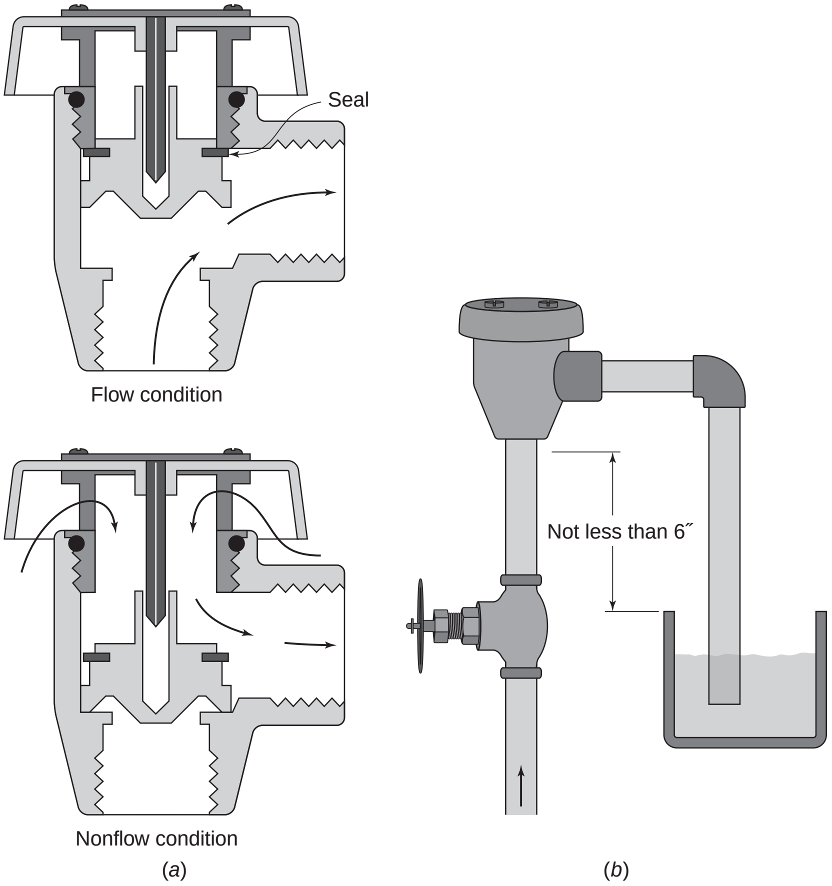
In general, these devices are available in 1/2 through 3 in. (12 through 75 mm) sizes and must be installed vertically, must not have shutoffs downstream, and must be installed at least 6 in. (152 mm) higher than the final outlet. They cannot be tested once they are installed in the plumbing system, but are, for the most part, dependable, trouble-free devices for backsiphonage protection.

Figure 9.20b shows the generally accepted installation requirements. Note that no shutoff valve is downstream of the device that would otherwise keep the atmospheric vacuum breaker under constant pressure.

9.4.4 Hose Bib Vacuum Breakers

Hose bib vacuum breakers are small devices that are a specialized application of the atmospheric vacuum breaker. They are generally attached to still cocks and in turn are connected to hose-supplied outlets such as garden hoses, slop sink hoses, and spray outlets. They consist of a spring-loaded check valve that seals against an atmospheric outlet when water supply pressure is turned on. Typical construction is shown in Fig. 9.21.

When the water supply is turned off, the device vents to the atmosphere, thus protecting against backsiphonage conditions. They should not be used as backpressure devices.



(*a*) (*b*)

Figure 9.20 Atmospheric vacuum breaker: (a) flow and nonflow conditions; (b) typical installation. Conversion factor: 1″ = 1 in. = 25.4 mm.

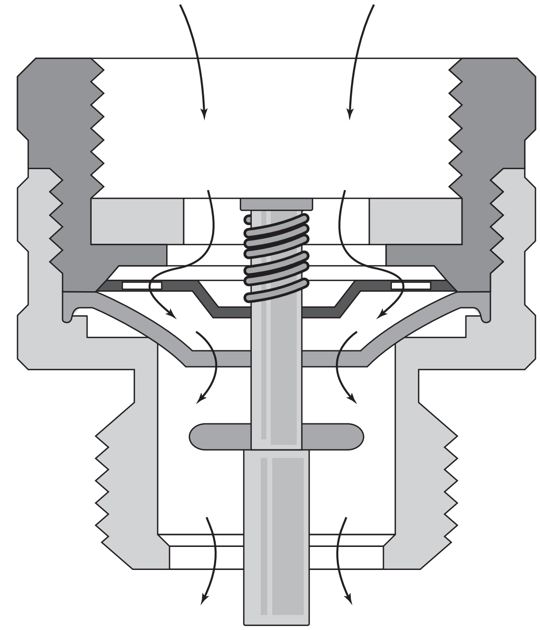


Figure 9.21 Hose bib vacuum breaker.

Manual drain options are available, together with tamper-proof versions.

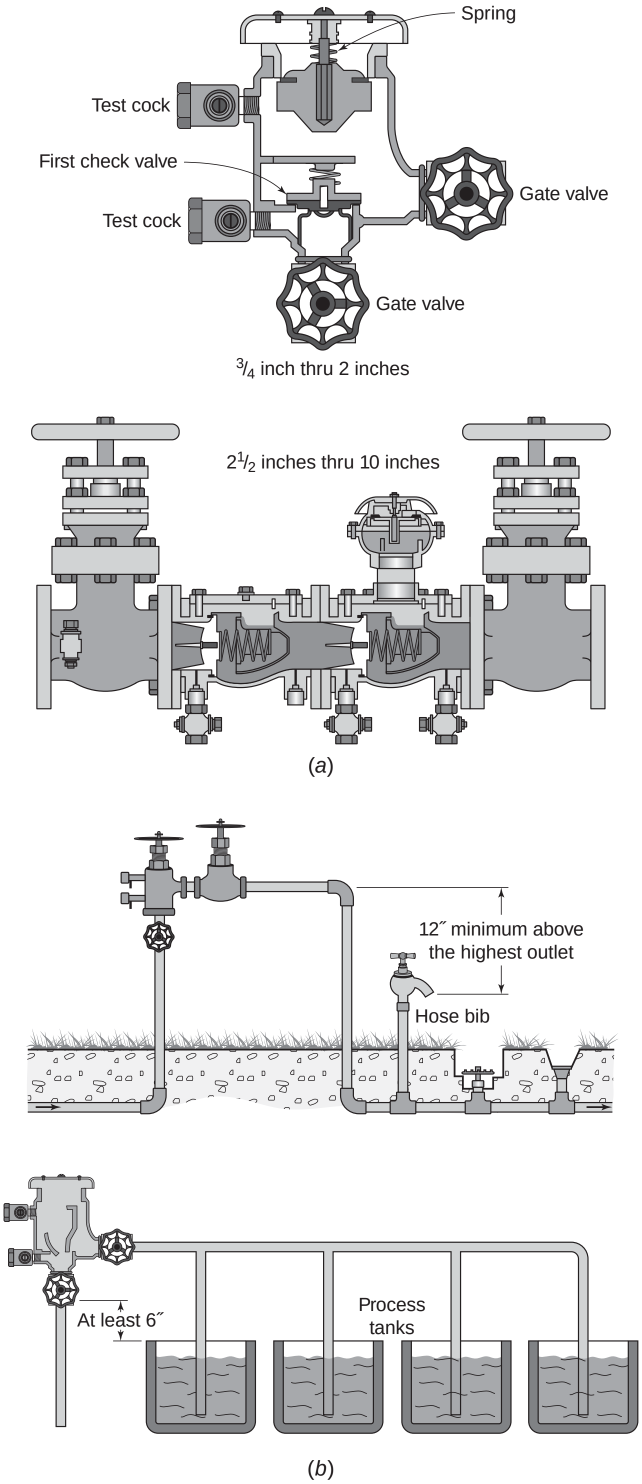
9.4.5 Pressure Vacuum Breakers

This device is an outgrowth of the atmospheric vacuum breaker and evolved in response to a need to have an atmospheric vacuum breaker that could be utilized under constant pressure and that could be tested in line. A spring on top of the disk and float assembly, two added gate valves, test cocks, and an additional first check were required to achieve this device (see Fig. 9.22a).

These units are available in $^{1}/_{2}$ through 10 in. (12 through 250 mm) sizes and have broad usage in the agriculture and irrigation market. Typical agricultural and industrial applications are shown in Fig. 9.22b.

Again, these devices may be used under constant pressure, but do not protect against backpressure conditions. As a result, installation must be at least 6–12 in. (150–300 mm) higher than the existing outlet.

A spill-resistant pressure vacuum breaker (SVB) is available that is a modification to the standard pressure vacuum breaker but specifically designed to minimize water spillage. Installation and hydraulic requirements are similar to the standard pressure vacuum breaker and the devices are recommended for internal use.



(*a*)

(*b*)

Figure 9.22 Pressure vacuum breaker: (a) types and (b) installations. Conversion factor: 1″ = 1 in. = 25.4 mm.

9.4.6 Double Check Valves with an Intermediate Atmospheric Vent

The need to provide a compact device in $^{1}/_{2}$ and $^{3}/_{4}$ in. (12 and 19 mm) pipe sizes that protects against moderate hazards, is capable of being used under constant pressure, and protects against backpressure resulted in this unique backflow preventer. Construction is basically a double check valve having an atmospheric vent located between the two checks (see Fig. 9.23a).

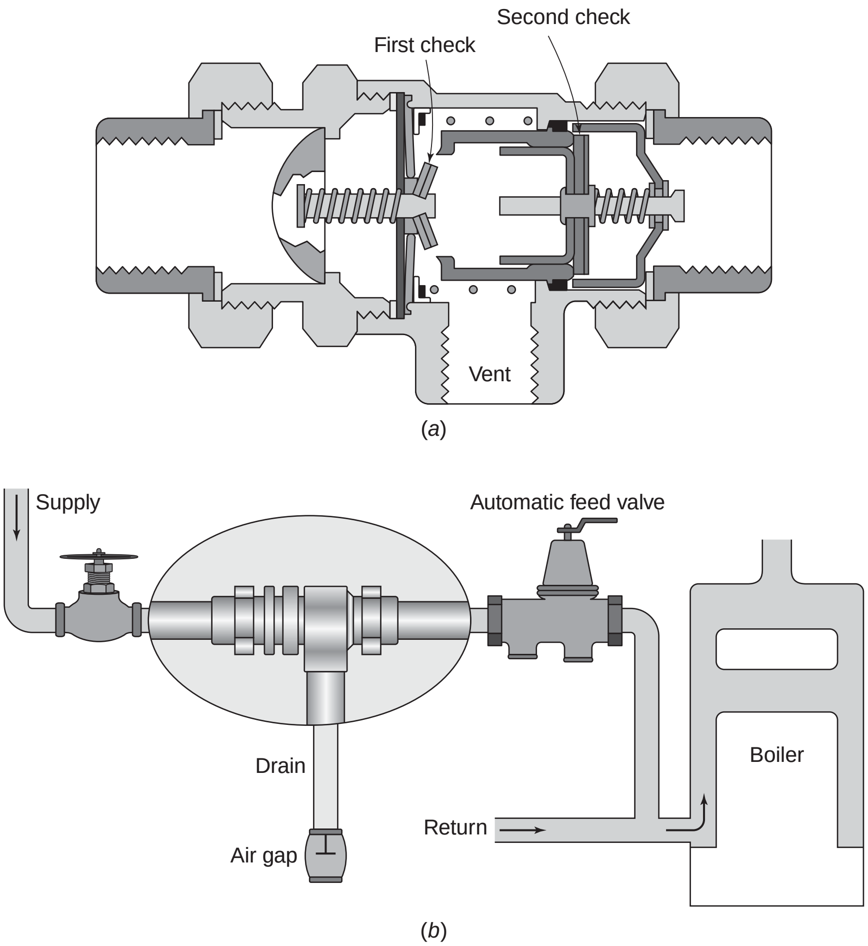


Figure 9.23 (a) Double check valve with atmospheric vent and (b) typical residential use of valve.

Line pressure keeps the vent closed, but zero supply pressure or backsiphonage will open the inner chamber to atmosphere. With this device, extra protection is obtained through the atmospheric vent capability. Figure 9.23b shows a typical use of the device on a residential boiler supply line.

9.4.7 Double Check Valves

A double check valve is essentially two single check valves coupled within one body and furnished with test cocks and two tightly closing gaze valves (see Fig. 9.24). The test capability feature gives this device a big advantage over the use of two independent check valves in that it can be readily tested to determine if either or both check valves are inoperative or fouled by debris. Each check is spring loaded closed and requires approximately a pound of pressure to open. The spring loading provides the ability to "bite" through small debris and still seal—a protection feature not prevalent in unloaded swing check valves.

Figure 9.24 shows a cross-section of a double check valve complete with test cocks. Double checks are commonly used to protect against low to medium hazard installations such as food processing steam kettles and apartment projects. They may be used under continuous pressure and protect against both backsiphonage and backpressure conditions.

9.4.8 Double Check Detector Check

This device is an outgrowth of the double check valve and is primarily utilized in fire line installations. Its purpose is to protect the potable supply line from possible contamination or pollution from fire line chemical additives, booster pump fire line backpressure, stagnant "black water" that sits in fire lines over extended periods of time, the addition of "raw" water through outside fire pumper connections (Siamese outlets), and the detection of any water movement in the fire line water due to fire line leakage or deliberate water theft. It consists of two spring-loaded check valves, a bypass assembly with water meter and double check valve, and two tightly closing gate valves (see Fig. 9.25).

The addition of test cocks makes the device testable to ensure proper operation of both the primary checks and the bypass check valve. In the event of very low fire line water usage (theft of water), the low pressure drop inherent in the

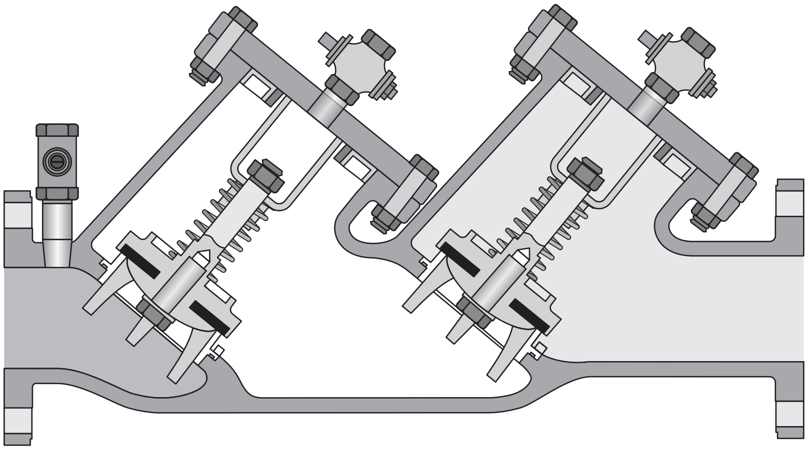


Figure 9.24 Double check valve.

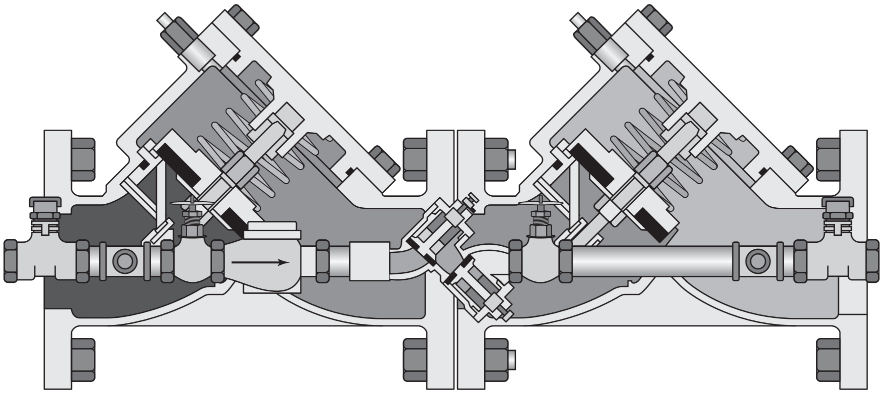


Figure 9.25 Double check detector check.

bypass system permits the low flow of water to be metered through the bypass system. In a high flow demand situation, such as that associated with deluge fire capability, the main check valves open, permitting high-volume, low restricted flow through the two large spring-loaded check valves.

9.4.9 Residential Dual Check

The need to furnish reliable and inexpensive backsiphonage and backpressure protection to individual residences resulted in the debut of the residential dual check. Protection of the main potable supply from household hazards such as home photography chemicals, toxic insect and garden sprays, and termite control pesticides used by exterminators reinforced a true need for such a device. Figure 9.26a shows a cutaway of the device.

This device is sized for $^1/_2$, $^3/_4$, and 1 in. (12, 19, and 25 mm) service lines and is installed immediately downstream of the water meter. The use of plastic check modules and elimination of test cocks and gate valves keep the cost reasonable while providing good, dependable protection. Typical installations are shown in Fig. 9.26b.

9.5 REDUCED PRESSURE PRINCIPLE BACKFLOW PREVENTER

Maximum protection is achieved against backsiphonage and backpressure conditions utilizing reduced pressure principle backflow preventers. These devices are essentially modified double check valves with an atmospheric vent capability placed between the two checks and designed such that this “zone” between the two checks is always kept at least 2 psi less than the supply pressure (see Fig. 9.27). With this design criterion, the reduced pressure principle backflow preventer can provide protection against backsiphonage and backpressure when both the first and second checks become fouled. They can be used under constant pressure and at high hazard installations. They are furnished with test cocks and gate valves to enable testing and are available in $^3/_4$ through 10 in. (19 through 250 mm) sizes.

The principles of operation of a reduced pressure principle backflow preventer are as follows: flow from the left enters the central chamber against the pressure exerted by the loaded check valve 1. The supply pressure is then reduced by a predetermined amount. The pressure in the central chamber is maintained lower than the incoming supply pressure

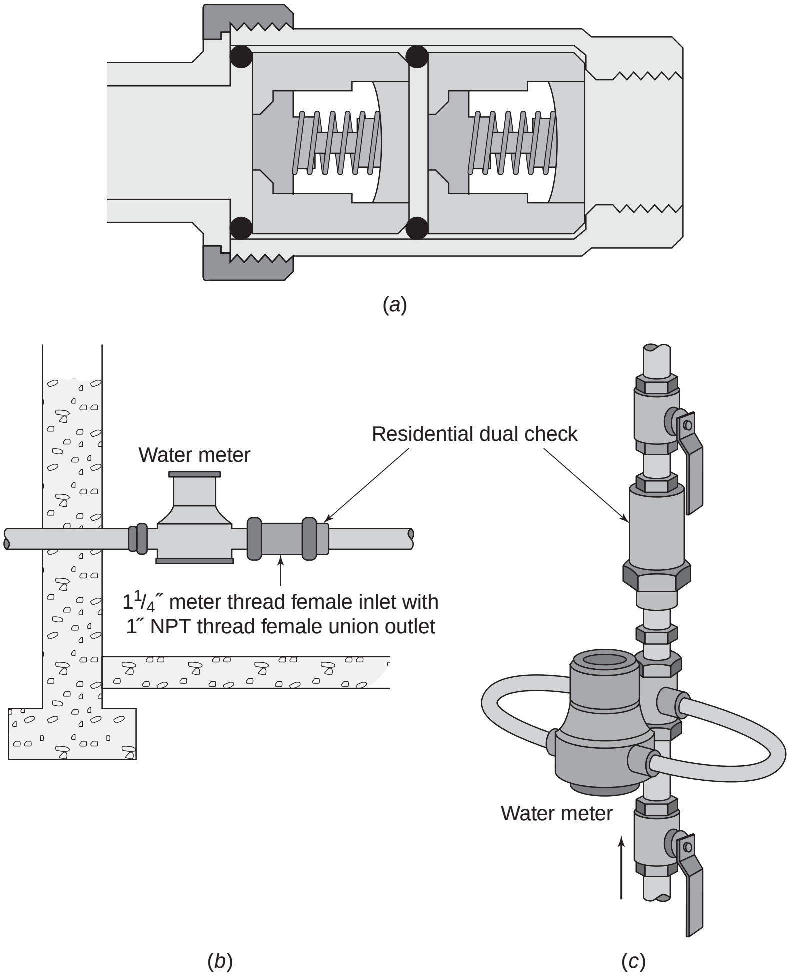


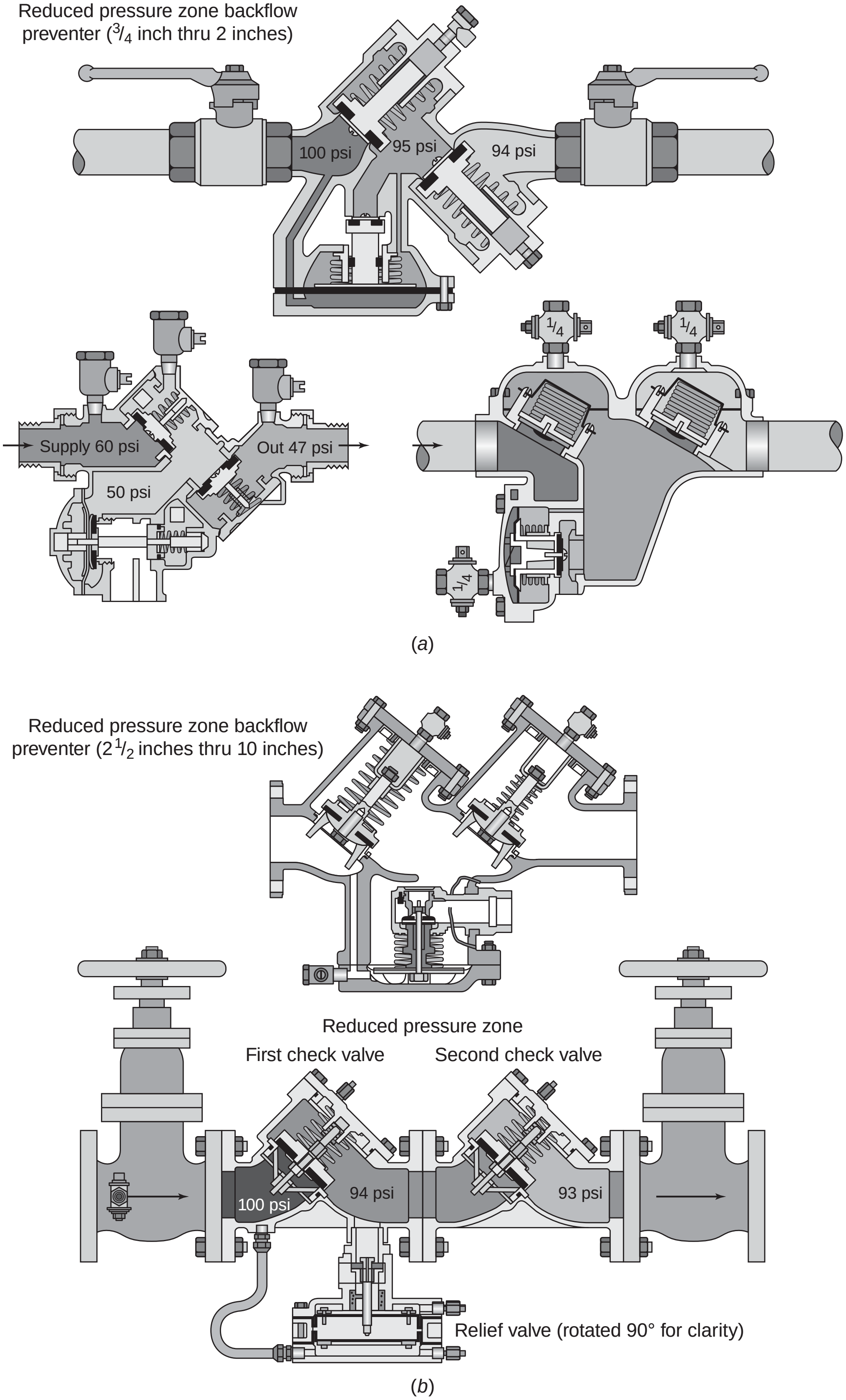
Figure 9.26 (a) Residential dual check, (b) residential installation, and (c) copper horn. Conversion factor: 1″ = 1 in. = 25.4 mm.

through the operation of the relief valve, which discharges to the atmosphere whenever the central chamber pressure approaches within a few psi of the inlet pressure. Check valve 2 is lightly loaded to open with a pressure drop of 1 psi (7 kPa) in the direction of flow and is independent of the pressure required to open the relief valve. In the event that the pressure increases downstream from the device, tending to reverse the direction of flow, check valve 2 closes, preventing backflow. Because all valves may leak as a result of wear or obstruction, the protection provided by the check valves is not considered sufficient. If some obstruction prevents check valve 2 from closing tightly, the leakage back into the central chamber would increase the pressure in this zone, the relief valve would open, and flow would be discharged to the atmosphere.

When the supply pressure drops to the minimum differential required to operate the relief valve, the pressure in the central chamber should be atmospheric; if the inlet pressure should become less than atmospheric pressure, the relief valve should remain fully open to the atmosphere to discharge any backflow water that is the result of backpressure and leakage of check valve 2.

Malfunctioning of one or both of the check valves or relief valve should always be indicated by a discharge of water from the relief port. Under no circumstances should plugging of the relief port be permitted because the device depends on an open port for safe operation. The pressure loss through the device may be expected to average between 10 and 20 psi (69.4 and 138.8 kPa) within the normal range of operation, depending on the size and flow rate of the device.

Reduced pressure principle backflow preventers are commonly installed on high hazard installations such as plating plants, where they protect primarily against backsiphonage potential; car washes where they protect against backpressure conditions; and funeral parlors and hospital autopsy rooms. The reduced pressure principle backflow preventer forms the backbone of cross-connection control programs. Because it is utilized to protect against backsiphonage from high hazard installations, and because high hazard installations are the first consideration in protecting public health and safety, these devices are installed in large quantities over a broad range of plumbing and water works installations. Figure 9.28 shows typical installations of these devices.



(*a*)

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Figure 9.27 Types of reduced pressure zone backflow preventers. Conversion factors: 1 inch = 25.4 mm; 1 psi = 6.94 kPa.

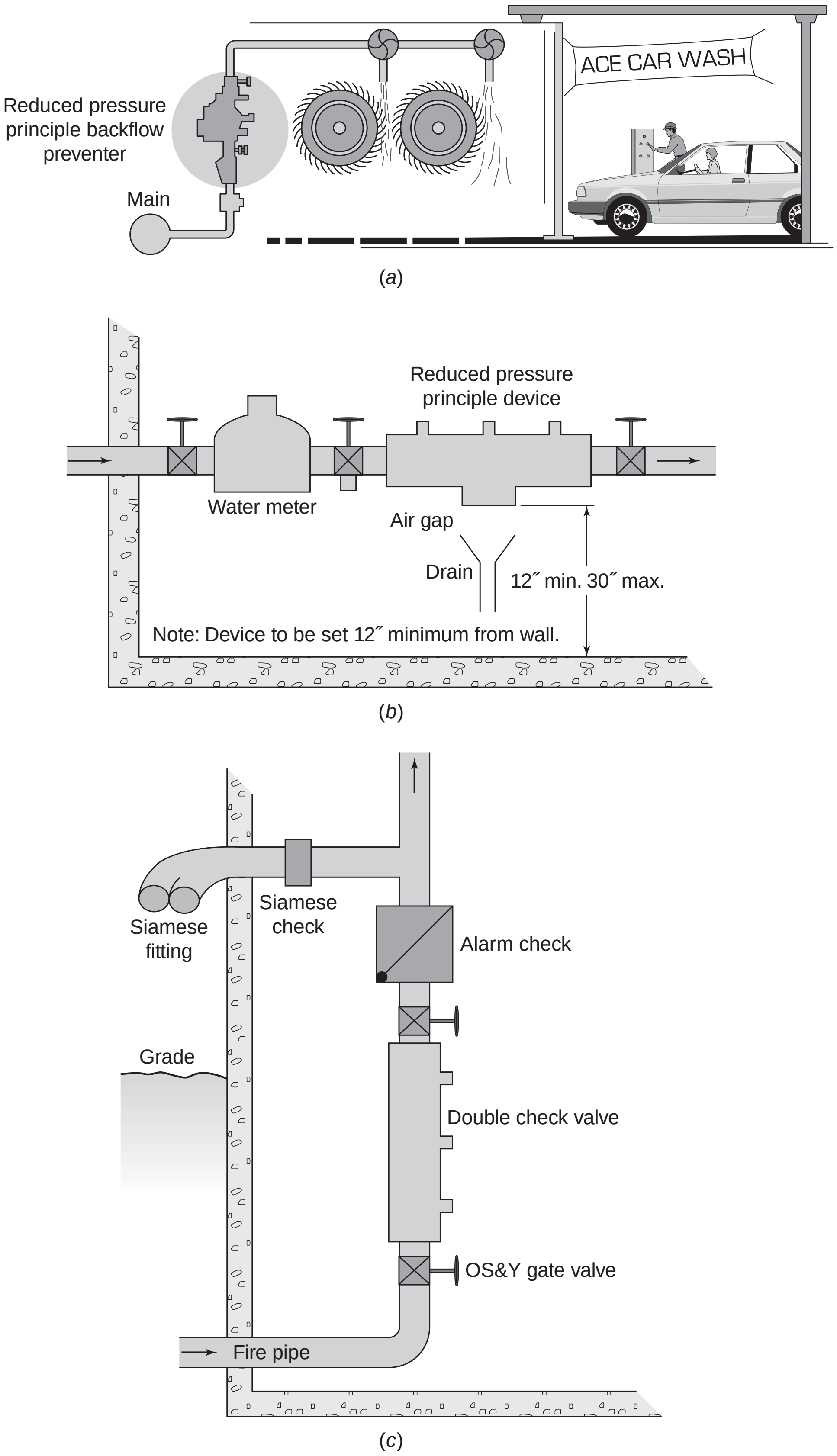

Figure 9.28 Reduced pressure device installations in (a) car wash, (b) water supply line, and (c) fire line. Conversion factor: 1″ = 1 in. = 25.4 mm.

9.6 ADMINISTRATION OF A CROSS-CONNECTION CONTROL PROGRAM

9.6.1 Responsibility

Under the provisions of the Safe Drinking Water Act, the federal government has established, through the US Environmental Protection Agency (U.S. EPA), national standards for safe drinking water. The states are responsible for the enforcement of these standards as well as the supervision of public water supply systems and the sources of drinking water. The water purveyor (supplier) is held responsible for compliance with the provisions of the Safe Drinking Water Act, to include a warranty that water quality provided by its operation is in conformance with the U.S. EPA standards at the source and is delivered to the customer without the quality being compromised as a result of its delivery through the distribution system.

Figure 9.29 depicts several options that are open to a water purveyor when considering cross-connection protection to commercial, industrial, and residential customers.

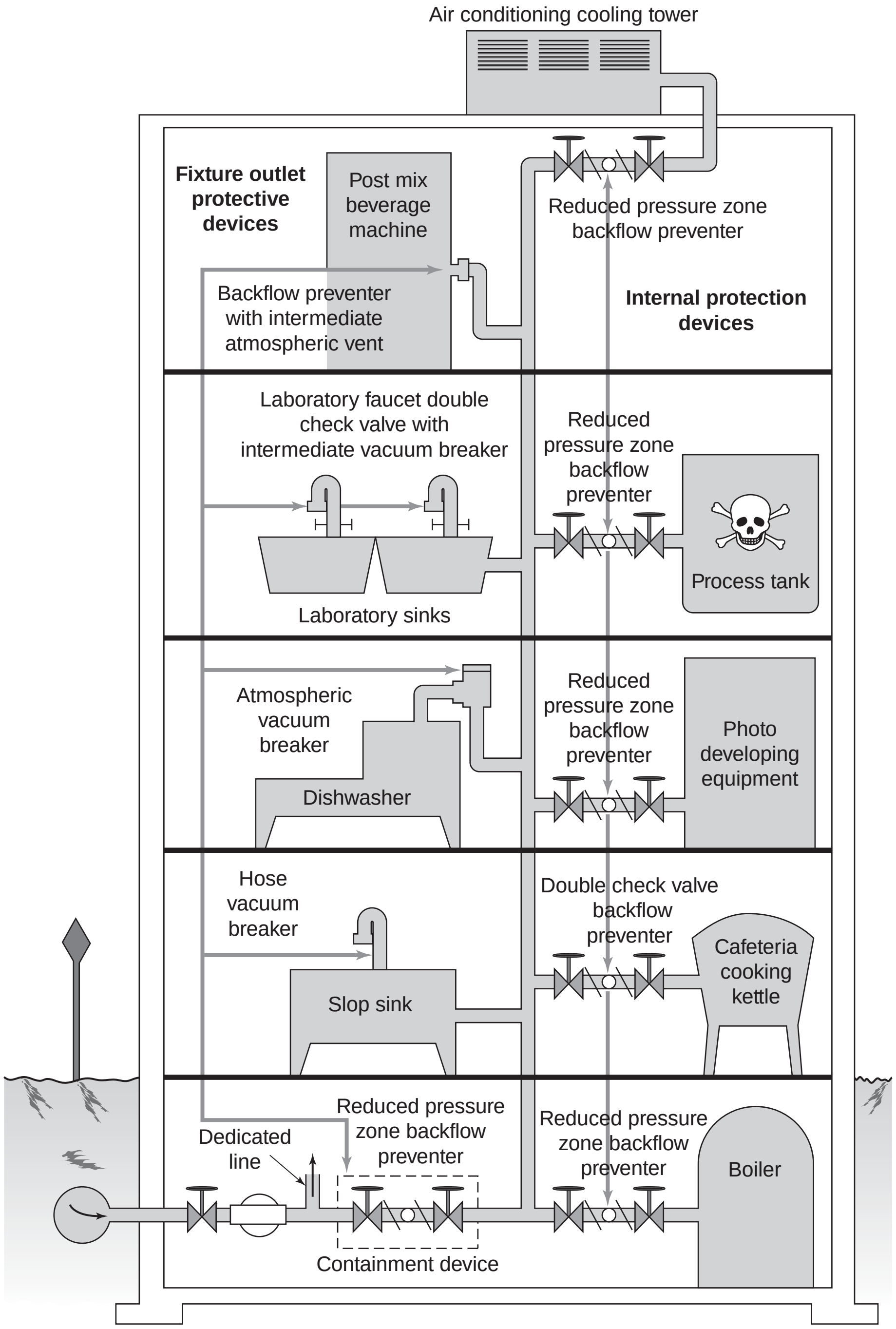

Figure 9.29 Cross-connection preventers.

The water supply company may elect to work initially on the "containment" theory. This approach utilizes a minimum of backflow devices and isolates the customer from the water main. It virtually insulates the customer from potentially contaminating or polluting the public water supply system. Although that type of "containment" does not protect the customer within his own building, it does effectively remove him from possible contamination to the public water supply system.

If the water supplier elects to protect its customers on a domestic internal protective basis and/or "fixture outlet protective basis," then cross-connection control protective devices are placed at internal high hazard locations as well as at all locations where cross-connections exist at the "last free-flowing outlet." This approach entails extensive cross-connective survey work on behalf of the water superintendent as well as constant policing of the plumbing within each commercial, industrial, and residential account.

In large water supply systems, fixture outlet protection cross-connection control philosophy, in and of itself, is a virtual impossibility to achieve and police due to the quantity of systems involved, the complexity of the plumbing systems inherent in many industrial sites, and the fact that many plumbing changes are made within industrial and commercial establishments that do not require the water department to license or otherwise endorse or ratify when contemplated or completed.

In addition, internal plumbing cross-connection control survey work is generally foreign to the average water supplier and is not normally a part of its employees' job descriptions or duties. Although it is admirable for the water supplier to accept and perform survey work, the supplier should be aware that it runs the risk of additional liability in an area that may be in conflict with plumbing inspectors, maintenance personnel, and other public health officials.

Even where extensive "fixture outlet protection" cross-connection control programs are in effect through the efforts of an aggressive and thorough water supply cross-connection control program, the water authorities should also have an active "containment" program in order to address the many plumbing changes that are made and that are inherent within commercial and industrial establishments. In essence, fixture outlet protection becomes an extension of the "containment" program.

Also, in order for the water supplier to provide maximum protection of the water distribution system, consideration should be given to requiring the owner of a premise (commercial, industrial, or residential) to provide, at his/her own expense, adequate proof that his/her internal water system complies with the local or state plumbing code(s). In addition, he/she may be required to install, have tested, and maintain all backflow protection devices that would be required—at his/her own expense.

The water supplier should have the right of entry to determine the degree of hazard and the existence of cross-connections in order to protect the potable water system. By so doing, the water supplier can assess the overall nature of the facility and its potential impact on the water system (determine the degree of hazard), personally see actual cross-connections that could contaminate the water system, and take appropriate action to ensure the elimination of the cross-connection or the installation of required backflow devices.

Assisting the water supplier in the total administration of a cross-connection control program requires that all public health officials, plumbing inspectors, building managers, plumbing installers, and maintenance personnel participate and share in the responsibility of protecting the public health and safety of individuals from cross-connections and contamination or pollution of the public water supply system.

9.6.2 Dedicated Line

In addition to the options just discussed, Fig. 9.29 also depicts the use of a "dedicated" potable water line. This line initiates immediately downstream of the water meter and is "dedicated" solely for human consumption: drinking fountains, safety showers, eye wash stations, and so on. It is very important that this piping be color coded throughout in accordance with local plumbing regulations, flow direction arrows added, and the piping religiously policed to ensure that no cross-connections to other equipment or piping are made that could compromise water quality. In the event that authorities feel the policing of this line cannot be reliably maintained or enforced, the installation of a containment device on this line should be considered.

9.6.3 Method of Action

A complete cross-connection control program requires a carefully planned and executed initial action plan followed by aggressive implementation and constant follow-up. Proper staffing and education of personnel is a requirement to ensure that an effective program is achieved. A recommended plan of action for a cross-connection control program should include the following characteristics:

1. Establish a cross-connection control ordinance at the local level and have it approved by the water commissioners, town manager, and so on, and ensure that it is adopted by the town or private water authority as a legally enforceable document.
2. Conduct public information meetings that define the proposed cross-connection control program, review the local cross-connection control ordinance, and answer all questions that may arise concerning the reason for the program, why and how the survey will be conducted, and the potential impact on industrial, commercial, and residential water customers. Have

state authorities and members of the local press and radio attend the meeting.

3. Place written notices of the pending cross-connection control program in the local newspaper and have the local radio station make announcements about the program as a public service notice.
4. Send employees who will administer the program to a course, or courses, on backflow tester certification, backflow survey courses, backflow device repair courses, and so on.
5. Equip the water authority with a backflow device test kit.
6. Conduct meeting(s) with the local plumbing inspection people, building inspectors, and licensed plumbers in the area who will be active in the inspection, installation, and repair of backflow devices. Inform them of the intent of the program and the part that they can play in the successful implementation of the program.
7. Prior to initiating a survey of the established commercial and industrial installations, prepare a list of these establishments from existing records, then prioritize the degree of hazard that they present to the water system: plating plants, hospitals, car wash facilities, industrial metal finishing and fabrication, mortuaries, and so on. These will be the initial facilities inspected for cross-connections; inspection of less hazardous installations follows.
8. Ensure that any new construction plans are reviewed by the water authority to assess the degree of hazard and ensure that the proper backflow preventer is installed concurrent with the potential degree of hazard that the facility presents.
9. Establish a residential backflow protection program that will automatically ensure that a residential dual check backflow device is installed automatically at every new residence.
10. As water meters are repaired or replaced at residences, ensure that a residential dual check backflow preventer is set with the new or reworked water meter. Be sure to have the owner address thermal expansion provisions.
11. Prepare a listing of all testable backflow devices in the community and ensure that they are tested by certified test personnel at the time intervals consistent with the local cross-connection control ordinance.
12. Prepare and submit testing documentation of backflow devices to the state authority responsible for monitoring these data.
13. Survey all commercial and industrial facilities and require appropriate backflow protection based on the containment philosophy and/or internal protection and fixture outlet protection. Follow up to ensure that the recommended devices are installed and tested on both an initial basis and a periodic basis consistent with the cross-connection control ordinance.

The surveys should be conducted by personnel experienced in commercial and industrial processes. The owners or owners' representatives should be questioned as to what the water is being used for in the facility and what hazards the operations may present to the water system (both within the facility and to the water distribution system) in the event that a backsiphonage or backpressure condition were to exist concurrent with a nonprotected cross-connection. In the event that experienced survey personnel are not available within the water authority to conduct the survey, consideration should be given to having a consulting firm perform the survey on behalf of the water department.

9.7 PRESSURE AND LEAKAGE TESTS OF WATER MAINS

All external contaminations of water mains should be prevented. Contaminants entering the water mains through leaks are other types of cross-connections. Section 6.9.10 discusses the required separation of water mains from sources of contamination. Even with proper separation of water mains from pollution sources, all water mains must still pass the required pressure and leakage tests to ensure that public health is protected. It is required to maintain a pressure of at least 20 psi (140 kPa) at ground level at all points in the water distribution system under all conditions of flow. Unfortunately accidents do happen due to human errors or unforeseen events. For instance, more than one major fire or major pipe leaks or other unanticipated high water demands might draw the hydraulic grade line down to a point that a negative pressure (vacuum) might be created at some service connections. This negative pressure could potentially cause backflow from a contaminated source of water, if the pipe leaks. This section discusses how the pressure and leakage tests can be conducted to prevent any possible backflow from a contaminated source of water.

9.7.1 Preparation for Pressure and Leakage Tests

Pressure and leakage tests are usually conducted after the trench has been partially backfilled for a new pipeline. Same tests can also be conducted frequently after an old pipeline is being suspected of leakage.

For a new pipeline installation, the pipe joints are often left uncovered until after the tests are completed. Each valved section of the water main is filled slowly with water, with air in the pipe being slowly released through corporation stops and hydrants. Corporation stop (or corporation cock) is a water service shutoff valve located at the street water main, and this shutoff valve cannot be operated from the ground surface because it is buried and there is no valve box. New pipes should not be filled with water at a rate greater than 1 foot (0.3048 m) of pipe length per second, because excessive velocities could cause pipe movements and leaks. For an existing water main with water flow, all air must also be removed through corporation stops and hydrants before the pressure and leakage testing starts.

9.7.2 Pressure and Leakage Tests

After the pipe is full of water, it should sit idle for at least 24 hours before the pressure and leakage tests start. Then the pressure is brought up by a booster pump to a required pressure level at least 50% higher than the normal expected operating pressure, or 150 psi (1034 kPa or 10.5 kg/cm^2), whichever is larger, and maintained for at least 4 hours with or without refilling the pipe with water. Any loss of water pressure would indicate leakage. In case the required pressure level cannot be maintained for at least 4 hours, the leakage is serious and the pressure test fails. No further leakage test is needed. If any joints do show serious leakage, they must be checked, adjusted, or repaired.

If the pressure test is successful, meaning that the required pressure level can be maintained for at least 4 hours, the leakage test is simultaneously conducted by measuring the amount of water needed to refill the pipe. If no refill water is needed during the 4-hour (or longer) successful pressure test, there is no pipe leakage. AWWA has specifications for allowable small leakage, which are introduced in below:

1. Asbestos-cement pipe: The longest pipe length commercially available is 13 ft (3.9624 m). At the test pressure of 150 psi (1041 kPa or 10.545 kg/cm^2), the allowable leakage is 30 gpd/mi-in. (27.78 Lpd/km-cm). Gpd/mi-in. stands for "gallons per day per mile of pipe per inch of diameter," while Lpd/km-cm stands for "liters per day per kilometer of pipe per centimeter of diameter."
2. Ductile iron pipe: The longest pipe length commercially available is 18 ft (5.4864 m). At the test pressure of 150 psi (1041 kPa or 10.545 kg/cm^2), the allowable leakage is 23.3 gpd/mi-in. (21.58 Lpd/km-cm) for both mechanical and push-on joints.
3. Plastic pipe: The pipe length is variable; therefore, the allowable leakage is decided differently from other types of pipes. At the test pressure of 150 psi (1041 kPa or 10.545 kg/cm^2), the allowable leakage is 1.45 gal (5.4883 L) per hour for each 100 joints for a 6 inch (152.4 mm) plastic pipe. At the same required test pressure, the allowable leakage is 1.88 gal (7.1158 L) per hour for each 100 joints for an 8 inch (203.2 mm) plastic pipe.

Equation (9.7) can be used for the calculation of the actual leakage of asbestos-cement and ductile iron potable water pipes:

$$L_{AD} = Q_D/Ld \tag{9.7}$$

where L_{AD} = actual leakage of either asbestos-cement or ductile iron pipes, gpd/mi-in. or Lpd/km-cm; Q_D = leak flow rate, gpd or Lpd; L = pipe length, mi or km; and d = pipe diameter, in. or cm.

Equation (9.8) can be used for the determination of the actual leakage of potable water plastic pipes:

$$L_P = Q_H/N \tag{9.8}$$

where L_P = actual leakage of plastic pipes, gph/100 joints or Lph/100 joints; Q_H = leak flow rate, gph or Lph; and N = number of 100 joints.

EXAMPLE 9.1 PRESSURE AND LEAKAGE TESTS PREPARATION

A 520-ft (158.5-m)-long section of a 12-in. (30.48-cm or 0.3048-m)-diameter water main is being filled with water for a pressure–leakage test. If the water main is filled at a flow rate of less than 1 foot of pipe length per second (1 ft/s or 0.3048 m/s), what should be the maximum flow meter reading for filling the water main?

Solution 1 (US Customary System):

Flow rate $Q = Av = [0.785 \times (1\text{ ft})^2] \times (1\text{ ft/s})$
$= 0.785\text{ ft}^3\text{/s} = (0.785\text{ ft}^3\text{/s})\ (7.48\text{ gal/ ft}^3)\ (60\text{ s/min})$
$= 352$ gpm.

For preparation of the pressure–leakage test, do not allow the flow meter's reading to exceed 0.785 ft^3/s or 352 gpm.

Solution 2 (SI System):

Flow rate $Q = Av = [0.785 \times (0.3048\text{ m})^2] \times (0.3048\text{ m/s}) = 0.0222\text{ m}^3\text{/s} = 1{,}334$ L/min.

For preparation of the pressure–leakage test, do not allow the flow meter's reading to exceed 0.0222 m^3/s or 1,334 L/min.

EXAMPLE 9.2 REQUIRED PRESSURE FOR A PRESSURE–LEAKAGE TEST

After the pipe is full of water for a pressure–leakage test, it should sit idle for at least 24 hours before the test start. Then the pressure is brought up by a booster pump to a required pressure level at least 50% higher than the normal expected operating pressure, or 150 psi (1034 kPa or 10.5 kg/cm^2), whichever is larger, and maintained for at least 4 hours. The normal expected operating pressure of the water pipe to be tested is 115 psi (798.1 kPa). At what pressure should the pipe be tested?

Solution 1 (US Customary System):

The test pressure = (normal operating pressure) × 1.5 = 115 psi × 1.5 = 172.5 psi, which is greater than 150 psi. Therefore, the selected test pressure is 172.5 psi.

Solution 2 (SI System):

The test pressure = (normal operating pressure) × 1.5 = 798.1 kPa × 1.5 = 1,197 kPa, which is greater than 1,034 kPa. Therefore, the selected test pressure is 1,197 kPa.

EXAMPLE 9.3 PRESSURE–LEAKAGE TEST

A 24-hour pressure–leakage test is performed on 1,200 ft (365.76 m) of 12-in. (30.48-cm)-diameter ductile iron pipe with a test pressure of 150 psi (1,034 kPa). The ductile iron pipes are 18 ft (5.4864 m) long each. According to AWWA, at the test pressure of 150 psi (1041 kPa or 10.545 kg/cm^2), the allowable leakage is 23.3 gpd/mi-in. (21.58 Lpd/km-cm) of ductile iron pipes for both mechanical and push-on joints. During the 24-hour test period, 22 gallons (83.27 L = 0.08327 m^3) of water were added to maintain the 150 psi (1034 kPa) pressure. Did the pipe pass the pressure–leakage test?

Solution 1 (US Customary System):

$L_{AD} = Q_D/Ld = 22\ \text{gpd}/[1{,}200\ \text{ft}\ (\text{mi}/5{,}280\ \text{ft}) \times 12\ \text{in.}] = 8.07\ \text{gpd/mi-in.}$

Since L_{AD} = actual leakage of ductile iron pipe = 8.07 gpd/mi-in., which was less than the allowable leakage rate of 23.3 gpd/mi-in., the pipe passed the pressure–leak test.

Solution 2 (SI System):

$L_{AD} = Q_D/Ld = 83.27\ \text{Lpd}/[0.36576\ \text{km} \times 30.48\ \text{cm}] = 7.469\ \text{Lpd/km-cm}.$

Since L_{AD} = actual leakage of ductile iron pipe = 7.469 Lpd/km-cm, which was less than the allowable leakage rate of 21.58 Lpd/km-cm, the pipe passed the pressure–leak test.

PROBLEMS/QUESTIONS

9.1 What is the definition of a cross-connection?

9.2 Why do we have cross-connections?

9.3 How many types of cross-connections are there?

9.4 What is the difference between backpressure and back-siphonage?

9.5 What types of devices can be used to correct cross-connections?

9.6 Successful control of cross-connection hazards depends not only on inspection for cross-connections by the water system and by water users, but also on an enforceable cross-connection control program. If a community subscribes to a modern plumbing code, such as the National Plumbing Code, its provisions will govern backflow and cross-connections. However, the water system must obtain authority to conduct a community inspection program through an ordinance or other means and carry out a comprehensive program. Please outline the major components of a water system's cross-connection control program.

9.7 Protection against sanitary deficiencies from cross-connections is an important engineering consideration in water and sewer system design. Visit the website www.10statesstandards.com to learn about the latest government regulations regarding the separation distance of drinking water mains crossing sewers.

9.8 How frequently should backflow preventers be tested to ensure their proper functioning? Who is qualified to perform backflow preventer testing? Consult with your local government for the local rules and regulations.

9.9 In addition to the many cross-connections that may exist on the premises of a water system's customers, the water system itself may own or control cross-connections. These potential cross-connections should be subject to the same scrutiny as those that are privately owned. Examples of cross-connections that can pose a risk to water quality and public health can be found in water treatment plants, pumping stations, or the distribution system. During a sanitary survey for a water treatment system, what are the potential cross-connections that should be identified by an inspector?

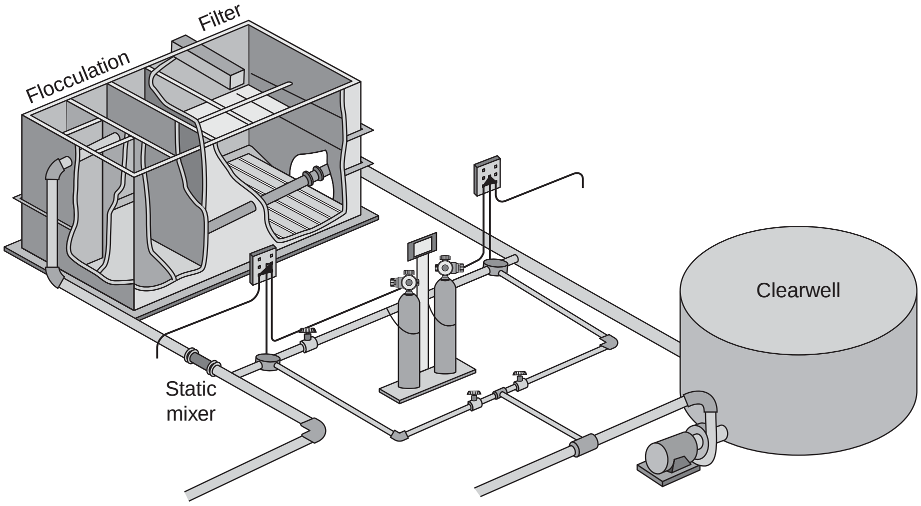

Figure 9.30 For Problem 9.10.

9.10 Figure 9.30 shows a chlorine feed system at a water treatment plant. What is required between the chlorine feeding cylinders and the chlorine feed points? The same figure also shows that one chlorine cylinder feeds chlorine to the influent water before chemical flocculation, and another chlorine cylinder feeds chlorine to the effluent from the filter. What are the technical terms of this chlorination system for cross-connection control? Can one chlorine cylinder be used to feed both the influent to the flocculation influent and the filter effluent? Please explain.

9.11 Figure 9.31 shows that an air relief valve is piped directly to a floor drain. Explain the problem and propose a solution.

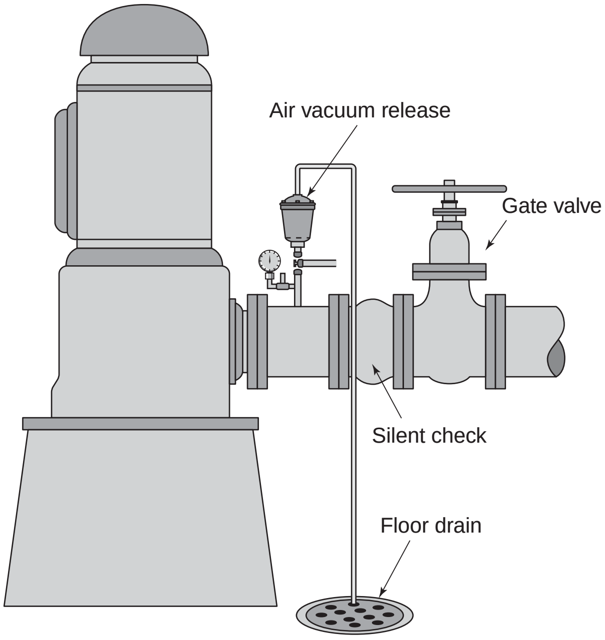

Figure 9.31 For Problem 9.11.

9.12 Figure 9.32 shows a direct cross-connection of a potable water supply line and an acid solution feed line. Propose your solution for the cross-connection control.

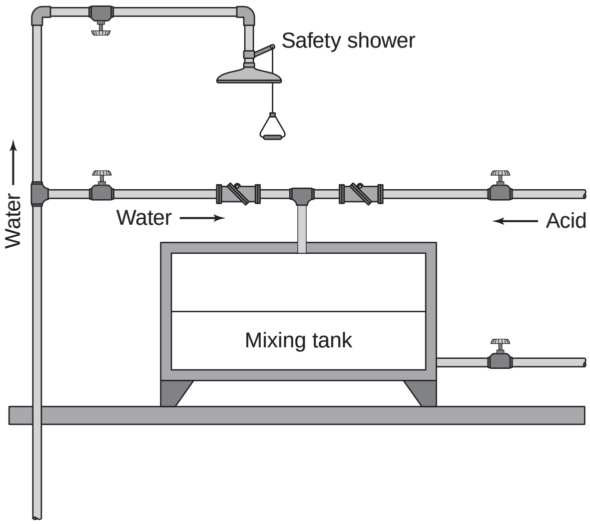

Figure 9.32 For Problem 9.12.

9.13 A 6-hour pressure–leakage test is performed on 500 ft (152.4 m) of 8 in. (203.2 mm) plastic pipe with a test pressure of 150 psi (1,041 kPa). The plastic pipes are 10 ft (3.048 m) long each. According to AWWA, at 150 psi (1,041 kPa), the allowable leakage is 1.88 gallons (7.1158 liters) per hour for each 100 joints for an 8 in. (203.2 mm) plastic pipe. During the 6-hour test period, 12 gallons (45.42 liters) of water were added to maintain the 150 psi (1,041 kPa) pressure. Did the pipe pass the pressure–leakage test?

9.14 The successful promotion of a cross-connection control and backflow prevention program in a municipality will depend on a legal authority to conduct such a program. Where a community has adopted a modern plumbing code, such as the National Plumbing Code, ASA A40.8–1955, or subsequent revisions thereof, provisions of the code will govern backflow and cross-connections. It then remains to provide an ordinance that will establish a program of inspection for the elimination of cross-connections and backflow connections within the community. A cross-connection control ordinance should have at least three basic parts:

1. Authority for establishment of a program

2. Technical provisions relating to eliminating backflow and cross-connections
3. Penalty provisions for violations

Review the cross-connection control ordinances of at least two US cities and suggest a model program for adoption by your municipality.

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Chapter 10

Water Quality Characteristics and Drinking Water Standards

10.1 OBJECTIVES OF WATER-QUALITY MANAGEMENT

Water-quality management is the central theme around which revolve prescriptions for the exploitation, preservation, and reclamation of those properties of water—physical, chemical, and biological—that are responsible for its extraordinary importance in urban and industrial societies. Within the *use cycle* of water in dwellings and industries, water-quality management places upon organized communities the obligation to seek out waters of suitable fitness. Within the *waste cycle* it obliges cities, towns, and industries to send back to the common water resource the spent waters or wastewater effluents of acceptable quality. Assigned to water-quality management thereby is a dual yet essentially unitary responsibility for both water supply and wastewater disposal that can prosper only when quality management establishes and honors reasonable and common objectives and necessary standards.

At one end of the quality spectrum of water lie objectives and standards for safe and palatable drinking waters; at the other end are quality requirements for spent waters or wastewater effluents to be introduced into receiving bodies of water or to be disposed of in other ways. Between the two, there are full water quality criteria for bathing, fishing, shellfish harvesting, and irrigation, and for industrial waters of many kinds. They, too, are of concern to this chapter.

Water-quality management, as part of water-resource management, shares a need for public and technological support that is normally available only in a well-disciplined and industrially mature society. In furtherance of water quality control, moreover, there must be adequate information not only on the nature and capacity of natural sources of water but also on their physical, chemical, and biological quality. Within the wider meaning of *water-quality management*, finally, there must be an understanding by engineers of the common properties of the many kinds of water on the earth: of brooks and rivers, of lakes and oceans, and of waters welling from the ground and falling from the sky (Fig. 10.1). "A river," in the words of Justice Oliver Wendell Holmes—to which can be added, in good conscience, a lake, a spring, or a well—"is more than an amenity, it is a treasure."

10.2 NATURAL AVAILABLE WATER RESOURCES

The source of water determines its inherent quality (Fig. 10.1). Rainwater absorbs the gases and vapors normally present in the atmosphere (oxygen, nitrogen, carbon dioxide, and rare gases) and sweeps particulates out of the air when droplets form about them. Salt nuclei (principally chlorides) reach the atmosphere from ocean spray and freshwater cataracts. Once the rain wets the earth's surface, however, it starts to acquire the properties of surface runoff. In *normal* times the composition of *surface waters* varies with the topography and vegetation of the catchment area and with land use and management. Both mineral and organic particulates are picked up by erosion, together with soil bacteria and other organisms, while salts and soluble substances are taken into solution. Natural and synthetic fertilizers enter the water along with biocide residues, even though the binding power of soils is remarkably strong. In times of *drought* much of the water flowing in surface channels is derived from rising groundwater, urban dry weather runoff, and, in some locations, treated wastewater effluent. In times *of flooding rainstorms* and *snow-melt*, lands not ordinarily eroded by runoff, and flood plains not usually occupied by surface streams, may contribute large amounts of silt to stream flows. Characteristic additions to water in lakes and ponds are algal and other growths that may produce odors and tastes. Swamp waters contain decaying vegetation that intensifies their color, odor, and taste. Cities and industries add wastes of many kinds. The sources of surface water supply may come from precipitation (rain, snow, etc.), river, lake, reservoir, and ocean, as shown in Fig. 10.1. Surface water picks up soil particles and man-made substances, which are detectable as turbidity, color, chemical matters, microorganisms, and radioactive substances. Surface water, except ocean water, is also called *fresh water*

Water Engineering: Hydraulics, Distribution and Treatment, First Edition. Nazih K. Shammas and Lawrence K. Wang.

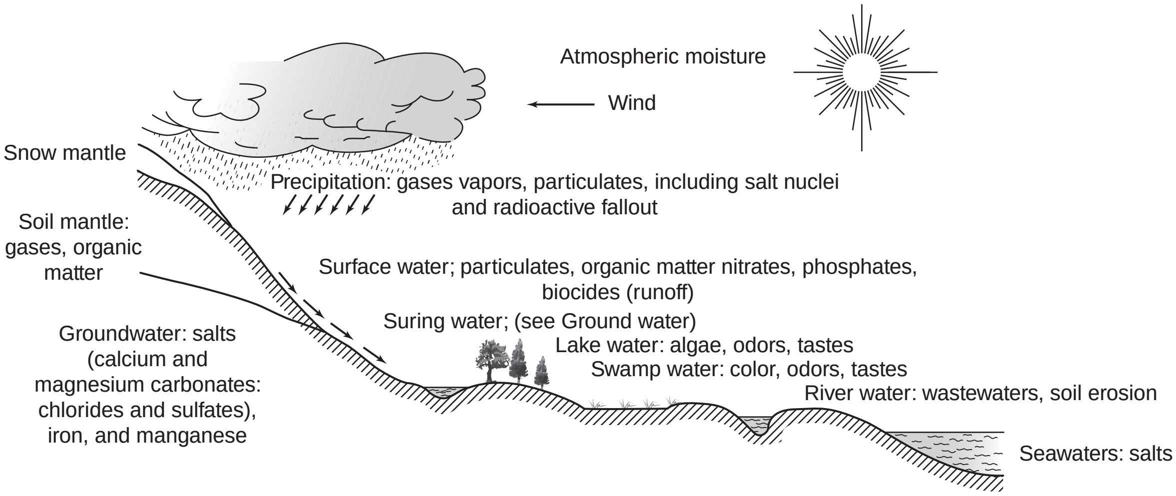

Figure 10.1 Characteristic properties of natural waters in the hydrological cycle (After Fair et al., 1971).

due to its low mineral content. Normally the fresh water of suitable quality from reservoirs and rivers is used for potable water production, under the conditions that the safe yield (either impoundment safe yield or river safe yield, depending on the source) will not be exceeded. Ocean water can be an important water supply source to many coastal regions especially in the Middle East. Classification of brackish, saline, and sea water is based on total dissolved solids (TDS) of 1,000–5,000, 5,000–30,000, and 30,000–35,000 mg/L, respectively.

Groundwaters absorb gases of decomposition and degradable organic matter within the pores of the soil through which they percolate. In the living earth rich in organic matter oxygen is removed from groundwaters and carbon dioxide is added. The pH is lowered and some of the soil minerals are dissolved. Calcium and magnesium carbonates, sulfates, and chlorides enter the water and increase its hardness. Iron and manganese, too, may be rendered soluble. Among gases of decomposition in the pores of rich soils are hydrogen sulfide and methane as well as carbon dioxide. Natural filtration of groundwaters removes organic matter and microbic life. Salts remain in solution. Groundwater moves within the aquifer although the movement is slow. The water table or artesian pressure surface slopes from areas of recharge to areas of discharge. The pressure differences represented by these slopes cause the flow of groundwater within the aquifer.

A *well* can be used to pump water from the groundwater reservoir. If wells extract water from an aquifer over a period of time at a rate such that the aquifer will become depleted or bring about other undesired results, then the *groundwater safe yield* of the aquifer is exceeded. Under these conditions, saltwater encroachment may occur where wells are located near the seashore (Fig. 10.1), or surface water may enter groundwater where wells are located near a river or lake.

A *spring* is an opening in the ground surface from which groundwater flows. Water may flow by force of gravity from water table aquifers or be forced out by artesian pressure, as shown in Fig. 10.1. When the water table or artesian pressure fluctuates, so does the flow of springs. When a source of groundwater is mixed with surface water, this groundwater is called the *groundwater under direct influence of surface water*.

10.3 PUBLIC HEALTH ISSUES AND DRINKING WATER TREATMENT

To slake man's thirst, drinking water must be wholesome and palatable. Accordingly it must not only be free from disease-producing organisms and toxic or otherwise physiologically undesirable substances, but also attractive to the senses. Most dangerous, in the early history of communal water supply in the industrializing countries of the world, were recurring epidemics of enteric fevers traceable to unwholesome drinking water. Most important, since drinking water has been made microbiologically safe, has become the provision of water that is also acceptable and generally useful in household and industry.

However, even today, human and mechanical failures, singly or in combination, occasionally lower the barriers to infection and allow contamination of water supplies that otherwise have had a long history of safety. Because this is so, water safety remains the unquestioned responsibility of water authorities, their engineers, and their general personnel down to the most recently hired workman. An example of inadequate water discipline is the outbreak of typhoid fever at Croydon (London), England in 1937. There a workman who happened to be a typhoid carrier failed to obey sanitary rules at work in a well shaft and contaminated the supply causing 341 cases of typhoid fever with 43 deaths. Another example is the outbreak of typhoid fever at Rochester, NY, in 1940 in which about 30,000 cases of mild enteritis and 5 cases of typhoid fever followed the inadvertent opening of a valve serving an emergency connection between the polluted industrial water supply of the city and its drinking water supply. From 1946 to 1960 there were, in the United States, 39 outbreaks of waterborne typhoid fever, 11 outbreaks with 563 cases of shigellosis, and 4 outbreaks with 24 cases of salmonellosis. In Madera, CA, in 1965, both shigellae and salmonellae were implicated in an outbreak of 2,500 cases of infection traced to the contamination of a well by irrigating an adjacent pasture with unchlorinated wastewater effluent. The attack rate was as high as 53.6% for water drawn from a 370 ft (113 m) well.

At the turn of the last century the annual death rate from typhoid fever still averaged 30 per 100,000 in US communities. By that time the principal epidemic focus of cholera had more or less retreated to Bengal (India and East Pakistan), where cholera could retain a foothold because of favorable climatic and social conditions. After the chlorine compounds (1908 in the United States) and the chlorine itself (1911 in the United States) were dosed to water for disinfection, the incidence of waterborne typhoid fever was driven substantially to the vanishing point at less than 1 per million in organized communities of the United States and Europe. Records of waterborne paratyphoid, namely, typhoid-like fevers or salmonelloses, are few.

In the case of ameba, or amoeba, only three major waterborne epidemics of amebic dysentery are on record in the United States. The first was confined to the patrons of two hotels; the second, to firemen and spectators at a stockyard fire; and the third, to workers in an industrial establishment. In the period from 1946 to 1960 there were 2 outbreaks with 36 cases.

Giardiasis is a diarrheal illness caused by a microscopic parasite, *Giardia intestinalis* (also known as *Giardia lamblia*). Once a person or animal has been infected with *Giardia*, the parasite lives in the intestine and is passed in feces. Because the parasite is protected by an outer shell, it can survive outside the body and in the environment for long periods of time. During the past two decades, *Giardia* infection has become recognized as a common cause of waterborne disease in humans in the United States. *Giardia* can be found worldwide and within every region of the United States.

Cryptosporidium is resistant to chemical disinfectants, remains viable in raw water for extended periods of time, and has been identified as a leading cause of waterborne disease outbreaks in the United States in recent decades. Since *Cryptosporidium* is smaller than *Giardia* and more resistant to chemical disinfectants than *Giardia*, it is assumed that if the required *Cryptosporidium* level of water treatment can be achieved then the required *Giardia* level of water treatment can also be achieved. *Cryptosporidium*, at 3–5 microns, is physically smaller than *Giardia* with a size of approximately 5–15 microns.

Legionella, which are aerobic, non-spore forming, typically flagellated, gram-negative bacteria, were named after a severe epidemic of pneumonia at an American Legion convention in 1976. The incident led to the isolation and characterization of the pathogenic bacteria, *L. pneumophila.*

In the spread of *schistosomiasis*, which results from the improper disposal of human feces, infection of man does not take place in the water itself, but when larvae released by their snail hosts are forced into the skin from shrinking water droplets as bathers or waders emerge from infected waters. Because the United States and Europe are free from the specific snail hosts of pathogenic *schistosomes*, they are also free of *schistosomiasis*. However, the snail hosts and larvae of *schistosomes* that cause *swimmer's itch*, a skin disease (cercarial dermatitis), do occur in some parts of the United States. They are transported from one body of water to another by infected water fowl.

Of the few reasonably well-documented waterborne outbreaks of virus infections, two possibly waterborne outbreaks of *poliomyelitis* in the United States and Canada, and the apparently waterborne outbreak of infectious hepatitis at New Delhi, India, in 1955–1956, are commonly listed in the epidemiological roster.

At the beginning of the industrial revolution, chlorination was the only drinking water treatment technology used for the sole purpose of disinfection. Today disinfection remains to be the most important process among all water treatment processes and has been significantly improved in terms of having more disinfection alternatives and more efficient disinfectant by-product control.

The widespread use of synthetically produced organic and inorganic chemical compounds, including pesticides, insecticides, polychlorinated biphenyls (PCBs), gasoline, pharmaceutical compounds, and industrial compounds containing heavy metals and volatile substances, has required the use of additional water treatment technologies, such as aeration, chemical precipitation, coagulation, sedimentation, flotation, filtration, carbon adsorption, ion exchange, ozonation, and membrane filtration, for drinking water production.

Substances that alter the quality of surface water or groundwater may be classified under four major categories: (1) physical, (2) chemical, (3) biological, and (4) radiological. The four categories and the US and international drinking water standards (World Health Organization (WHO), Canada, and Mexico) are presented in subsequent sections when available. All drinking water standards require not only inactivation of pathogenic microorganisms, but also removal of toxic or undesirable impurities listed in the drinking water standards. Section 10.8 discusses the drinking water standards in detail. It should be noted that the drinking water standards are similar to any other laws, which are subject to change from time to time. The reader should always search for and use the latest local drinking water standards for water quality control.

The following definitions apply to both primary and secondary US standards:

1. *Maximum contaminant level* (MCL): The highest level of a contaminant that is allowed in drinking water. MCLs are set as close to MCLGs (maximum contaminant level goals) as feasible using the best available treatment technology and taking cost into consideration. MCLs are enforceable standards.
2. *Maximum contaminant level goal* (MCLG): The level of a contaminant in drinking water below which there is no known or expected risk to health. MCLGs allow for a margin of safety and are nonenforceable public health goals.
3. *Maximum residual disinfectant level* (MRDL): The highest level of a disinfectant allowed in drinking water. There is convincing evidence that addition of a disinfectant is necessary for control of microbial contaminants.
4. *Maximum residual disinfectant level goal* (MRDLG): The level of a drinking water disinfectant below which there is no known or expected risk to health. MRDLGs do not reflect the benefits of the use of disinfectants to control microbial contaminants.
5. *Treatment technique* (TT): A required process intended to reduce the level of a contaminant in drinking water. US Environmental Protection Agency's (US EPA) surface water treatment rules require systems using surface water or groundwater under the influence of surface water to (1) disinfect their water and (2) filter their water or meet criteria for avoiding filtration so that the contaminants are controlled at the allowable levels.

The following are similar terms used by different countries: (1) maximum contaminant level (MCL) and maximum residual disinfectant level (MRDL)—United States; (2) *maximum acceptable concentration* (MAC)—Canada; (3) *maximum allowable limit* (MAL)—Mexico; and (4) *guideline value* (GV)—WHO. For simple comparison of the US drinking water standards with international drinking water standards (WHO, Canada, and Mexico), the "maximum allowable concentration" represents MCL, MAC, MAL, and GV in this chapter; while "*recommended concentration*" and "*recommended limit*" represent the drinking water standards, which are not legally enforceable.

The US Primary Drinking Water Standards and the US Secondary Drinking Water Standards are summarized in Section 10.8. The Mexico Drinking Water Standards are discussed in Problem 10.14. The drinking water standards listed and discussed in this book are for reference only. An engineer in any country, any state/province must know how to search for the most current applicable drinking water standards from the literature for his/her specific water engineering project.

10.4 PHYSICAL CHARACTERISTICS AND CONSTITUENTS

To be palatable, water must be significantly free from color, turbidity, taste, and odor, of moderate temperature in summer and winter, and well aerated. At least four human perceptions respond to these qualities: the senses of sight (color and turbidity), taste, smell (odor), and touch (temperature). If the pleasant sound of running water is considered one of its qualities, the sensory appeal of water becomes complete.

10.4.1 Color

Dissolved organic material from decaying vegetation and certain inorganic matters cause color in water. Specifically color is of vegetable provenance, like the *meadow-tea* in the running brook of New England. However, water may also become discolored by industrial wastes, natural iron and manganese, and the products of corrosion. Occasionally excessive algae blooms or the growth of aquatic microorganisms may also impart color. Color itself is not usually objectionable from the standpoint of health. Its presence is esthetically objectionable. To appeal to visitor as well as native, the color of given water should be low. Yet the accustomed color of a water supply may have been quite high in the past, without eliciting comment. The recommended limit for color is 15 color units in the United States and Canada, and the MAL for color is 20 color units in Mexico.

10.4.2 Turbidity and Particle Count

The presence of suspended matter such as clay, silt, finely divided organic materials, plankton, and other inorganic and organic materials in water is known as turbidity. Following a rainfall, variations in the raw water turbidity may be considered an indication of surface or other introduced pollution. Turbidity may also come from eroding clay banks, or from industrial wastes, products of corrosion, and growths of other microorganisms. Nephelometers are now the standard instruments for measurement of low turbidities in water and give results in terms of the *nephelometric turbidity unit* (NTU). Turbidities in excess of 5 NTU are easily detectable in a glass of water and are usually objectionable for esthetic reasons.

Turbidity is the traditional parameter for water quality control in water treatment. Turbidity is used to indicate water quality and filtration effectiveness (e.g., whether disease-causing organisms are present). Higher turbidity levels are often associated with higher levels of disease-causing microorganisms such as viruses, parasites, and some bacteria. These organisms can cause symptoms such as nausea, cramps, diarrhea, and associated headaches. Particle counts which are measured by particle counters are gaining popularity to evaluate water treatment efficiencies and finished water quality. A particle count of 50–100 particles/mL (in the size range of 2.5–150 μm) represents a finished water quality having turbidity of 0.2–0.3 NTU. Both the US and Canada turbidity standards are complex. The readers are referred to Section 10.8 for details. Briefly speaking, the US Primary Drinking Water Standards require that (a) at no time can water treatment plant's effluent turbidity go above 1 NTU and (b) the filtration plants must keep the effluent turbidity at 1 NTU or below, and must not exceed 0.3 NTU in 95% of daily samples in any month. The Mexico MAL for turbidity is 5 NTU in drinking water.

10.4.3 Taste and Odor

The words taste and odor are often used loosely and interchangeably. Actually there are about five tastes: sour, sweet, bitter, hot and salt— strictly confined in their perception to the taste buds of the tongue. Hot is not associated with drinking water. Odors appear to be without limit in number and are known to change in quality as the concentration of the odorous compounds, or the intensity of their smell, is varied. However, careful screening of odors suggests that there may be certain fundamental odors from which all odors could be compounded. The smallest number in any classification is four: sweet or fragrant, sour or acid, burnt or empyreumatic, and goaty or caprylic.

Taste and odor in water can be caused by the growth of algae or by the foreign matter such as organic compounds, inorganic salts, or dissolved gases. These taste- and odor-causing substances may come from domestic, agricultural, or natural sources. Generally speaking, tastes and odors should not be sufficiently intense to impress themselves on the user without his knowingly searching for them. Acceptable waters should be free from any objectionable taste or odor at point of water use. Legally the taste of drinking water is not regulated in the United States. The US Secondary Drinking Water Standards, however, recommend that the odor of drinking water be 3 *threshold odor number* (TON) or below. Mexican and Canadian governments require that the taste and odor of drinking water be inoffensive.

10.4.4 Temperature

The most desirable drinking waters are consistently cool and do not have temperature fluctuations of more than a few degrees. Groundwater and surface water from mountainous areas generally meet these criteria. The most palatable water temperature is between 50°F and 60°F (10°C–15°C).

10.4.5 Foamability

Foam in water is usually caused by concentrations of detergents greater than 1 mg/L as MBAS (methylene blue active substance). While foam itself is not hazardous, the user should understand that if enough detergent is reaching a water supply to cause a noticeable froth to appear on a glass of water, other possibly hazardous substances of waste origin are also likely to be present. The US Secondary Drinking Water Standards recommend that the foaming agents be limited to 0.5 mg/L as MBAS in drinking water. The foaming agents are not regulated by Mexican and Canadian governments.

10.5 CHEMICAL CHARACTERISTICS AND CONSTITUENTS

A variety of toxic substances may conceivably find their way into water supplies, among them (a) toxic substances leached from mineral formations such as fluorapatites; (b) phytotoxins manufactured by specific algae; (c) heavy metals dissolved from waterworks structures, principally metallic pipes and improperly manufactured plastic pipes, or added as water treatment chemicals (certain polyelectrolytes, for instance); (d) toxic compounds contained in industrial and household wastes emptied into water courses or discharged into the ground; (e) radioactive substances in fallout and from the nuclear energy industry; and (f) pesticides reaching water courses from chemical dusts or sprays applied to crops and to land and water surfaces for the control of agricultural blights, nuisance insects, vectors of human and animal diseases, trash fish, water weeds, and the like.

Whereas the hazards of infective and toxic contaminants are self-evident, certain qualities of water supplies touching only on the fringe of wholesomeness are not well understood. Excessive mineralization is one, insufficient mineralization is a second, and industrial organic chemicals are a third.

Some highly mineralized waters possess laxative properties; waters containing magnesium and sulfate ions are examples. Other highly mineralized or hard waters consume much soap before they form lather. The resulting causticity irritates the skin of sensitive persons, and *winter chapping* and *dishpan hands* may become chronic complaints.

Iodide and fluoride offer striking examples of insufficient mineralization. Glaciated or otherwise heavily leached soils may not contain enough iodine to satisfy physiological requirements. Goiter is then endemic. However, the remedy for iodine deficiency does not seem to be the introduction of iodides into drinking water, even though this was done at one time with apparent success. Distribution of iodide tablets to school children and, later, the iodization of table salt have proved to be fully as effective and more economical. By contrast, fluoridation of water for the control of dental caries will continue to be justified until a more economical and manageable method of furnishing physiologically needed amounts is brought forward. Water of high sodium content, including some softened waters, may be troublesome to persons on low-salt diets. Why soft waters appear to be correlated with more cardiovascular disease than moderately hard and fluoridated waters needs further study. The mottling of tooth enamel observed, for instance, in the mid- and southwest United States, is caused by excessive concentrations of fluorides in water. By contrast, small amounts of fluoride are ingested safely and lower the prevalence of dental caries.

Literally hundreds of new chemical compounds are being introduced into our environment daily. Few of these are assessed for their potential impact on the health of man, particularly on the *synergistic* effect they may have when acting together or in concert with other kinds of environmental insults. The impact of the long-term ingestion of low levels of pollutants in water is very difficult to ascertain. Studies made in Holland showed that cancer death rates in municipalities with a water system tended to be lower than in those without a water system and that municipalities receiving their drinking water from polluted rivers had a higher cancer death rate than those taking their water from purer underground sources. The following sections discuss specific chemical constituents, their characteristics, and their drinking water standards (WHO, United States, Canada and Mexico) when available, for the purpose of comparison and general reference.

10.5.1 Metals

10.5.1.1 Arsenic Arsenic (As) occurs naturally and is also used in insecticides. It is found in tobacco, shellfish, drinking water, and the air in some locations. The maximum allowable concentrations of arsenic in drinking water by the standards are 0.01 mg/L (WHO, United States, and Canada) and 0.05 mg/L (Mexico). If persons drink water that continuously exceeds the standard by a substantial amount over a lifetime, they may suffer from fatigue and loss of energy. Extremely high levels can cause toxicity.

10.5.1.2 Barium Although not as widespread as arsenic, barium also occurs naturally in the environment in some areas. It can also enter water supplies through hazardous industrial waste discharges or releases. Small doses of barium are not harmful. However, it is quite dangerous when consumed in large quantities. The maximum allowable concentrations of barium in drinking water by the standards are 2 mg/L (United States), 1 mg/L (Canada), and 0.7 mg/L (WHO and Mexico).

10.5.1.3 Cadmium Only minute amounts of cadmium are found in natural waters in the United States. Hazardous waste discharges from the electroplating, photography, insecticide, and metallurgy industries can increase cadmium levels. Another common source of cadmium in drinking water is from galvanized pipes and fixtures if the pH of a water supply is not properly controlled. The sources of cadmium exposure are the foods we eat and cigarette smoking. The maximum allowable concentrations of cadmium in drinking water by the standards are 0.005 mg/L (United States, Canada, and Mexico) and 0.003 mg/L (WHO).

10.5.1.4 Chromium Chromium is commonly released to the environment from the electroplating industry and is extremely hazardous. Some studies suggest that in minute amounts, chromium may he essential to human beings. The maximum allowable concentrations of total chromium in drinking water by the standards are 0.05 mg/L (WHO, Mexico, and Canada) and 0.1 mg/L (United States).

10.5.1.5 Lead Lead sources include lead and galvanized pipes, auto exhausts, and hazardous waste releases. The maximum allowable concentrations of lead in drinking water by the standards are 0.01 mg/L (WHO and Canada), 0.015 mg/L (United States) and 0.025 mg/L (Mexico) of water. Excessive amounts well above this standard may result in nervous system disorders or brain or kidney damage. In the United States, lead and copper together are also regulated by a TT. The reader is referred to Section 10.5.1.9 for the details.

10.5.1.6 Mercury Large increases in mercury levels in water can be caused by industrial and agricultural use and waste releases. The health risk from mercury is greater from mercury in fish than simply from waterborne mercury. Mercury toxicity may be acute, in large doses, or chronic, from lower doses taken over an extended time period. The maximum allowable concentrations of mercury in drinking water by the standards are 0.002 mg/L (United States), 0.001 mg/L (Mexico), and 0.006 mg/L (WHO). The US level (0.002 mg/L) is 13% of the total allowable daily dietary intake of mercury.

10.5.1.7 Selenium Selenium is found in meat and other foods due to water pollution. Although it is believed to be essential in the diet, there are indications that excessive amounts of selenium may be toxic. Studies are underway to determine the amount required for good nutrition and the amount that may be harmful. The maximum allowable concentrations of selenium in drinking water by the standards are 0.01 mg/L (WHO and Canada) and 0.05 mg/L (United States). If selenium came only from drinking water, it would take an amount many times greater than the standard to produce any ill effects.

10.5.1.8 Silver Silver is sometimes released to the environment by the photographic industry and is considered to be toxic at high concentration. Because of the evidence that silver, once absorbed, is held indefinitely in tissues, particularly the skin, without evident loss through usual channels of elimination or reduction by transmigration to other body sites, and because of other factors, the recommended concentration of silver in drinking water by the US Secondary Drinking Water Standards is 0.1 mg/L.

10.5.1.9 Copper Copper is found in some natural waters, particularly in areas where these ore deposits have been mined. Excessive amounts of copper ions can occur in corrosive water that passes through copper pipes. Copper in small amounts is not considered detrimental to health, but will impart an undesirable taste to the drinking water. For this reason, the limits for copper by the standards are 1.0 mg/L (recommended; United States and Canada) and 2.0 mg/L (MAL; Mexico). In the United States, lead and copper are also regulated by a TT that requires water systems to control the corrosiveness of their water. If more than 10% of tap water samples exceed the action level, water systems must take additional steps. For copper, the action level is 1.3 mg/L and for lead is 0.015 mg/L.

10.5.1.10 Iron Small amounts of iron are frequently present in water because of the large amount of iron present in the soil and because corrosive water will pick up iron from pipes. The presence of iron in water is considered objectionable because it imparts a brownish color to laundered goods and affects the taste of beverages such as tea and coffee. Recent studies indicate that eggs spoil faster when washed in water containing iron in excess of 10 mg/L. The US and Canada recommended limits for iron are both 0.3 mg/L, and the Mexican MAL is also 0.3 mg/L.

10.5.1.11 Manganese Manganese is normally associated with iron. There are two reasons for limiting the concentration of manganese in drinking water: (a) to prevent esthetic and economic damage and (b) to avoid any possible physiological effects from excessive intake. The domestic user finds that manganese produces a brownish color in laundered goods and impairs the taste of beverages, including coffee and tea. Serious surface and groundwater pollution problems have developed from existing and abandoned mining operations. Among the worst are those associated with coalmine operations, where heavy concentrations of iron, manganese, sulfates, and acids have resulted from the weathering and leaching of minerals (pyrites). The US and Canada recommended limits for manganese are both 0.05 mg/L, while the Mexican MAL is 0.15 mg/L.

10.5.1.12 Lead Lead is a toxic heavy metal. A brief or prolonged exposure of the body to lead ions can be seriously injurious to health. Prolonged exposure to relatively small quantities of lead ions may result in serious illness or death. Lead taken into the body in quantities in excess of certain relatively low "normal" limits is a cumulative toxicity. Excessive lead may occur in the source water but the usual cause of excessive lead is corrosive water in contact with lead-painted roofs or the use of lead pipes. These conditions must be corrected to provide a safe water supply. The maximum allowable concentrations of lead in drinking water by the standards are 0.01 mg/L (WHO and Canada), 0.025 mg/L (Mexico), and 0.015 mg/L (action level; United States).

10.5.1.13 Zinc Zinc is found in some natural waters, particularly in areas where these ore deposits have been mined. Zinc is not considered detrimental to health, but it will impart an undesirable taste to drinking water. The US and Canada recommended limits for zinc are both 5.0 mg/L, while the Mexican MAL is also 5.0 mg/L.

10.5.1.14 Sodium Sodium present in a water supply can be detected by a laboratory analysis. When home water softeners utilizing the ion-exchange method are used, the amount of sodium should be increased. For this reason, water that has been softened should be analyzed for sodium when a precise record of individual sodium intake is recommended. For healthy persons, the sodium content of water is unimportant because the intake from salt is so much greater, but for persons placed on a low-sodium diet because of heart, kidney, or circulatory ailments or complications of pregnancy, sodium in water must be considered. The usual low-sodium diets allow for 20 mg/L sodium in the drinking water. When this limit is exceeded, such persons should seek a physician's advice on diet and sodium intake. The Mexican MAL for sodium and the Canada recommended limit for sodium are both 200 mg/L.

10.5.2 Anions

10.5.2.1 Chlorides Most waters contain some chloride ions. The amount of chloride present can be caused by the leaching of marine sedimentary deposits and by pollution from sea water, brine, or wastewaters. Chloride concentrations in excess of about 250 mg/L usually produce a noticeable taste in drinking water. In areas where the chloride content is higher than 250 mg/L and all other criteria are met, it may be necessary to use a water source that exceeds this limit. An increase in chloride content in water may indicate possible pollution from waste sources, particularly if the normal chloride content is known to be low. Accordingly 250 mg/L is the MAL in Mexico, and also the recommended limit in the United States and Canada.

10.5.2.2 Fluorides High levels of fluoride in drinking water can cause brown spots on the teeth, or mottling, in children up to 12 years of age. Adults can tolerate 10 times more than children. In the proper amounts, however, fluoride in drinking water prevents cavities during formative years. This is why many communities add fluoride in controlled amounts to their water supply. The maximum amount of fluoride allowed in drinking water by the standard ranges from 0.4 to 2.0 mg/L depending on one average maximum daily air temperature. The hotter the climate, the lower the amount allowed, for people tend to drink more in hot climates. In hot areas, the maximum allowable level for fluoride is 2.0 mg/L of water. Optimum concentrations from 0.7 to 1.2 mg/L fluoride ions are recommended. Legally the maximum allowable concentration of fluoride in drinking water by the standards is 1.5 mg/L in Canada and Mexico, but the recommended limit for fluoride is 2.0 mg/L in the United States.

10.5.2.3 Nitrates and Nitrites Nitrate (NO_3^-) in drinking water above the standard poses an immediate threat to children less than 3 months of age. In some infants, excessive levels of nitrate have been known to react with the hemoglobin in the blood to produce an anemic condition commonly known as "blue baby." If the drinking water contains an excessive amount of nitrate, it should not be given to infants under 3 months of age and should not to be used to prepare formula. Nitrate can be removed from water by ion exchange, RO, or distillation. Nitrates in excess of normal concentrations, often in shallow wells, may be an indication of seepage from livestock manure deposits. In some polluted wells, nitrite (NO_2^-) will also be present in concentrations greater than 1 mg/L and is even more hazardous to infants. When the presence of high nitrite concentration is suspected the water should not be used for infant feeding. The nitrate concentration should be determined, and if excessive, advice should be obtained from health authorities about the suitability of using the water for drinking by anyone. The maximum allowable concentrations of nitrate in drinking water by the standards are 10 mg/L as N (United States and Mexico), 45 mg/L as NO_3^- (Canada), and 50 mg/L as NO_3^- (WHO). The maximum allowable concentrations of nitrite are 1.0 mg/L as N (United States), 0.05 mg/L as N (Mexico), and 3 mg/L as NO_2^- (WHO).

10.5.2.4 Sulfates Waters containing high concentrations of sulfate caused by the leaching of natural deposits of magnesium sulfate (Epsom salts) or sodium sulfate (Glauber's salt) may be undesirable because of their laxative effects. The limits for sulfate content in drinking water by standards are 250 mg/L (US Secondary Drinking Water Standards), 400 mg/L (Mexico MAL), and 500 mg/L (Canada GV).

10.5.2.5 Phosphates Phosphates and nitrogen are two major nutrients causing eutrophication in natural waters. Serious water quality problems may result from uncontrolled algae and aquatic plant blooms in lakes and reservoirs when phosphorus concentration exceeds 0.05 mg/L, if phosphorus is the limiting nutrient. Suggested adult daily intake for phosphorus is 800 mg/day. A deficiency in phosphorus may result in weakness, bone pain, and rickets.

10.5.3 Alkalinity and pH

Alkalinity is a normal characteristic of natural waters. It provides buffering capacity to water for maintaining an optimum pH and is also needed in chemical precipitation and coagulation process for water treatment. Alkalinity is imparted to water by bicarbonate, carbonate, or hydroxide components. The presence of these compounds is determined by standard methods involving titration with various indicator solutions. Knowledge of the alkalinity components is useful in the treatment of water supplies. pH is a measure of the hydrogen ion concentration in water. It is also a measure of the acid or alkaline content. pH values range from 0 to 14, where 7 indicates neutral water; values less than 7, increasing acidity; and values greater than 7, increasing alkalinity. The pH of water in its natural state often varies from 5.5 to 9.0. Determination of the pH value assists in the control of corrosion, the determination of proper chemical dosages, and adequate control of disinfection. pH of the drinking water should be maintained within the range from 6.5 to 8.5. For healthy fresh water aquatic habitat, the desirable bicarbonate alkalinity ranges from 30 to 135 mg/L as $CaCO_3$. Legally the recommended pH range is 6.5–8.5 in the United States and Canada, and the required pH range is 6.5–8.5 in Mexico.

10.5.4 Hardness, Calcium and Magnesium, Carbonate and Bicarbonate

Hard water and soft water are relative terms. Hard water retards the cleaning action of soaps and detergents, causing an expense in the form of extra work and cleaning agents. Furthermore, when hard water is heated it will deposit a hard

EXAMPLE 10.1 NITRATE CONCENTRATION CONVERSION

The maximum allowable concentrations of nitrate in drinking water by the standards are 10 mg/L as N (United States and Mexico), 45 mg/L as NO_3^- (Canada), and 50 mg/L as NO_3^- (WHO). What are the differences among the four drinking water standards on nitrate?

Solution:

To be able to compare the various standards, it is necessary to convert them to the same base. Here the standard are converted as N:

Atomic weight of N = 14.
Atomic weight of O = 16.
Ratio of $N/NO_3^- = 14/[14 + 3 \times 16] = 14/62$.
US standard for nitrate = **10 mg/L as N**.
Mexico standard for nitrate = **10 mg/L as N**.
Canada standard for nitrate = $45 \times (14/62)$ = **10.16 mg/L as N**.
WHO standard for nitrate = $50 \times (14/62)$ = **11.29 mg/L as N**.

scale (as in a kettle, heating coils, or cooking utensils) with a consequent waste of fuel. Calcium and magnesium salts, which cause hardness in water supplies, are divided into two general classifications: (a) carbonate or temporary hardness and (b) noncarbonate or permanent hardness. Carbonate or temporary hardness is so called because heating the water will largely remove it. When the water is heated, bicarbonates break down into insoluble carbonates that precipitate as solid particles which adhere to a heated surface and the inside of pipes. Noncarbonate or permanent hardness is so called because it is not removed when water is heated. Noncarbonate hardness is due largely to the presence of the sulfates and chlorides of calcium and magnesium in the water. The classifications of water based on hardness are (a) soft water, 0–40 mg/L of total hardness as $CaCO_3$; (b) moderately hard water, 41–100 mg/L of total hardness as $CaCO_3$; (c) hard water, 101–300 mg/L of total hardness as $CaCO_3$; (d) very hard water, 301–500 mg/L of total hardness as $CaCO_3$; and (e) extremely hard water, greater than 500 mg/L of total hardness as $CaCO_3$. The total hardness in drinking water is unregulated in the United States. The Mexico MAL for total hardness and Canada recommended concentration for total hardness in drinking water are both 500 mg/L as $CaCO_3$.

10.5.5 Total Dissolved Solids and Conductivity

TDS in natural water are due to soluble inorganic mineral compounds. The TDS concentration in normal surface waters is generally less than 200 mg/L. The recommended upper limit of TDS in irrigation water is 1,500 mg/L. The specific conductance or the electrical conductivity (EC) of water is related to TDS linearly, or $TDS = k\,(EC)$, where TDS is in mg/L, EC is in μmho/cm, and k is a constant varying from 0.5 to 0.9 (average 0.64). A maximum of 500 mg/L TDS in drinking water supplies is recommended by the US EPA, although TDS is not a measure of the safety or harmfulness of water. The Canada GV for TDS is 500 mg/L and the Mexico MAL for TDS is 1,000 mg/L.

10.5.6 Dissolved Oxygen

The level of dissolved oxygen (DO) in natural surface water is an indication of the extent of pollution by oxygen-demanding substances. Low DO concentrations are associated with low-quality water; aquatic animal may die if DO concentration is below certain tolerable level, and water treatment plants may face taste and odor problems. In treated water, the DO is one of the most important factors influencing the corrosion rate of metal treatment facilities and pipelines.

10.5.7 Pesticides

Millions of pounds (1 lb = 0.454 kg) of pesticides are used on croplands, forests, lawns, and gardens in the United States each year. A large quantity of hazardous pesticides is also released by the pesticide industry to the environment. Besides, careless use of pesticides can also contaminate water sources and make the water unsuitable for drinking. Numerous cases have been reported where individual wells have been contaminated when the house was treated for termite control or the lawn was treated for weed control. The use of pesticides near wells is not recommended. Otherwise, these hazardous pesticides drain off into surface waters or seep into underground water supplies. Many pesticides pose health problems if they get into drinking water and the waters not properly treated. The maximum allowable concentrations for pesticides in drinking water are

1. Endrin, 0.002 mg/L (United States) and 0.0006 mg/L (WHO);
2. Lindane, 0.0002 mg/L (United States) and 0.002 mg/L (WHO);
3. Methoxychlor, 0.04 mg/L (United States) and 0.02 mg/L (WHO);
4. Toxaphene, 0.003 mg/L (United States);
5. 2,4-D, 0.07 mg/L (United States); 0.03 mg/L (WHO); 0.1 mg/L (Canada); 0.05 mg/L (Mexico);
6. 2,4,5-TP silvex, 0.05 mg/L (United States); 0.009 mg/L (WHO);
7. Aldrin and dieldrin pesticides, 0.00003 mg/L (WHO, Mexico); 0.0007 mg/L (Canada); unregulated (United States)

10.5.8 PCBs, CFCs, and Dioxin

PCB, chlorofluorocarbon (CFC), petroleum products, and dioxin are major toxic contaminants in water, soil, and air. PCBs are a family of compounds that were used extensively in electrical equipment, such as transformers, because of their insulating and heat transferring qualities. They are suspected human carcinogens and have been linked to liver, kidney, and other health problems. Freon is a commercial trademark for a series of fluorocarbon products used in refrigeration and air-conditioning equipment, as aerosol propellants, blowing agents, fire extinguishing agents, and cleaning fluids and solvents. Many types contain chlorine as well as fluorine and should be called chlorofluorocarbons (CFCs). Dioxins form a family of aromatic compounds known chemically as di-benzo-*p*-dioxins. Each of these compounds has a nucleus triple ring structure consisting of two benzene rings interconnected to each other through a pair of oxygen atoms. Most interest has been directed toward the isomer 2,3,7,8-TCDD, which is the most toxic dioxin. Humans exhibit symptoms effecting on enzyme and nervous systems and muscle and joint pains after exposure to dioxin. When a water supply source is known to be contaminated by PCB, CFC, or dioxins, the raw water must be properly treated by the advanced water treatment technology, such as carbon adsorption and membrane filtration. For instance, the water of Hudson River, New York, is a water supply source to many communities along the river, but has been contaminated by PCBs from General Electric Company. The MCL of PCB and dioxin (2,3,7,8-TCDD) in drinking water by the US standards are 0.0005 mg/L for PCB and 0.00000003 mg/L for dioxin.

10.5.9 Asbestos

Asbestos is an airborne contaminant, and also a waterborne contaminant, regulated by many state governments in the United States. The health effect of asbestos in water, however, is not totally known. According to the US Primary Drinking Water Standards, the MCL of asbestos in drinking water is 7 million fibers per liter (MF/L).

10.5.10 Residual Disinfectants

Chlorine, chlorine dioxide, chloramines, and ozone are common chemical disinfectants. UV is a physical disinfectant. Only the residual of chlorine, chlorine dioxide, and chloramines are regulated by the US EPA. The residual chlorine or chloramines can cause eye and nose irritation and stomach discomfort if either concentration exceeds 4 mg/L (as Cl_2). If the residual chlorine dioxide concentration in drinking water is over 0.8 mg/L (as ClO_2), it will have an adverse effect on infants and young children, causing anemia or nervous system illness. In the United States, *the* MRDLs for chloramines, chlorine, and chlorine dioxide are 4 mg/L (as Cl_2), 4 mg/L (as Cl_2), and 0.8 mg/L (as ClO_2), respectively.

WHO also has established the GVs for residual disinfectants: (1) chlorine, 5 mg/L; (2) monochloramine, 3 mg/L; (3) dichloroisocyanurate, 40 mg/L; and (4) sodium, 50 mg/L. Sodium plays an important role when sodium hypochlorite or sodium dichloroisocyanurate is dosed into water as a disinfectant.

Ammonia, which is used in chloramination process, is regulated by Mexican government. The Mexico MAL for ammonia (as NH_3) is 0.5 mg/L.

10.5.11 Disinfectant By-products

Disinfection by-products (DBP) are the by-products of drinking water disinfection. Different disinfectants will product different DBP. They are all cancer-causing substances. The maximum allowable concentrations (MCL of United States, MAC of Canada, MAL of Mexico, or GV of WHO) of four common DBP are (a) bromate = 0.01 mg/L (United States, Canada, WHO); (b) chlorite = 1.0 mg/L (United States, Canada); 0.7 mg/L (WHO); (c) haloacetic acids (HAAs) = 0.06 mg/L (United States); 0.08 mg/L (Canada); and (d) total trihalomethanes (TTHMs) = 0.08 mg/L (United States); 0.1 mg/L (Canada); 0.2 mg/L (Mexico). TTHM includes bromodichloromethane, bromoform, dibromochloromethane, and chloroform. HAA5 includes five compounds—dichloroacetic acid, trichloroacetic acid, monochloroacetic acid, bromoacetic acid, and dibromoacetic acid.

The GVs established by WHO for DBP are bromate, 0.01 mg/L; bromodichloromethane, 0.06 mg/L; bromoform, 0.1 mg/L; chlorate, 0.7 mg/L; chlorite, 0.7 mg/L; chloroform, 0.3 mg/L; dibromoacetonitrile, 0.07 mg/L; dibromochloromethane, 0.1 mg/L; dichloroacetate, 0.05 mg/L; dichloroacetonitrile, 0.02 mg/L; monochloroacetate, 0.02 mg/L; *n*-nitrosodimethylamine, 0.0001 mg/L; trichloroacetate, 0.2 mg/L; and 2,4,6-trichlorophenol, 0.2 mg/L.

Although there is no collective MCLG in the United States for this contaminant group, there are individual MCLGs for some of the individual contaminants:

1. Trihalomethanes: Bromodichloromethane (0); bromoform (0); dibromochloromethane (0.06 mg/L); chloroform (0.07 mg/L).
2. Haloacetic acids: Dichloroacetic acid (0); trichloroacetic acid (0.02 mg/L); monochloroacetic acid (0.07 mg/L). Bromoacetic acid and dibromoacetic acid are regulated with this group but have no MCLGs.

10.5.12 Other Organic and Inorganic Contaminants

There are simply too many organic and inorganic contaminants to be introduced individually. The readers are referred to Section 10.8 for details. Many contaminants regulated by the US EPA are not regulated by Canada, Mexico, or WHO. The reverse is also true. For instance, the MAL of Mexico and the GV of WHO for DDT are both 0.001 mg/L, but DDT is not regulated by the US EPA at all. The specific types of toxic organics and inorganics and their derivatives in water sources can differ between countries or regions depending on social, cultural, technological, and agricultural factors. The toxic DDT and its metabolites are not regulated in the United States because DDT has been totally banned by the US EPA for its application.

Another example is the application and regulatory control of acrylamide and epichlorohydrin in the United States. Each water system must certify, in writing, to the state (using third-party or manufacturer's certification) that when acrylamide and epichlorohydrin are used in drinking water systems, the combination (or product) of dose and monomer level does not exceed the levels specified, as follows: acrylamide = 0.05% dosed at 1 mg/L (or equivalent) and epichlorohydrin = 0.01% dosed at 20 mg/L (or equivalent). The two chemicals, however, are not regulated in many other countries.

Pharmaceuticals are the emerging contaminants introduced into surface and groundwater sources by discharges from individuals using these chemicals, from uncontrolled drug disposal, and from agricultural runoff from livestock manure. Routine monitoring for pharmaceuticals in drinking water and advanced drinking water treatment to reduce the pharmaceuticals may become necessary in the future.

10.6 BIOLOGICAL CHARACTERISTICS AND CONSTITUENTS

Five categories of parasitic organisms infective or undesirable to man are found in water: bacteria, protozoa, worms, viruses, fungi, and algae. Bacteria, protozoa, worms are classified as animals, which have the following characteristics: (a) release energy and may have sensory organs and a nervous system, (b) release carbon dioxide, (c) cell walls composed principally of protein matter, a nitrogen bearing compound, (d) metabolism depends upon digestion of food within an alimentary canal, and (e) digestion process is "analytic" by which ingested materials are decomposed to supply materials for synthesis of proteins required for growth and reproduction. Algae are organisms in the plant kingdom, which have the following characteristics: (a) store energy, (b) have no sensory organs or nervous system, (c) release oxygen, (d) cell walls composed of cellulose—a carbohydrate, (e) metabolism depends upon absorption of water and gases by root hairs and natural openings called stomata, and there is no digestion of food, and (f) utilize carbon dioxide from the air and water and nitrogen salts to synthesize carbohydrates and proteins needed for growth and reproduction. Bacteria, protozoa, worms, viruses, and fungi are disease-producing organisms. Some of these complete their life cycle by passage through an intermediary aquatic host; others are merely transported by water from man to man. Fungi and algae are responsible for taste and odor in water and therefore are undesirable.

10.6.1 Bacteria

The principal bacterial waterborne diseases of the middle latitudes, typhoid fever and cholera, are two highly specific infections that exacted their awful toll of sickness and death in the cities emerging from the industrial revolution. *Paratyphoid* (*salmonellosis*) and *bacillary dysentery* (*shigellosis*) as well as *hemorrhagic jaundice* (*leptospirosis*) are waterborne diseases in a less direct sense.

Leptospirosis, which is also known as hemorrhagic jaundice or Weil's disease, is traceable to swimming or wading in polluted canals, streams, and lakes. Rats and dogs are among the carriers of the spirochetes causing the fairly large group of associated diseases.

Since there are several thousand species of bacteria, it is difficult to control pathogenic bacteria in drinking water individually and effectively. Coliform bacteria are naturally present in the environment. Total coliforms (including fecal coliform and *Escherichia coli* (*E. coli*)) which cause no health threat in themselves are used to indicate whether other potentially harmful bacteria may be present in drinking water. No MCL/MAC/MAL is specified for heterotrophic plate count (HPC) bacteria in drinking water, however. Instead, increases in HPC concentrations above baseline levels are considered undesirable.

Legionella, *Mycobacterium avium complex*, *Aeromonas hydrophila*, and *Helicobacter pylori* are considered as the emerging bacterial waterborne pathogens in Canada. These emerging pathogens have been linked to gastrointestinal illness in human populations. In the United States, only *Legionella* which causes a type of pneumonia (Legionnaire's disease) is being controlled by an effective TT, and the US EPA-specified MCLG is zero.

10.6.2 Protozoa (Including *Cryptosporidium* and *Giardia lamblia*)

Although it is estimated that between 1% and 10% of the US population are carriers of amebic cysts, the reported incidence of waterborne amebic dysentery (*amebiasis*) has been low. Among the probable reasons are (a) the relatively small number of cysts excreted by carriers and (b) the relatively large size and weight of the cysts, which account for their natural removal from water by sedimentation as well as by filtration. Relatively massive incursion of pollution into water-distribution systems by backflow from house drainage systems and by cross-connections with unsafe water supplies has generally been associated with waterborne outbreaks.

Giardia infection can cause a variety of intestinal signs or symptoms, which include diarrhea, gas or flatulence, greasy stools that tend to float, stomach or abdominal cramps and upset stomach, or nausea. These symptoms may lead to weight loss and dehydration.

Cryptosporidium is a parasite commonly found in lakes and rivers, especially when the water is contaminated with sewage and animal wastes. *Cryptosporidium* has caused several large waterborne disease outbreaks of gastrointestinal illness, with symptoms that include diarrhea, nausea, and/or stomach cramps. Despite not being identified until 1976, it is one of the most frequent causes of waterborne disease (drinking water and recreational water) among humans in the United States. In March and April 1993, an outbreak of *cryptosporidiosis* in Milwaukee resulted in diarrheal illness in an estimated 403,000 persons.

Although *Giardia* and *Cryptosporidium* can be responsible for severe and, in some cases, fatal gastrointestinal illness, it is impossible to establish numerical MCL (United States), MAC (Canada), or MAL (Mexico) for these pathogenic protozoa in drinking water at present. In the United States, the MCLG of *Giardia* and *Cryptosporidium* in drinking water is zero. In the United States and Canada, the TT in place should achieve at least a 3-log reduction and/or inactivation of cysts and oocysts, unless source water quality requires a greater log reduction and/or inactivation. TT is a required process intended to reduce the level of a contaminant in drinking water. The US Surface Water Treatment Rules require water systems using surface water or groundwater under the direct influence of surface water to (a) disinfect their water and (b) filter their water or meet criteria for avoiding filtration so that the following contaminants are controlled at the following levels: 99% removal/inactivation of *Cryptosporidium*, *99.9%* removal/inactivation of *Giardia lamblia*, and 99.99% removal/inactivation of viruses.

10.6.3 Worms

The eggs and larvae of intestinal worms may reach water courses from human and animal carriers either directly or in washings from the soil. The eggs or larvae involved are relatively small in number, and the organisms themselves are relatively large in size. Hence worm infections are sporadic and occur only under grossly insanitary conditions or through gross mismanagement of wastewater disposal systems. Irrigation of crops that are consumed raw may transmit any one of the common intestinal worms; irrigation of grasslands may infect cattle and, through them, man.

In some parts of the world the minute crustacean *Cyclops* ingests the larvae of the guinea worm, a nematode or roundworm, that infects man through drinking water and is released by him to water again when skin ulcers filled with larvae break while he wades or swims in freshwater.

Since worms are much bigger than protozoa (including *Cryptosporidium* and *Giardia lamblia*), it is assumed that after *Giardia lamblia* and *Cryptosporidium* are removed by the required TT, such as membrane filtration, all worms will also be removed.

10.6.4 Viruses, Fungi, and Algae

Viruses are the smallest microorganisms known to scientists. Organisms this small will pass freely through the openings of most filters or membranes. Viral diseases are many and affect all forms of life. Aside from the virus of infectious hepatitis, members of half a dozen virus groups, constituting over 100 different strains, are known to be excreted in the feces of infected persons. Isolation of these enteric organisms is especially common during the summer months, when streams are normally low. Yet the number of waterborne outbreaks of virus diseases has been small. Moreover, only two of the six groups—the Echo and Reo groups but neither the Polio, Coxsackie A and B, and Adeno groups, nor the hepatitis virus—produce enteric symptoms in man. Why water has not played a more significant role in the spread of these infections awaits explanation. Their small size is favorable to water transport, and the relatively small numbers in which they are probably excreted—apparently of the order of 10^6 daily per infected person rather than the 10^{11} for typhoid organisms—may militate against their completing the cycle from man to man via the water route. Other gastrointestinal upsets, apparently waterborne, possibly of viral origin, and associated with heavy pollution of water supplies, more especially during periods of severe and prolonged drought, are on record in the United States as well as abroad.

Rickettsiae are similar in some respects to viruses and in others to bacteria. They are intermediate in size between viruses and bacteria, from 0.3 to 0.5 micron for most rickettsiae cells. They multiply by fission as bacteria do, but like viruses must do so in the presence of a host. *Rickettsial* diseases are common, such as Rocky Mountain spotted fever, Q fever, and typhus which are diseases of man.

Fungi unlike algae have no chlorophyll in their cellular structure. Water forms of fungi include *phycomycetes*, similar to algae, and *ascomycetes*, commonly called yeasts and molds. Although parasitic fungi occur in water, they do not appear to significantly infect man through water.

Algae are organisms containing chlorophyll. They are frequently found in surface waters exposed to sunlight and during the summer months. They become so numerous as to produce “blooms” which appear to completely cover whole sections of a reservoir or lake. Under the influence of sunlight chlorophyll enables the organism to combine water and carbon dioxide to form complex chemicals and produce oxygen as a by-product. Although the production of oxygen by the photosynthesis process is beneficial in that it promotes oxidation of organic debris, supports life, algae are the chief cause of bad tastes and odors in a water supply. Algae are normally controlled at the raw water source, such as a lake or reservoir using an algacide.

Although enteric viruses and fungi can be responsible for severe and, in some cases, fatal illnesses, it is impossible to establish MCL/MAC/MAL for enteric viruses and fungi in drinking water at present. Nevertheless, the US (MCLG is set at zero for enteric viruses. TT and watershed or wellhead protection measures known to reduce the risk of waterborne outbreaks should be implemented and maintained if source water is subject to water contamination or if enteric viruses/fungi have been responsible for past waterborne outbreaks. In Canada, where viruses removal is required, TT should achieve at least a 4-log reduction and/or inactivation for water quality control. In the United States, the water systems using surface water or groundwater under the direct influence of surface water are required to (a) disinfect their water and (b) filter their water or meet criteria for avoiding filtration so that *Cryptosporidium*, *Giardia lamblia*, and viruses can be removed/inactivated by 99%, 99.9%, and 99.99%, respectively.

10.6.5 Coliform Indicator Parameter

Pathogenic microorganisms from the biotechnology industry, agricultural industry, hospitals, and so on may cause waterborne diseases. Coliforms are naturally present in the environment as well as feces; fecal coliforms and *E. coli* come only from human and animal fecal waste. Coliform bacteria are also called enteric bacteria. The terms “coliform” and "enteric" are simply labels that describe bacteria that are found in the digestive tract and, as a result, in fecal material. The term "fecal coliforms" are considered to be those coliform group bacteria that are present specifically in the gut and feces of warm-blooded animals. The major species in the fecal coliform group is *E. coli*. Of the five genera of coliform bacteria, only *E. coli* is generally not found growing and reproducing in the environment. As a result, *E. coli* is considered to be the coliform bacteria species that is most indicative of fecal pollution and the possible presence of enteric pathogens. The term "total coliform" refers to the sum of the coliform bacteria present in a given water sample. All coliform bacteria tests begin by placing water in or on a growth medium for reproduction of all species of coliforms. If a sample is identified as positive, it means that one or more of the coliform bacteria are present. Fecal coliforms are the subgroup of the total coliforms that can ferment lactose in 24 ± 2 hours at 44.5 ± 0.2°C. *E. coli* are those fecal coliforms that can hydrolyze beta glucuronides. Figure 10.2 shows the relationship of total coliforms, fecal coliforms, and *E. coli* and their respective characteristics. Fecal coliform and *E. coli* are bacteria whose presence indicates that the water may be contaminated with human or animal wastes. Disease-causing microbes (pathogens) in these waters can cause diarrhea, cramps, nausea, headaches, or other symptoms. These pathogens may pose a special health risk for infants, young children, and people with severely compromised immune systems.

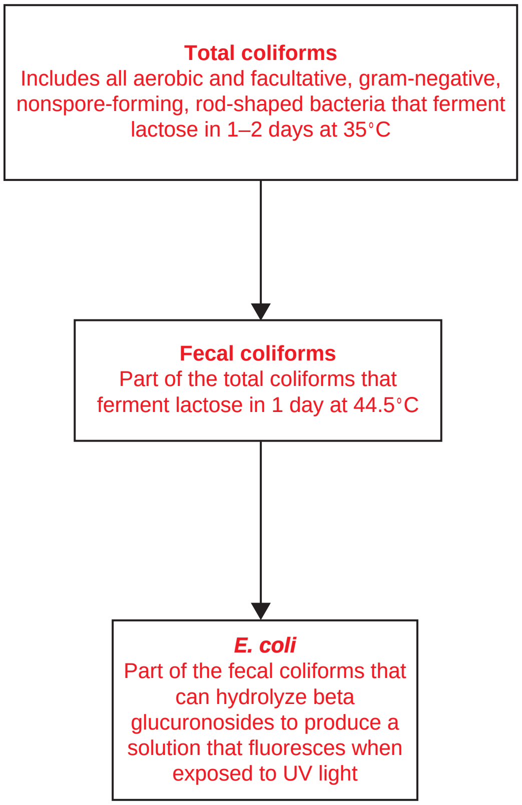

Figure 10.2 Characteristics of the coliform bacteria.

Coliform bacteria—regulated by both the US EPA and the state governments—are only indicators showing whether or not the water has been properly disinfected. For a disinfected water, a zero count of coliform bacteria indicates that the water is properly disinfected, and other microorganisms are assumed to be sterilized. The US Drinking Water Standards have established limits for the mean concentration of coliform bacteria in a series of water samples and the frequency at which concentrations may exceed the mean. The results are expressed either in terms of a direct count of bacteria per unit volume—if the *membrane filter* (MF) procedure is used—or in terms of the "*most probable number*" (MPN)—if the multiple tube fermentation method is used. This latter term (MPN) is an estimate based on mathematical formulas of probability. The recommended standards for drinking water are roughly equivalent to restricting the coliform concentration to not more than one organism for each 100 milliliters of water. Application of the US Drinking Water Standards to individual water supplies is difficult due to the low frequency with which samples can be properly collected and examined. Bacteriological examinations indicate the presence or absence of contamination in the collected sample only and are indicative of quality only at the time of collection. A sample positive for coliforms is a good indication that the source may have been contaminated by surface washings or fecal material. On the other hand, a negative result cannot be considered assurance of a continuously safe supply unless the results of a thorough sanitary survey of the surrounding area, together with subsequent negative samples, support this position.

In the United States, the MCLG of total coliforms in drinking water is set at "zero," but the MCL is more complicated. For water systems that collect 40 or more routine water samples per month, in no more than 5% samples *total coliform-positive* (TC-positive) in a month should be detected. For water systems that collect fewer than 40 routine samples per month, no more than one sample can be TC-positive per month. Every sample that has total coliform must be analyzed for either fecal coliforms or *E. coli*. If there are two consecutive TC-positive samples, and one is also positive for *E. coli* fecal coliform, the water system has an acute MCL violation.

10.6.6 Heterotrophic Plate Count (HPC)

HPC has no health effects; it is an analytic method used to measure the variety of bacteria that are common in water. The lower the concentration of bacteria in drinking water, the better maintained the water system is. HPC measures a range of bacteria that are naturally present in the environment. No MAC/MAL has been specified by Canada and Mexico for HPC bacteria in drinking water. However, increases in HPC concentrations above baseline levels are considered undesirable. In the United States, no more than 500 bacterial colonies per milliliter of HPC is allowed in drinking water.

10.6.7 Infections from Water-Related Sources

If infections conveyed by excrement as well as by wastewaters and their sludges are taken into account, the ways of spreading enteric disease increase in number and the list of possible infections becomes longer. Common modes of transmission other than through drinking water are (1) through watercress, or shellfish harvested from or stored in sewage-polluted water (typhoid, paratyphoid, bacillary dysentery, and infectious hepatitis); (2) through vegetables and fruits contaminated by feces, sewage, or sewage sludge (typhoid, paratyphoid, the dysenteries, parasitic worms, and infectious hepatitis); (3) through exposure to soil contaminated by human dung (hookworm); (4) through all manner of food contaminated by flies and other vermin that feed also on human fecal matter (typhoid, paratyphoid, the dysenteries, and infectious hepatitis); (5) through milk and milk products contaminated by utensils that have been washed in polluted water (typhoid, paratyphoid, and bacillary dysentery); (6) through fish and crayfish from polluted waters eaten raw too soon after salting (flukes and tapeworms); and (7) through bathing or other exposure to polluted waters (*leptospirosis* and *schistosomiasis*).

Other chains of infection link (1) tuberculosis to the milk of cows infected by drinking from polluted streams running through pastures below tuberculosis sanitaria; (2) eye, ear, nose, and throat infections to heavily patronized bathing pools even though their waters are well treated and contain appreciable amounts of free chlorine; and (3) enteric diseases, in general, to lack of hand-washing facilities and to primitive methods of excreta disposal.

10.6.8 Reduction of Infections by Water Quality Management

The accomplishments of water quality control are best exemplified for water supply, for which cause and effect are usually quite clear. For wastewater this is not so, because there can be no sewerage without water supply and the results of sewerage are masked by the impact of water supply. For discernible effects, one must look, instead, at the record of enteric disease in rural and urban areas. To this purpose Leach and Maxcy recorded the rates of typhoid fever per 100,000 population in communities of different sizes or types, shown in Table 10.1. These statistics they interpreted as showing relatively good sanitary protection (1) in rural areas because of lack of contact and (2) in larger communities because of good community sanitation, including both water supply and wastewater disposal.

Table 10.1 Typhoid fever and size of community

Size of population	Morbidity[a] (per 100,000)
Rural	52
500–1,000	443
1,000–2,500	307
2,500–5,000	180
5,000–10,000	165
10,000–25,000	118
>25,000	63

[a] Morbidity is defined as the "cases per 100,000 population."

An example of what can be accomplished by the introduction and intelligent management of public water supplies is offered by the Commonwealth of Massachusetts. In this state an engineering division was organized in 1886 to protect the purity of inland waters. Among its accomplishments was an increase in public water supplies and with it the striking reduction in typhoid fever portrayed in Table 10.2.

An observation that should be made in the interest of developing countries is that the introduction of water supplies imposes a peculiarly heavy responsibility on water authorities for strict and effective supervision of water quality. Otherwise, drinking water may become the disseminator of enteric infections in large-scale epidemics such as occurred in North America during the second half of the nineteenth century. Important, too, is the realization that epidemics of enteric infections like typhoid fever leave large numbers of chronic carriers in their wake. What should be deduced from American experience is that a rising tide of waterborne disease must have preceded the high-water mark reached by typhoid fever before careful management of public water supplies during the first half of the twentieth century could dam the flooding tide.

10.7 RADIOLOGICAL CHARACTERISTICS AND CONSTITUENTS

The development and use of atomic energy as a power source and mining of radioactive materials have made it necessary to establish limiting concentrations for the intake into the body of radioactive substances, including drinking water. The effects of human exposure to radiation or radioactive materials are viewed as harmful and any unnecessary exposure should be avoided. The concentrations of radioactive materials specified in the current US Drinking Water Standards are intended to limit the human intake of these substances

Table 10.2 Reduction of typhoid fever accompanying the spread of public water supplies in Massachusetts

Year	Population with public water supply (%)	Deaths from typhoid fever (per 100,000)
1890	80	37
1900	90	22
1910	92	12
1920	94	2
1930	96	2
1940	97	1

so that the total radiation exposure of any individual will not exceed those defined in the Radiation Protection Guides recommended by the US EPA. Man has always been exposed to natural radiation from water, food, and air. The amount of radiation to which the individual is normally exposed varies with the amount of background radioactivity. Water of high radioactivity is unusual. Nevertheless it is known to exist in certain areas, either from natural or man-made sources.

Gross alpha particle activity, gross beta particle activity, and total radium-226 and radium-228 are found from radioactive wastes, uranium deposits, and certain geological formations and are cancer-causing energy. The MCLs for gross alpha particle activity, gross beta particle activity, and total radium-226 and radium-228 and uranium are set by the US EPA at alpha particles = 15 pCi/L; beta particles and photo emitters = 4 mrem/year; combined radium-226 and radium-228 = 5 pCi/L; and uranium = 0.03 mg/L. Again the state government may have more stringent Drinking Water Guidelines. For instance, the Commonwealth of Massachusetts Drinking Water Standards include additional photon activity, tritium, strontium-90, and radon-222 for the state enforcement. Radon in groundwater can be effectively removed by granular activated carbon.

The health-based MAC guidelines for radiological parameters in Canada are different and listed below: cesium-137 = 10 Bq/L; iodine-131 = 6 Bq/L; lead-210 = 0.2 Bq/L; radium-226 = 0.5 Bq/L; strontium-90 = 5 Bq/L; tritium = 7,000 Bq/L; and uranium = 0.02 mg/L. Becquerel (Bq) is the unit of activity of a radioactive substance, or the rate at which transformations occur in the substance. One becquerel (1 Bq) is equal to one transformation per second and approximately equals to 27 picocuries (pCi).

10.8 DRINKING WATER QUALITY STANDARDS

Selected portions of the WHO's *Guidelines for Drinking-water Quality* and the Canada's *Guidelines for Canadian Drinking Water Quality* are available in the appendices of this book. The Mexico Drinking Water Standards are discussed in Problem 10.14. This section only summarizes and briefly discusses the US drinking water standards.

In the United States the standards promulgated and revised from time to time by the US EPA are widely followed by governmental agencies and public utilities.

The Water Standards include general rules relating to water sources and their protection and specific rules defining required bacteriological quality and acceptable limits of significant physical and chemical characteristics of water delivered to the consumer. There are two types of rules *primary and secondary*.

The National Primary Drinking Water Regulations (NPDWRs or primary standards) are legally enforceable standards that apply to public water systems (PWSs). Primary standards protect public health by limiting the levels of contaminants in drinking water. These regulations cover the microorganisms (Table 10.3), DBP (Table 10.4), disinfectants (Table 10.5), inorganic chemicals (Table 10.6), organic chemicals (Table 10.7), and radionuclides (Table 10.8). *The National Secondary Drinking Water Regulations* (NSDWRs or secondary standards) are nonenforceable guidelines regulating contaminants that may cause cosmetic effects (such as skin or tooth discoloration) or aesthetic effects (such as taste, odor, or color) in drinking water. US EPA recommends secondary standards (Table 10.9) to water systems but does not require systems to comply. However, states may choose to adopt them as enforceable standards.

The number of bacteriological samples collected from representative points throughout the water-distribution system must be in keeping with the size of the population at risk. Typical minimum monthly numbers are prescribed as follows:

Population served (thousands)	1–2	10	50	100	900	2,000	4,500
Number of samples	2	12	50	95	300	400	500

The physical characteristics of water supplied to a community should be examined at least once a week, samples again being drawn from representative points throughout the system. Turbidity, color, odor, and taste should not be so high as to offend the senses of sight, taste, or smell. Maximum acceptable values for surface waters used without treatment other than disinfection are 5 units of turbidity, 15 units of color, and a TON of 3. For filtered surface waters and for groundwaters, these values are excessively high.

US EPA's surface water treatment rules require systems using surface water or groundwater under the direct influence of surface water to (1) disinfect their water and (2) filter their water or meet criteria for avoiding filtration so that the following contaminants are controlled at the following levels:

- *Cryptosporidium*: At least 99% removal.
- *Giardia lamblia*: 99.9% removal/inactivation.
- Viruses: 99.99% removal/inactivation.
- *Legionella*: No limit, but US EPA believes that if *Giardia* and viruses are removed/inactivated, *Legionella* will also be controlled.
- Turbidity: At no time can turbidity (cloudiness of water) go above 5 nephelolometric turbidity units (NTU); systems that filter must ensure that the turbidity go no higher than 1 NTU (0.5 NTU for conventional or direct filtration) in at least 95% of the daily samples in any month. Turbidity may never exceed 1 NTU, and must not exceed 0.3 NTU in 95% of daily samples in any month.
- Heterotrophic plate count (HPC): No more than 500 bacterial colonies/mL.
- Long Term 1 Enhanced Surface Water Treatment: Surface water systems or GWUDI systems (i.e.,

Table 10.3 US National Primary Drinking Water Regulations—microorganisms

Contaminant	MCLG[c]	MCL[c] or TT[a]	Potential health effects from ingestion of water	Sources of contaminant in drinking water
Cryptosporidium	Zero	TT	Gastrointestinal illness (e.g., diarrhea, vomiting, cramps)	Human and animal fecal waste
Giardia lamblia	Zero	TT	Gastrointestinal illness (e.g., diarrhea, vomiting, cramps)	Human and animal fecal waste
Heterotrophic plate count (HPC)	n/a #/mL	TT #/mL	HPC has no health effects; it is an analytic method used to measure the variety of bacteria that are common in water. The lower the concentration of bacteria in drinking water, the better maintained the water system is	HPC measures a range of bacteria that are naturally present in the environment
Legionella	Zero	TT	Legionnaire's disease, a type of pneumonia	Found naturally in water; multiplies in heating systems
Total coliforms (including fecal coliform and *E. coli*)	Zero #/100 mL	5.0%[b] #/100 mL	Not a health threat in itself; it is used to indicate whether other potentially harmful bacteria may be present[b]	Coliforms are naturally present in the environment as well as feces; fecal coliforms and *E. coli* only come from human and animal fecal waste
Turbidity	n/a NTU	TT NTU	Turbidity is a measure of the cloudiness of water. It is used to indicate water quality and filtration effectiveness (e.g., whether disease-causing organisms are present). Higher turbidity levels are often associated with higher levels of disease-causing microorganisms such as viruses, parasites, and some bacteria. These organisms can cause symptoms such as nausea, cramps, diarrhea, and associated headaches	Soil runoff
Viruses (enteric)	zero	TT	Gastrointestinal illness (e.g., diarrhea, vomiting, cramps)	Human and animal fecal waste

[a] TT, treatment technique.

[b] See Sections10.3, 10.4 and 10.6.1 to 10.6.6.

[c] MCL, maximum contaminant level; MCLG, maximum contaminant level goal.

Table 10.4 US National Primary Drinking Water Regulations—disinfection by-products residuals (DBPR)

Contaminant	MCLG (mg/L)[a]	MCL or TT (mg/L)[b]	Potential health effects from ingestion of water	Sources of contaminant in drinking water
Bromate	Zero	0.010	Increased risk of cancer	By-product of drinking water disinfection
Chlorite	0.8	1.0	Anemia; infants and young children: nervous system effects	By-product of drinking water disinfection
Haloacetic acids (HAAs)	n/a[b]	0.060[c]	Increased risk of cancer	By-product of drinking water disinfection
Total trihalomethanes (TTHMs)	n/a[b]	0.080[c]	Liver, kidney, or central nervous system problems; increased risk of cancer	By-product of drinking water disinfection

[a] Units are in mg/L unless otherwise noted.

[b] See Sections 10.5.10 and 10.5.11.

[c] The MCL values are the same in the stage 2 DBPR as they were in the stage 1 DBPR, but compliance with the MCL is based on different calculations. Under stage 1, compliance is based on a running annual average (RAA). Under stage 2, compliance is based on a locational running annual average (LRAA), where the annual average at each sampling location in the distribution system is used to determine compliance with the MCLs. The LRAA requirement became effective on April 1, 2012, for systems on schedule 1; October 1, 2012, for systems on schedule 2; and October 1, 2013, for all remaining systems.

Table 10.5 US National Primary Drinking Water Regulations—disinfectants[a]

Contaminant	MRDLG (mg/L)	MRDL (mg/L)	Potential health effects from ingestion of water	Sources of contaminant in drinking water
Chloramines (as Cl_2)	4.0	4.0	Eye/nose irritation; stomach discomfort, anemia	Water additive used to control microbes
Chlorine (as Cl_2)	4.0	4.0	Eye/nose irritation; stomach discomfort	Water additive used to control microbes
Chlorine dioxide (as ClO_2)	0.8	0.8	Anemia; infants and young children: nervous system effects	Water additive used to control microbes

[a] See Section 10.5.10.

the water systems that treat the groundwater under direct influence of surface water) serving fewer than 10,000 people must comply with the applicable Long Term 1 Enhanced Surface Water Treatment Rule provisions (e.g., turbidity standards, individual filter monitoring, *Cryptosporidium* removal requirements, updated watershed control requirements for unfiltered systems).

- Long Term 2 Enhanced Surface Water Treatment Rule: Surface water systems or GWUDI systems must comply with the additional treatment for *Cryptosporidium* specified in this rule.
- Filter backwash recycling: The Filter Backwash Recycling Rule requires systems that recycle to return specific recycle flows through all processes of the system's existing conventional or direct filtration system or at an alternate location approved by the state.

Important chemical characteristics should be determined at least twice a year. More frequent analyses are required when there are reasonable doubts about the constancy of recorded information or when the supply is fluoridated.

Drinking water should not contain impurities in hazardous concentrations, be excessively corrosive, or retain treatment chemicals in excessive concentrations. Substances that are possibly deleterious physiologically should not be permitted to reach the consumer. When, in the judgment of the reporting agencies and certifying authorities, other more suitable supplies are or can be made available, chemical impurities should not be present in quantities above the concentrations shown in Table 10.6. The presence of toxic substances in excess of the concentrations listed constitutes grounds for rejection of the supply.

Table 10.5 shows that the US Primary Drinking Water Standards for fluoride is 4 mg/L, while Table 10.9 shows that the US Secondary Drinking Water Standards for fluoride is 2 mg/L. It means that fluoride present in drinking water should not average more than 4 mg/L (enforceable), but it is recommended that fluoride be limited to 2 mg/L (nonenforceable). Where fluorides are added to drinking water, their average concentration should be kept within the upper and lower control limits shown in Table 10.10.

10.9 INDUSTRIAL WATER QUALITY STANDARDS

Industries may set stricter or more lenient quality standards than are commonly subscribed to by municipalities. Thus the common criteria for cooling waters are normally broader or less specific than for process waters and boiler feed waters. Generally prescribed for *cooling waters is* that they shall neither generate scales or sludges in process or ancillary plant equipment nor support slimes, insect larvae, mussels, or other aquatic organisms in conduits, tanks, and related portions of the cooling system. Cooling waters can be conserved by recycling. Salt and brackish waters are used as well as fresh water. In fact, about 20% of the water supplied to US industry is brackish. Most of it comes from the sea, but some is pumped from the ground. Saline waters are most readily acceptable if they are to be used in *once-through* cooling. However, the inland disposal of brackish waters is difficult—so difficult, indeed, as to prohibit drawing heavily on this resource. Deep-well injection may offer the only satisfactory means of disposal.

Most *process waters* must be cleaner than cooling waters. Municipal supplies are generally good enough for process use but not necessarily for boiler feed. By contrast, about 60% of the water developed by industry itself has to be purified in order to meet process needs. In tanneries the proportion is as little as 16%; in the manufacture of photographic supplies it is 100%. Process waters, too, may be either recycled or reused in other processes for which their quality is or can be made suitable. Removal of secondary contaminants may then offer opportunities for salvaging heat and useful product or process components.

To illustrate the scope of quality objectives, the production of canned goods, milk, meats, beverages, and ice normally imposes sanitary requirements on process waters that surpass drinking water standards. In general this is so because drinking water can be protected by chlorination and because foods must usually be free from color, tastes, and odors other than their own. Excessive hardness and trace concentrations of iron, manganese, and other metals may also be objectionable. Many paper-making processes, for instance, cannot tolerate even small amounts of iron, manganese, or hardness. By contrast, breweries, distilleries, and bakeries

Table 10.6 US National Primary Drinking Water Regulations—inorganic chemicals

Contaminant	MCLG (mg/L)[a]	MCL or TT (mg/L)[a]	Potential health effects from ingestion of water	Sources of contaminant in drinking water
Antimony	0.006	0.006	Increase in blood cholesterol; decrease in blood sugar	Discharge from petroleum refineries; fire retardants; ceramics; electronics; solder
Arsenic	Zero[c]	0.010	Skin damage or problems with circulatory systems, and may have increased risk of getting cancer	Erosion of natural deposits; runoff from orchards, runoff from glass and electronics production wastes
Asbestos (fiber >10 micrometers)	7 million fibers/L	7 million fibers/L	Increased risk of developing benign intestinal polyps	Decay of asbestos cement in water mains; erosion of natural deposits
Barium	2	2	Increase in blood pressure	Discharge of drilling wastes; discharge from metal refineries; erosion of natural deposits
Beryllium	0.004	0.004	Intestinal lesions	Discharge from metal refineries and coal-burning factories; discharge from electrical, aerospace, and defense industries
Cadmium	0.005	0.005	Kidney damage	Corrosion of galvanized pipes; erosion of natural deposits; discharge from metal refineries; runoff from waste batteries and paints
Chromium (total)	0.1	0.1	Allergic dermatitis	Discharge from steel and pulp mills; erosion of natural deposits
Copper	1.3	TT[b] action level = 1.3	Short term exposure: gastrointestinal distress. Long-term exposure: liver or kidney damage. People with Wilson's disease should consult their personal doctor if the amount of copper in their water exceeds the action level	Corrosion of household plumbing systems; erosion of natural deposits
Cyanide (as free cyanide)	0.2	0.2	Nerve damage or thyroid problems	Discharge from steel/metal factories; discharge from plastic and fertilizer factories
Fluoride	4.0	4.0	Bone disease (pain and tenderness of the bones); children may get mottled teeth	Water additive which promotes strong teeth; erosion of natural deposits; discharge from fertilizer and aluminum factories
Lead	Zero	TT[b] action level = 0.015	Infants and children: delays in physical or mental development; children could show slight deficits in attention span and learning abilities. Adults: kidney problems; high blood pressure	Corrosion of household plumbing systems; erosion of natural deposits
Mercury (inorganic)	0.002	0.002	Kidney damage	Erosion of natural deposits; discharge from refineries and factories; runoff from landfills and croplands
Nitrate (measured as nitrogen)	10	10	Infants below the age of 6 months who drink water containing nitrate in excess of the MCL could become seriously ill and, if untreated, may die. Symptoms include shortness of breath and blue-baby syndrome	Runoff from fertilizer use; leaching from septic tanks, sewage; erosion of natural deposits
Nitrite (measured as nitrogen)	1	1	Infants below the age of 6 months who drink water containing nitrite in excess of the MCL could become seriously ill and, if untreated, may die. Symptoms include shortness of breath and blue-baby syndrome	Runoff from fertilizer use; leaching from septic tanks, sewage; erosion of natural deposits
Selenium	0.05	0.05	Hair or fingernail loss; numbness in fingers or toes; circulatory problems	Discharge from petroleum refineries; erosion of natural deposits; discharge from mines
Thallium	0.0005	0.002	Hair loss; changes in blood; kidney, intestine, or liver problems	Leaching from ore-processing sites; discharge from electronics, glass, and drug factories

[a] Units are in mg/L unless otherwise noted.

[b] See Sections 10.5.1 and 10.5.2.

[c] The MCL values are the same in the stage 2 DBPR as they were in the stage 1 DBPR, but compliance with the MCL is based on different calculations. Under stage 1, compliance is based on a running annual average (RAA). Under stage 2, compliance is based on a locational running annual average (LRAA), where the annual average at each sampling location in the distribution system is used to determine compliance with the MCLs. The LRAA requirement became effective on April 1, 2012, for systems on schedule 1; October 1, 2012, for systems on schedule 2; and October 1, 2013, for all remaining systems.

Table 10.7 US National Primary Drinking Water Regulations—organic chemicals

Contaminant	MCLG (mg/L)[a]	MCL or TT (mg/L)[a]	Potential health effects from ingestion of water	Sources of contaminant in drinking water
Acrylamide	Zero	TT[b]	Nervous system or blood problems; increased risk of cancer	Added to water during sewage/wastewater treatment
Alachlor	Zero	0.002	Eye, liver, kidney or spleen problems; anemia; increased risk of cancer	Runoff from herbicide used on row crops
Atrazine	0.003	0.003	Cardiovascular system or reproductive problems	Runoff from herbicide used on row crops
Benzene	Zero	0.005	Anemia; decrease in blood platelets; increased risk of cancer	Discharge from factories; leaching from gas storage tanks and landfills
Benzo(a)pyrene (PAHs)	Zero	0.0002	Reproductive difficulties; increased risk of cancer	Leaching from linings of water storage tanks and distribution lines
Carbofuran	0.04	0.04	Problems with blood, nervous system, or reproductive system	Leaching of soil fumigant used on rice and alfalfa
Carbon tetrachloride	Zero	0.005	Liver problems; increased risk of cancer	Discharge from chemical plants and other industrial activities
Chlordane	Zero	0.002	Liver or nervous system problems; increased risk of cancer	Residue of banned termiticide
Chlorobenzene	0.1	0.1	Liver or kidney problems	Discharge from chemical and agricultural chemical factories
2,4-D	0.07	0.07	Kidney, liver, or adrenal gland problems	Runoff from herbicide used on row crops
Dalapon	0.2	0.2	Minor kidney changes	Runoff from herbicide used on rights of way
1,2-Dibromo-3-chloropropane (DBCP)	Zero	0.0002	Reproductive difficulties; increased risk of cancer	Runoff/leaching from soil fumigant used on soybeans, cotton, pineapples, and orchards
o-Dichlorobenzene	0.6	0.6	Liver, kidney, or circulatory system problems	Discharge from industrial chemical factories
p-Dichlorobenzene	0.075	0.075	Anemia; liver, kidney, or spleen damage; changes in blood	Discharge from industrial chemical factories
1,2-Dichloroethane	Zero	0.005	Increased risk of cancer	Discharge from industrial chemical factories
1,1-Dichloroethylene	0.007	0.007	Liver problems	Discharge from industrial chemical factories
cis-1,2-Dichloroethylene	0.07	0.07	Liver problems	Discharge from industrial chemical factories
trans-1,2-Dichloroethylene	0.1	0.1	Liver problems	Discharge from industrial chemical factories
Dichloromethane	Zero	0.005	Liver problems; increased risk of cancer	Discharge from drug and chemical factories
1,2-Dichloropropane	Zero	0.005	Increased risk of cancer	Discharge from industrial chemical factories
Di(2-ethylhexyl) adipate	0.4	0.4	Weight loss, liver problems, or possible reproductive difficulties	Discharge from chemical factories
Di(2-ethylhexyl) phthalate	Zero	0.006	Reproductive difficulties; liver problems; increased risk of cancer	Discharge from rubber and chemical factories
Dinoseb	0.007	0.007	Reproductive difficulties	Runoff from herbicide used on soybeans and vegetables
Dioxin (2,3,7,8-TCDD)	Zero	0.00000003	Reproductive difficulties; increased risk of cancer	Emissions from waste incineration and other combustion; discharge from chemical factories
Diquat	0.02	0.02	Cataracts	Runoff from herbicide use
Endothall	0.1	0.1	Stomach and intestinal problems	Runoff from herbicide use

(*continued*)

Table 10.7 (*Continued*)

Contaminant	MCLG (mg/L)[a]	MCL or TT (mg/L)[a]	Potential health effects from ingestion of water	Sources of contaminant in drinking water
Endrin	0.002	0.002	Liver problems	Residue of banned insecticide
Epichlorohydrin	Zero	TT[b]	Increased cancer risk and over a long period of time, stomach problems	Discharge from industrial chemical factories; an impurity of some water treatment chemicals
Ethylbenzene	0.7	0.7	Liver or kidneys problems	Discharge from petroleum refineries
Ethylene dibromide	Zero	0.00005	Problems with liver, stomach, reproductive system, or kidneys; increased risk of cancer	Discharge from petroleum refineries
Glyphosate	0.7	0.7	Kidney problems; reproductive difficulties	Runoff from herbicide use
Heptachlor	Zero	0.0004	Liver damage; increased risk of cancer	Residue of banned termiticide
Heptachlor epoxide	Zero	0.0002	Liver damage; increased risk of cancer	Breakdown of heptachlor
Hexachlorobenzene	Zero	0.001	Liver or kidney problems; reproductive difficulties; increased risk of cancer	Discharge from metal refineries and agricultural chemical factories
Hexachlorocyclopentadiene	0.05	0.05	Kidney or stomach problems	Discharge from chemical factories
Lindane	0.0002	0.0002	Liver or kidney problems	Runoff/leaching from insecticide used on cattle, lumber, gardens
Methoxychlor	0.04	0.04	Reproductive difficulties	Runoff/leaching from insecticide used on fruits, vegetables, alfalfa, livestock
Oxamyl (Vydate)	0.2	0.2	Slight nervous system effects	Runoff/leaching from insecticide used on apples, potatoes, and tomatoes
Polychlorinated biphenyls (PCBs)	Zero	0.0005	Skin changes; thymus gland problems; immune deficiencies; reproductive or nervous system difficulties; increased risk of cancer	Runoff from landfills; discharge of waste chemicals
Pentachlorophenol	Zero	0.001	Liver or kidney problems; increased cancer risk	Discharge from wood-preserving factories
Picloram	0.5	0.5	Liver problems	Herbicide runoff
Simazine	0.004	0.004	Problems with blood	Herbicide runoff
Styrene	0.1	0.1	Liver, kidney, or circulatory system problems	Discharge from rubber and plastic factories; leaching from landfills
Tetrachloroethylene	Zero	0.005	Liver problems; increased risk of cancer	Discharge from factories and dry cleaners
Toluene	1	1	Nervous system, kidney, or liver problems	Discharge from petroleum factories
Toxaphene	Zero	0.003	Kidney, liver, or thyroid problems; increased risk of cancer	Runoff/leaching from insecticide used on cotton and cattle
2,4,5-TP (Silvex)	0.05	0.05	Liver problems	Residue of banned herbicide
1,2,4-Trichlorobenzene	0.07	0.07	Changes in adrenal glands	Discharge from textile finishing factories
1,1,1-Trichloroethane	0.20	0.20	Liver, nervous system, or circulatory problems	Discharge from metal degreasing sites and other factories
1,1,2-Trichloroethane	0.003	0.005	Liver, kidney, or immune system problems	Discharge from industrial chemical factories
Trichloroethylene	Zero	0.005	Liver problems; increased risk of cancer	Discharge from metal degreasing sites and other factories
Vinyl chloride	Zero	0.002	Increased risk of cancer	Leaching from PVC pipes; discharge from plastic factories
Xylenes (total)	10	10	Nervous system damage	Discharge from petroleum factories; discharge from chemical factories

[a] Units are in mg/L unless otherwise noted.

[b] See Sections 10.5.7 to 10.5.9 and 10.5.12.

Table 10.8 US National Primary Drinking Water Regulations—radionuclides[b]

Contaminant	MCLG	MCL or TT	Potential health effects from ingestion of water	Sources of contaminant in drinking water
Alpha particles	None[a] zero	15 picocuries per liter (pCi/L)	Increased risk of cancer	Erosion of natural deposits of certain minerals that are radioactive and may emit a form of radiation known as alpha radiation
Beta particles and photon emitters	None[a] zero	4 millirems per year	Increased risk of cancer	Decay of natural and man-made deposits of certain minerals that are radioactive and may emit forms of radiation known as photons and beta radiation
Radium-226 and radium-228 (combined)	None[a] zero	5 pCi/L	Increased risk of cancer	Erosion of natural deposits
Uranium	Zero	30 μg/L	Increased risk of cancer, kidney toxicity	

[a] The MCL values are the same in the stage 2 DBPR as they were in the stage 1 DBPR, but compliance with the MCL is based on different calculations. Under stage 1, compliance is based on a running annual average (RAA). Under stage 2, compliance is based on a locational running annual average (LRAA), where the annual average at each sampling location in the distribution system is used to determine compliance with the MCLs. The LRAA requirement became effective on April 1, 2012, for systems on schedule 1; October 1, 2012, for systems on schedule 2; and October 1, 2013, for all remaining systems.
[b] See Section 10.7.

prefer hard waters. In electroplating, trace substances may destroy brightness, corrosion resistance, and other primary properties of deposited metals. Brackish waters that are satisfactory for cooling may be too corrosive for other uses; the chloride tolerance of steel rolling mills, for example, is only 150 mg/L. Specific quality standards for industrial process waters are identified in Table 10.11.

10.10 BATHING WATERS

The examination of swimming pool and other bathing waters must have somewhat different objectives from the examination of drinking water. There are distinctions, too, between safeguarding water in swimming pools and at bathing beaches, more particularly saltwater beaches. Swimming pool water is readily amenable to purification, including maintenance of disinfecting concentrations of chlorine or other halogens; the water at bathing beaches and similar places is not. The use of bathing beaches and outdoor pools, on the other hand, is normally confined to the warm season of the year. Bright sunlight and low incidence of respiratory infections may then combine with less crowding of bathers at most bathing beaches to decrease health hazards.

Table 10.9 List of US National Secondary Drinking Water Regulations

Contaminant	Secondary standard
Aluminum	0.05–0.2 mg/L
Chloride	250 mg/L
Color	15 (color units)
Copper	1.0 mg/L
Corrosivity	Noncorrosive
Fluoride	2.0 mg/L
Foaming agents	0.5 mg/L
Iron	0.3 mg/L
Manganese	0.05 mg/L
Odor	3 threshold odor number
pH	6.5–8.5
Silver	0.10 mg/L
Sulfate	250 mg/L
Total dissolved solids	500 mg/L
Zinc	5 mg/L

In the absence of satisfactory tests for nose, mouth, and throat organisms, as well as skin organisms, the 24-h, 37°C agar plate count serves a useful purpose in the examination of *swimming pool waters.* It is generally supplemented by tests for coliform bacteria as measures of the effectiveness of bather supervision, all bathers being required to take a cleansing shower before entering the pool or returning to it after using the toilet.

Reasoning statistically, the bacteriological standards for bathing waters should be more stringent, the longer the bathing season and the more likely the ingestion of bathing water (by small children, for instance). Accordingly, saltwater bathing should be relatively safer. Judgment of needed water quality should be based also on the prevalence of enteric disease in the drainage area, information obtained in pollution surveys, and bacteriological examinations.

Natural bathing waters should be free from *schistosome cercariae* that can infect man or cause skin irritation (swimmer's itch) and from *leptospirae* of all types.

10.11 FISHING AND SHELLFISH WATERS

Researchers have listed five categories or groups of impurities as hazardous to fish life: (1) matter that settles, such as sawdust, and deprives fish of natural foods by depositing

Table 10.10 Recommended control limits concentrations of fluoride in drinking water

Annual average of maximum daily air temperatures, °F (°C)	Recommended control limits in fluoride concentrations (mg/L)		
	Lower	Optimum	Upper
50.0–53.7 (10.0–12.1)	0.9	1.2	1.7
53.8–58.3 (12.2–14.6)	0.8	1.1	1.5
58.4–63.8 (14.7–17.7)	0.8	1.0	1.3
63.9–70.6 (17.8–21.4)	0.7	0.9	1.2
70.7–79.2 (21.5–26.2)	0.7	0.8	1.0
79.3–90.5 (26.3–32.5)	0.6	0.7	0.8

Conversion factor: °C = (°F − 32) × 5/9.

a pollutional carpet on the bottom of streams and lakes; (2) substances that exert sufficient oxygen demand to lower the DO content below the level needed to support fish in their normal spawning, foraging, migrating, and other activities at all stages of development. To flourish at normal water temperatures, most food fish require at least 4, and trout at least 5, mg/L of DO. As water temperatures rise, the biochemical oxygen demand (BOD) increases, the DO saturation value declines, and the rate of respiration of the fish and their threshold of asphyxiation go up. For a temperature rise of 10°C, for example, the oxygen intake of goldfish increases more than threefold and the point of asphyxiation of trout almost twofold. In the absence of adequate DO, moreover, fish are more susceptible to toxic metals and other hazards; (3) compounds that lift the pH above 8.4 or drop it below 6.8, more or less, may be directly lethal, and pH changes may throw out of balance the tolerances of fish to high temperatures and low DO concentrations. Acid wastes are especially detrimental; (4) among wastes that increase salinity and, with it, osmotic pressure, are oil-well brines. A specific conductance per cm of 150–500 mho × 10^{-6} at 25°C, with a maximum of 1,000–2,000 mho × 10^{-6}, is considered permissible; and (5) wastes that contain specifically toxic substances include insecticides such as the chlorinated hydrocarbons and organic phosphorus compounds that have become of special interest because of their wide use in pest control. The death of minnows offers a direct and meaningful screening test for tolerable limits and threshold limits of toxic substances.

Shellfish Waters: Shellfish—more specifically oysters, clams, and mussels—grown in polluted waters have been responsible for important outbreaks of disease, including typhoid fever and infectious hepatitis. The reasons for this are understandable. The bivalves pass large volumes of water through their gills and other organs and strain out food particles that include living organisms of many kinds. Moreover, some shellfish are eaten raw. Accordingly, the

Table 10.11 Quality tolerances for industrial process waters

Industry	Turbidity*	Color**	Hardness	Alkalinity	Fe + Mn (mg/L)	Total solids (mg/L)	Other
			as mg/L of $CaCO_3$				
Food products							
Baked goods	10	10	†	…	0.2	…	*a*
Beer	10	—	…	75–150	0.1	500–1,000	*a*,*b*
Canned goods	10	…	25–75	…	0.2	…	*a*
Confectionery	…	…	…	…	0.2	100	*a*
Ice	5	5	…	30–50	0.2	300	*a*,*c*
Laundering	…	…	50	…	0.2	…	…
Manufactured products							
Leather	20	10–100	50–135	135	0.4	…	…
Paper	5	5	50	…	0.1	200	*d*
Paper pulp	15–20	10–20	100–180	…	0.1–1.0	200–300	*e*
Plastics, clear	2	2	…	…	0.02	200	…
Textiles, dyeing	5	5–20	20	…	0.25	…	*f*
Textiles, general	5	20	20	…	0.5	…	…

Stated values are general averages only.

There is much local variance.

†Some hardness is desirable.

a Must conform to standards for potable water.

b NaCl no more than 275 mg/L

c SiO_2 no more than 10 mg/L; Ca and Mg bicarbonates are troublesome; sulfates and chlorides of Na, Ca, and Mg each no more than than 300 mg/L

d No slime formation.

e Noncorrosive.

f Constant composition; residual alumina no more than 0.5 mg/L

*Turbidity unit = NTU

**color unit = CU

Source: After Fair et al. (1971).

saline waters of tidal estuaries in which they are cultivated must be unpolluted.

10.12 IRRIGATION WATERS

The quality of irrigation waters is of interest in relation to (1) resource developments in which available waters are exploited in parallel for agricultural and municipal purposes, (2) schemes in which the waters made available for urban use are derived wholly or in part from the underflow of irrigated fields, and (3) wastewater disposal by irrigating agricultural areas either by direct discharge from the drainage system or by diversion of sewage-polluted receiving waters. Narrower in its implications, yet of broad concern, is the quality of municipal waters applied to the parks, lawns, and gardens of the community. McKee and Wolf have summarized the properties of irrigation waters excellently in a fashion suited to the watering of most plants under most conditions.

Because the introduction of wastewater systems accompanies an advancing economy, it is generally possible to limit or dispense with utilization of the fertilizing constituents of wastewaters in order to protect the public health. Accordingly most health authorities prohibit the irrigation of vegetables, garden truck, berries, or low-growing fruits with partially treated or undisinfected municipal wastewaters. The watering of vineyards or orchards where windfalls or fruit lie on the ground is also forbidden. Only nursery stock; vegetables raised exclusively for seed purposes; cotton; and field crops such as hay, grain, rice, alfalfa, fodder corn, cow beets, and fodder carrots are allowed to be watered with municipal wastewaters. However, milk cows and goats are not permitted to be pastured on irrigated land moist with such wastewaters and must be kept away from irrigation ditches that carry them. Even when produce from irrigated areas is to be cooked before consumption, irrigation with wastewaters must be stopped at least a month prior to harvest. Commercial canning of irrigated crops is sometimes permitted under proper control by health authorities. Where water is scarce, full reclamation of wastewaters for watering rather than fertilizing may be justified.

10.13 QUALITY OF WATER FROM VARIOUS SOURCES

Virtually all the water used to supply human requirements has at some time, usually quite recently, fallen to the surface of the earth as rain or some other form of precipitation. At this stage, the quantity of foreign material it contains is likely to be at a minimum. Nevertheless, even rain water is not chemically pure H_2O. Not only does it dissolve the gases of the atmosphere as it falls, but it also collects dust and other solid materials suspended in the air. Since the atmospheric solids depend upon both the composition of the soil below and the materials released into the air from combustion, industrial processes, and other sources, analyses of rain or other forms of precipitation reveal surprising variations. In general, however, rainwater may be expected to be very soft, to be low in total solids and alkalinity, to have a pH value somewhat below neutrality, and to be quite corrosive to many metals. A *typical* analysis, subject to the variations mentioned above, might appear as in Column 3) of Table 10.12.

After the water reaches the surface of the ground, it passes over soil and rock into lakes, streams, and reservoirs, or it percolates through the soil and rock into the groundwater. In the process, a great variety of materials may be dissolved or taken into suspension. Consequently, it may be expected that the composition of both the surface waters and the groundwater of a given area reflects the geology of the region, that is, the composition of the underlying rock formations and of the soils derived from them. In general, the

Table 10.12 Comparison of water quality from various sources

Constituent (1)	Unit (2)	Rain water (3)	Surface water from geologic limestone formation (4)	Well water in limestone area (5)	Surface water from insoluble soil/rock region (6)	Groundwater from insoluble soil/rock region (7)
Hardness	mg/L as $CaCO_3$	19	120	201	46	61
Calcium	mg/L as $CaCO_3$	16	80	142	30	29
Magnesium	mg/L as $CaCO_3$	3	40	59	16	32
Sodium & Potassium	mg/L as Na	6	19	20	9	26
Bicarbonate	mg/L as $CaCO_3$	12	106	143	42	60
Chloride	mg/L	9	23	23	5	7
Sulfate	mg/L	10	38	59	12	17
Nitrate	mg/L as N	0.1	0.4	0.06	1.5	—
Iron	mg/L	—	0.3	0.18	1.1	1.8
Silica	mg/L as SiO_2	—	18	12	30	—
Carbon Dioxide	mg/L as $CaCO_3$	4	4	14	—	53
pH	Units	6.8	7.8	7.4	—	6.6
Acidity	mg/L as $CaCO_3$	4	—	—	—	—
Ammonium ion	mg/L as N	0.8	—	—	—	—

presence of readily soluble formations near the surface, such as gypsum, rock salt, or the various forms of limestone, produces relatively marked effects upon the waters of the area. On the other hand, in the presence of less soluble formations, such as sandstone or granite, the composition of the water tends to remain more like that of rain. As one might expect, local variations are often considerable and occasionally extreme, both in the concentration of any one constituent and in the proportions of the various materials present. The examples given below should be considered with this in view. They are *typical* only in that they are not remarkable.

Surface water, in an area in which limestone is an important constituent of the geologic formations, might have a constituent's composition similar to that shown in Column 4) of Table 10.12.

In such an area, the groundwater often contains more hardness and bicarbonate than the surface waters. This is due in part to the longer period of contact with soil and rock and in part to the fact that carbon dioxide, contributed by the decomposition of organic matter in the soil, greatly increases the solubility of some of the constituents. The analysis shown in Column 5 of Table 10.12 might be considered *typical* of well or spring water in a limestone area.

In areas in which the underlying formations are insoluble, that is, where they consist of sand, sandstone, clay, shale, or igneous rocks, the waters tend to be softer and more acidic. In general, their content of most dissolved materials is lower. Acidity, however, may be higher than in hard water areas, since carbon dioxide picked up from the soil is not neutralized. Excepting in some areas of igneous rock, iron also tends to be higher in soft waters, since many of the iron compounds of soils and rocks are dissolved by the acidity of the waters. In many soft water areas, the differences between groundwaters and surface waters are not as pronounced as in hard water regions, although many exceptions to this generality could be cited.

A more or less typical analysis of surface water in a region of generally insoluble soils and rocks is shown in Column 6 of Table 10.12.

Groundwater from a similar region might give analytical results similar to the constituents shown in Column 7) of Table 10.12.

It is worth reemphasizing that each of the constituents listed in the analyses above may vary over a wide range from place to place. For example, waters are known with hardness values of less than 10 mg/L, and others have concentrations over 1,000 mg/L. Those quoted have been chosen to represent rather moderate, ordinary values occurring in two distinct types of situations common in the United States. It would be a mistake, however, to expect any water sample to correspond exactly to any one of the analyses given as examples.

10.14 GOOD QUALITY WATER

Since waters from various sources may vary so markedly in composition, one may reasonably question which source should be considered most desirable. The problem has several practical consequences. For example, if a choice exists among several available sources, the final decision may rest upon judgment of their relative quality. Also, when the composition is modified by treatment, the objective is to approach, if not always to attain, the ideal.

The characteristics of *good quality water* are implied in earlier sections of this chapter, which discuss the objectives of water management and the standards formally adopted by the US EPA. Reviewing those sections will make it evident that the properties desired are mostly negative. That is, the objectives and standards are directed principally to avoiding undesirable qualities. The properties of *good water* may then be summarized in qualitative terms as follows:

1. Absence of harmful concentrations of toxic chemical substances
2. Absence of the causative microorganisms and viruses of disease
3. Lowest possible levels of color, turbidity, suspended solids, odor, and taste
4. Lowest possible temperature
5. Minimum corrosivity to metals
6. Least possible tendency to deposit scale
7. Lowest possible content of staining materials, such as iron, manganese, and copper

This may appear to suggest that the ideal water contains the lowest possible quantity of total solids but this is not the case. Extremely soft waters tend to be excessively corrosive to metals, and many persons find them unpalatable. Moreover, they seem to be less effective in removing soap by rinsing than waters containing a little hardness.

Although there has been no formal recognition of a set of analytical values characterizing the ideal water, the values shown in Table 10.13 would probably be considered generally acceptable as an approximation.

The relationships among calcium, bicarbonate, carbon dioxide, and pH should be such as to minimize scaling and corrosion. In some cases, these concentrations may dictate the most desirable concentrations of sulfate, chloride, magnesium, sodium, and potassium.

10.15 SELF-PURIFICATION AND STORAGE

Nature provides some degree of self-purification for all water that has been polluted or contaminated by the introduction of wastes, whether they originate as domestic sewage, industrial wastes, or drainage from yards, streets, and agricultural areas. The rate at which process occurs depends upon the nature and amount of polluting material as well as the physical, chemical, and biological conditions and characteristics of the water itself. Erroneous ideas are prevalent, however, particularly as to the value of aeration and its effect on flowing water. For instance, statements are sometimes made

Table 10.13 Generally acceptable characteristics of ideal water

Constituent	Concentration
Alkyl benzene sulfonate	Less than 0.1 mg/L, preferably 0
Arsenic	Less than 0.01 mg/L, preferably 0
Barium	Less than 1 mg/L, preferably 0
Bicarbonate	150 mg/L as $CaCO_3$
Cadmium	Less than 0.01 mg/L, preferably 0
Calcium	70 mg/L as $CaCO_3$
Carbon chloroform extract	Less than 0.2 mg/L, preferably 0
Carbon dioxide	6 mg/L as $CaCO_3$
Chloride	Less than 250 mg/L, preferably 0
Chromium, hexavalent	Less than 0.05 mg/L, preferably 0
Coliform bacteria	Less than 1 per 100 mL
Color	Less than 15 color units, preferably 0
Copper	Less than 1 mg/L, preferably 0
Cyanide	Less than 0.01 mg/L, preferably 0
Fluoride	Approximately 0.9 mg/L
Hardness	70 mg/L as $CaCO_3$
Iron	Less than 0.1 mg/L, preferably 0
Lead	Less than 0.05 mg/L, preferably 0
Magnesium	Preferably 0
Manganese	Less than 0.02 mg/L, preferably 0
Nitrate	Less than 10 mg/L, preferably 0
pH	7.8
Phenols	Less than 0.001 mg/L, preferably 0
Selenium	Less than 0.01 mg/L, preferably 0
Silver	Less than 0.05 mg/L, preferably 0
Sodium and potassium	37 mg/L as Na
Sulfate	Less than 250 mg/L, preferably 0
Suspended solids	Not detectable
Temperature	33–40°F (0.6–4.4°C)
Threshold odor number	Less than 3 TON, preferably 0
Total dissolved solids	Less than 500 mg/L
Turbidity	Less than 5 NTU, preferably 0
Zinc	Less than 5 mg/L, preferably 0

to the effect that "water will purify itself in flowing seven miles" or that natural aeration occurring at waterfalls and rapids will "oxidize" or kill bacteria. Actually, distance in itself has nothing whatever to do with self-purification in a flowing stream. Neither does aeration have much if any direct effect in killing bacteria. Time is the important factor, together with proper conditions of temperature, sunlight, velocity of flow, and many other complex chemical, physical, and biological characteristics. Quiescent sedimentation in a reservoir for a period of about a month may result generally in purification equivalent to that of filtration. Sluggish flow in a stream for a long distance may accomplish the same results.

Essentially, the same action takes place in a natural lake or in an impounding. Furthermore, a considerable amount of vertical mixing may occur due to variations in the density of the water. The changes of density, in turn, are caused by the differences of temperature of the water at the various levels in the lake or reservoir. The vertical mixing takes place continuously, but is most noticeable in the spring and fall when temperature changes are most rapid and mixing consequently most vigorous throughout the entire depth of the water. Very often this *turnover* of a lake or reservoir results in the occurrence of tastes and odors in the water supply, which may be due to changes in the types and numbers of microorganisms present or to changes in the chemical and physical quality of the water.

10.16 OBJECTIVES OF WATER EXAMINATION

Basic to successful water quality management is an understanding of the manageable properties of water under the wide variety of conditions in which it is found on the earth and put to use by man. Accordingly, water is examined to identify its salient properties and, if need be, their amenity to change. This is not a routine undertaking, even though it may appear to be so. The properties of a given source of water vary with (1) its hydrology, in time and season, as well as distance of passage over and through the soil, and (2) its use, in flow through collection, transmission, purification, and distribution works. In a similar fashion, but in stepped-up intensity, the properties of a given wastewater vary with (1) its use, in flow through collection, treatment, and disposal works, and (2) the hydrology of the bodies of water into which the wastewaters are discharged, in time and season, as well as distance of passage over and through the soil.

Most pronounced are the variations in properties of water during purification and treatment and, with them, the requirements for analytical supervision or monitoring of water quality and response. Depending on the objectives of quality management, monitoring information may be provided by (1) automatic sensing or sampling and measuring devices, (2) samples collected in the field and carried to the laboratory for analysis, (3) functional testing of existing or contemplated operational procedures, and (4) research procedures through which new insight is gained into the behavior of water under promising conditions of quality management or control.

A good way to anticipate the probable composition of water samples in preparation for their analysis and to explain the analytical results obtained is to become familiar with the conditions under which these waters occur in nature. The scope of field surveys is dictated by circumstances. A single survey, no matter how carefully made, shows only what conditions were at the times and places of sampling, no more.

10.17 METHODS OF EXAMINATION

If the results obtained by different laboratories are to be comparable and have legal validity, suitable methods of collection and analysis must be agreed on by the profession and accepted by government. In the United States, *Standard Methods for the Examination of Water and Wastewater* have been prepared, approved, and published jointly by the Water

Environment Federation, American Public Health Association, and American Water Works Association, and these methods are used in support of water quality standards at all levels of responsible government. Water quality standards normally identify the concentration of component properties shown by experience or scientific judgment to be safe, desirable, and acceptable, and to be attainable from available water sources. Effluent quality standards serve as criteria for the maintenance of acceptable conditions in receiving bodies of water or on land areas. Together these standards must rest on a broad understanding of the regional water and land economies and on the cost of available treatment methods.

10.18 STANDARD TESTS

Although many of the tests employed in the examination of water and wastewater samples are identical for both kinds of water, the information sought through their analysis may have very different objectives. The essential purpose of an analysis of municipal water is to determine raw water quality, probable need for and response to purification, possible change during distribution, and usefulness in households and industry. The essential purpose of a wastewater analysis is to find the composition, concentration, and condition of the raw wastewater, possible effects on the collecting system, probable response to treatment, and possible influences on receiving bodies of water or land.

Domestic waters are characterized as safe or unsafe, pure or impure, palatable or unpalatable, hard or soft, corrosive or stable, and sweet or saline, as the case may be. Industrial waters are categorized in terms of the purposes they are intended to serve. Some of the tests included in water analyses are more useful and more generally applicable than others. Some are of long standing and forge important bonds with the past; others are more recent in concept. Some give direct information on specific constituents; others are inferential in character.

10.19 EXPRESSION OF ANALYTICAL RESULTS

The results of chemical analysis are commonly expressed in *milligrams per liter* (mg/L) or as *parts per million* (ppm). For industrial wastes of high specific gravity, however, it is customary to use mg/L only and to add the specific gravity. For concentrations below 1 mg/L, *micrograms per liter* (μg/L) or *parts per billion* (ppb) may be substituted; for high concentrations (above 10^5 mg/L), *percentage values* are useful. Other useful expressions are *quantity units*, such as *pounds per day* (lb/day) or *kilograms per day* (kg/day); and the *population equivalent* (more especially for the BOD) equal to lb/day and divided by the pertinent per capita release (such as 0.17 lb/day/capita or 77.3 gm/day/capita for BOD). Mineral analyses may be reported as concentrations of ions, expressing the results in milliequivalents per liter (me/L), and equal to the sum of the atomic weights of constituent atoms divided by the number of charges normally associated with the different ions.

Hydrogen ion concentration is stated in terms of the *pH value*, or negative logarithm of the hydrogen ion concentration. Specific conductivity measurements for TDS, for example, are expressed in mhos or *μmhos/cm*, mhos being the reciprocal of resistance, or ohms^{-1}. Odor intensity is recorded as the reciprocal of the dilution ratio with odor-free water at which the odor remains just faintly discernible. Color and turbidity are reported in *units of color* and *units of turbidity* corresponding to standard color and turbidity simulants based on 1 mg/L of platinum in potassium chloroplatinate for color and 1 mg/L of silica in *diatomaceous* or *fuller's earth* for turbidity. Bacteriological results are reported, respectively, as *plate counts/mL;* as *MPNs* of coliform organisms/*100 mL* when the serial-dilution technique is employed; and as coliform colonies/100 mL when the MF technique is used. Algae and other plankton organisms are counted as individual cells, or their size, area, or bulk is measured in standard units 20 μ in length, 400 μ^2 in area, or 8,000 μ^3 in volume.

10.20 TAPPING A SOURCE OF WATER

PWSs come in all shapes and sizes, and no two are exactly the same. They may be publicly or privately owned and maintained. All PWSs must have at least 15 service connections or serve at least 25 people per day for 60 days of the year. Drinking water standards apply to water systems differently based on their type and size:

- *Community water system* (there are approximately 54,000): A PWS that serves the same people year-round. Most residences including homes, apartments, and condominiums in cities, small towns, and mobile home parks are served by community water systems (CWSs).
- *Noncommunity water system*: A PWS that serves the public but does not serve the same people year-round. There are two types of noncommunity systems:
  - *Nontransient noncommunity water system* (there are approximately 20,000): A noncommunity water system that serves the same people more than 6 months per year, but not year-round; for example, a school with its own water supply is considered a nontransient system.
  - *Transient noncommunity water system* (there are approximately 89,000): A noncommunity water system that serves the public but not the same individuals for more than 6 months; for example, a rest area or campground may be considered a transient water system.

While PWSs design may vary, they all share the same goal: providing safe, reliable drinking water to the communities they serve. To do this, most water systems must treat their water. The types of treatment provided by a specific PWS vary depending on the size of the system, whether they use groundwater or surface water, and the quality of the source water.

Large-scale water supply systems tend to rely on surface water sources, while smaller systems tend to rely on groundwater. Around 32% of the populations served by CWSs drink water that originates as groundwater. Groundwater is usually pumped from wells ranging from shallow to deep (50–1,000 ft or 15–305 m). The remaining 68% of the population served by CWSs receive water taken primarily from surface water sources like rivers, lakes, and reservoirs. Due to rapid development of membrane filtration technology, ocean water has become a new source of water supply.

PROBLEMS/QUESTIONS

10.1 Discuss what is implied by the following phrase using examples of rainwater, surface water, and groundwater: "The source of water determines its inherent quality."

10.2 What are the five categories of parasitic organisms found in water that are infective to man?

10.3 Discuss the importance of the presence of *Giardia* and *Cryptosporidium* in water supply.

10.4 What are the sources of the variety of toxic substances that find their way into water supplies?

10.5 Explain the difference between the primary and secondary drinking water regulations and standards.

10.6 Differentiate between MCL and MCLG.

10.7 The desired properties of good quality water are mostly negative. That is, the objectives and standards are directed principally to avoiding undesirable qualities of impurities. Summarize these water qualities in qualitative terms.

10.8 It may appear that the ideal drinking water contains the lowest possible quantity of total solids. Is this true or false and why?

10.9 The general appearance of a stream provides a useful guide to the degree of pollution. Describe the four zones of self-purification in a stream.

10.10 Field surveys of water quality are called either sanitary surveys, pollutional surveys, or industrial waste surveys. Explain when each naming is applied.

10.11 Water samples may be either grab samples or composite samples. Explain the difference between the two types.

10.12 The maximum allowable concentration of nitrite in drinking water by the standards are 1.0 mg/L as N (United States), 0.05 mg/L as N (Mexico), and 3 mg/L as NO_2^- (WHO). Compare the three standards expressing the nitrite concentration as N.

10.13 Table 10.14 shows the characteristics of two water resources A and B. In your opinion, which source is likely to be groundwater and which is surface water? Give your reasons.

Table 10.14 Water analysis of two water samples for Problem 10.13

Constituent	Unit	Water A	Water B
Turbidity	NTU	15	4
Coliforms	#/100 mL	20	2
Hardness	mg/L as $CaCO_3$	140	225
Calcium	mg/L as $CaCO_3$	90	170
Sodium	mg/L as Na	30	80
Bicarbonate	mg/L as $CaCO_3$	116	175
Chloride	mg/L	23	123
Sulfate	mg/L	48	79
Nitrate	mg/L as N	0.4	0.6
Iron	mg/L	0.3	0.2
Silica	mg/L as SiO_2	20	10

10.14 Assume that you are an American engineer working on a water engineering project in Mexico. Find the current Mexico Drinking Water Standards from the Internet and compare the current Mexico standards with the current US Primary and Secondary Drinking Water Standards.

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Chapter 11

Water Treatment Systems

11.1 PURPOSE OF WATER TREATMENT

The variety of operations that bring about the purification of water is large and the phenomena with which water-quality management must occupy itself are many. This may explain the high degree of empiricism that characterizes much of the information on which the design of treatment works and their operation must still be based. Yet it has become possible to identify, by carefully controlled laboratory experiments, the fundamental nature of the purification responses of water and to interpret them mathematically. In an important and far-reaching sense, therefore, the concepts and models that have been created for this purpose offer the prospect of codifying patterns of purification behavior shared in common and laying a foundation of purification theory that will strengthen the art of water treatment by giving it the support of a more perfect science.

The methods employed in the treatment of water depend, to a large extent, on the purpose for which the supply is to be used and the quality of the water being treated. For domestic use, it is desirable to remove any materials, either in suspension or in solution, which are detrimental to the appearance and esthetic appeal of the water. It is absolutely necessary to remove, inactivate, or kill any detrimental microorganisms and to remove harmful chemical substances. On the other hand, industrial requirements for water quality vary, depending upon the use. For example, for steam generation the control of scale formation is of paramount importance to most industrial plants, while textile mills and paper mills demand freedom from iron and manganese. The pharmaceutical industry requires ultra-pure water for production of drugs.

In general, the many methods normally employed in water treatment practice usually have as their main objective the reduction of the total quantity of foreign substances in the water. Even when the treatment process involves the addition of certain materials, the end result is usually the removal of more material than has been added. There are cases, however, in which certain constituents are removed by substituting other substances, and in some circumstances the content of certain substances may be increased deliberately, in order to impart certain desirable characteristics to the water.

It is the purpose of this chapter to introduce the student of water-quality management to the fundamental ways and means, or unit operation, by which water purification is affected. By proceeding in this way, it will become evident that the study of water treatment is marked by "a quest for unity behind the appearance of the many."

11.2 TREATMENT OF RAW WATER

The amount and type of treatment applied by a public water system (PWS) varies with the source type and quality. Many groundwater systems can satisfy all federal requirements without applying any treatment, while others need to add chlorine or additional treatment. Because surface water systems are exposed to direct wet weather runoff and to the atmosphere and are therefore more easily contaminated, federal and state regulations require that these systems treat their water. Disinfection of drinking water is one of the major public health advances of the twentieth century. However, the disinfectants themselves can react with naturally occurring materials in the water to form unintended disinfection by-products (DBPs), which may pose health risks. A major challenge for water suppliers is balancing the risks from microbial pathogens and DBPs. In the United States, the Stage 1 Disinfectants and Disinfection By-products Rule (Stage 1 DDBR) and the Enhanced Surface Water Treatment Rule (ESWTR) together address these risks. Other countries have similar rules and regulations. Water suppliers use a variety of treatment processes to remove contaminants from drinking water. These individual processes may be arranged in a "treatment train" (a series of processes applied in sequence).

The unit operations and unit processes most commonly used in a public water supply system include screening, pumping, aeration, chemical feeding, rapid mixing, coagulation–flocculation, clarification (sedimentation clarification or flotation clarification), filtration, disinfection, and corrosion control. Some treatment trains also include ion exchange (IX), recarbonation, membrane filtration, activated alumina (AA) adsorption, granular activated carbon (GAC) adsorption, powdered activated carbon (PAC) adsorption, ultraviolet (UV), diatomaceous earth (DE) filtration,

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Water Engineering: Hydraulics, Distribution and Treatment, First Edition. Nazih K. Shammas and Lawrence K. Wang.

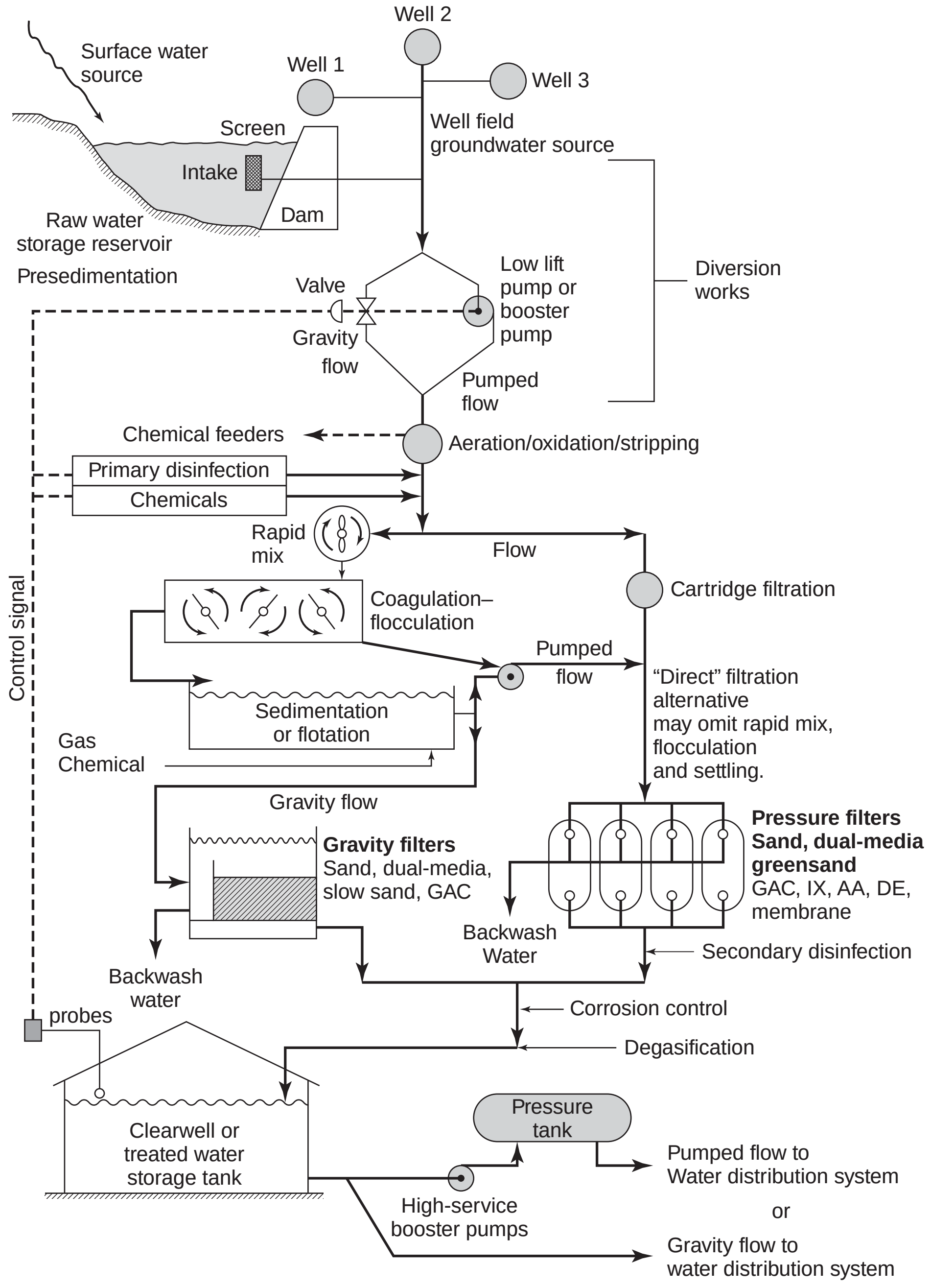


Figure 11.1 Selection of water treatment systems for municipalities (A modified figure).

dual-media filtration, slow sand filtration, and advanced oxidation processes (AOPs). Clearwell, water storage, and water distribution are not unit processes, but are required components of a complete water supply system. Water utilities select a combination of treatment processes most appropriate to treat the contaminants found in the raw water used by the system. Figure 11.1 shows the schematic of a typical public water supply system for treating groundwater and/or surface water and GWUDI (groundwater under direct influence of surface water).

The most important factor in the selection of an appropriate water treatment system is the raw water quality. The water system can be very simple if the raw water quality is excellent, or very complex if the raw water has many contaminants. Without knowing the raw water quality details, a general concept of water treatment can be found from Fig. 11.1. The center and left-hand sides of the figure indicate the water treatment train for surface water (including GWUDI)—presedimentation, screening, pumping, aeration, primary disinfection, chemical feeding, rapid mixing, coagulation–flocculation, sedimentation (or flotation), gravity filtration, corrosion control, secondary disinfection, clearwell, and water storage and distribution. For treatment of groundwater, presedimentation, screening, rapid mixing, coagulation–flocculation, and sedimentation (or flotation) usually can be omitted. The center and right-hand sides of Fig. 11.1 show the common water treatment train for groundwater—pumping water from well field, aeration, primary disinfection, chemical feeding, pressure filtration (sand, dual-media filtration, greensand filtration, granular activated carbon (GAC), ion exchange (IX), activated alumina (AA), diatomaceous earth filtration (DE), membrane filtration, etc.), corrosion control, secondary disinfection, clearwell, and water storage and distribution. There are exceptions, however. Alternative treatment trains and chemical use should be considered as means of reducing waste handling and disposal problems. Bench-scale laboratory tests, pilot plant studies, or full-scale demonstrations may be required to establish adequacy for meeting some specific water quality standards. Various surface water and groundwater treatment systems are introduced in the examples and practiced in the home work problems.

EXAMPLE 11.1 THE SIMPLEST WATER SUPPLY SYSTEM TO TREAT A GROUNDWATER SOURCE

The purposes of water treatment are (a) producing potable water, which meets all drinking water standards; (b) maintaining the high water quality throughout the entire water distribution system; (c) providing sufficient water flow and pressure throughout the entire water distribution system; and (d) protecting the water distribution system. Figure 11.2 shows the well field and the water distribution system. Assume the raw well water has an excellent quality that meets all drinking water standards for five consecutive years, except for its pH, which is in the range of 5.5–6.0. Recommend a simple public water supply system and explain the purposes of each recommended water facility.

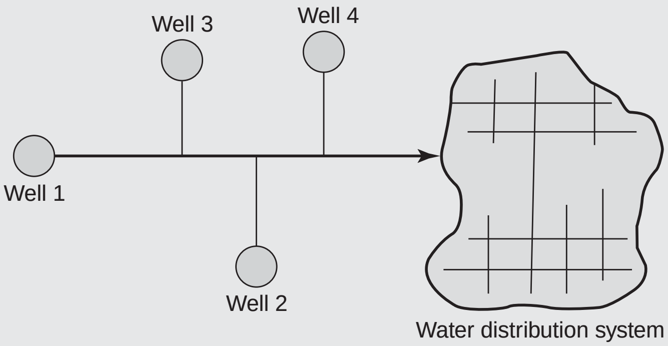


Figure 11.2 Well field and water distribution system.

Solution:

Since the raw well water quality is excellent, many unit operations and processes listed in Fig. 11.1 can be eliminated. The recommended simple water supply system consists of the well field (for raw groundwater supply), pumping (for moving water), chemical feeding (for adjusting pH adjustment), corrosion control (for water distribution system protection), secondary disinfection with chlorine (for maintaining residual chlorine in the water distribution system), clearwell (for providing chlorine contact time), water storage (for providing sufficient water flow and pressure), and the water distribution system (for delivering water to the consumers).

EXAMPLE 11.2 TREATMENT OF A GROUNDWATER SOURCE AFFECTED BY A CONTAMINATED DISPOSAL AREA

Figure 11.3 shows the water table, river, bedrock floor, and well field (wells #5, #6, and #7) from which the raw well water is pumped to an existing small water treatment plant (WTP) with minimum treatment for corrosion control and disinfection. The well field may supply enough raw water for the future consumption if all three wells are used. The existing contaminated disposal area is a municipal landfill site, which is up-gradient from wells #5, #6, and #7 by 320 ft (97.5 m), 430 ft (131.1 m), and 550 ft (167.6 m), respectively, measured from the closest edge of landfill to the closest edge of each well. Discuss the location of the municipal well field and related legal issues concerning a proposed plant expansion. Recommend an expanded/improved WTP system to satisfy the legal and technical requirements for the treatment of well water. Assume the raw well water quality is excellent and crystal clear, except that there are occasional coliform and *Giardia* cysts outbreaks from all three wells. In the State of New York, USA, the required minimum separation distance to protect water wells from landfill contamination is 300 ft (91.4 m).

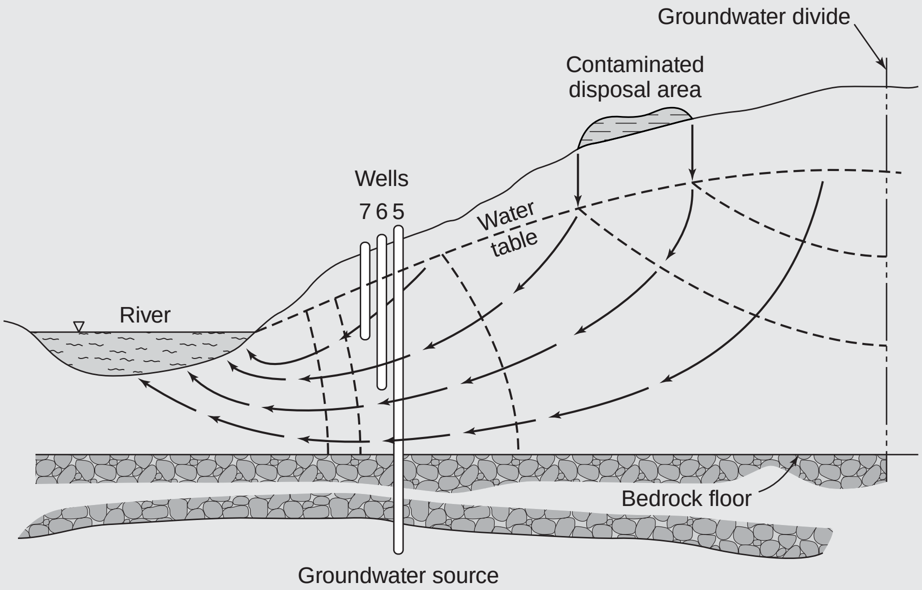


Figure 11.3 Location of water supply wells and pollution source (A modified figure).

Solution:

The contaminated area is located up-gradient of the wells at least 300 ft (91.4 m) away. The location of three existing wells is acceptable only in terms of the horizontal separation distances. In accordance with the Ten State standards, a well shall be located up-gradient of any potential or known source of contamination unless property boundaries, site topography location of structures, and accessibility make it require a different location. All three wells are located down-gradient of the landfill. Neither the existing landfill, nor the existing WTP can be relocated. The groundwater sources from wells #6 and #7 are above the bedrock floor, and directly under the influence of surface runoff water (GWUDI). Well #5 was drilled through the bedrock, and its water is expected to be better; however, well #5 alone cannot supply enough quantity of water. The community should dig wells #6 and #7 deeper through the bedrock and upgrade the existing simple chlorination WTP to a complete water filtration plant in accordance with the Surface Water Treatment Rule (SWTR) aiming at *Giardia* cysts removal. Since the raw well water turbidity is low, the flocculation and clarification (sedimentation) shown in Fig. 11.1 could be eliminated, resulting in a direct filtration WTP. The recommended small public water supply system includes pumping water from well field, chemical feeding (pH adjustment, primary disinfection using UV), sand filtration, corrosion control, secondary disinfection (chlorination), clearwell, high service booster pumping, pressure tank, water storage, and water distribution system. As stated earlier, bench-scale tests, pilot plant studies, or demonstrations may be required by the review authority to establish adequacy for *Giardia* cysts removal by direct filtration WTP. If the existing plant is to be upgraded to a complete conventional filtration plant (including rapid mixing, flocculation, and clarification by either sedimentation or dissolved air flotation (DAF)), no bench-scale test or pilot plant studies will be necessary. Furthermore, a simple water disinfection plant described in Example 11.1 could be sufficient, if (a) the well leak from well #5 can be prevented, (b) wells #6 and #7 can be extended to below the bedrock, and (c) the ruling of GWUDI could be lifted after demonstration of excellent raw water quality for an extended period of time. This example shows that selection of a water system for a community is not a simple engineering task. There are multiple solutions to a problem.

11.3 UNIT OPERATIONS AND UNIT PROCESSES

The concept of physical unit operations and chemical/biochemical unit processes has been developed from the analytical expression of common procedures in chemical and environmental engineering. Since a physical operation is frequently used in combination with a chemical or biochemical process, the term of unit operation has been used interchangeably with that of unit process. As has been suggested, any physical operation, or chemical/biochemical process, on whatever scale conducted, may be resolved into a coordinated series of what may be called unit actions. The number of these basic unit operations/processes is not very large, and their combination can form a feasible process system for particular application. The kind of thinking that emerged from this early pronouncement of principle has contributed greatly to the development of chemical engineering and, in the course of time, also to the advancement of water and wastewater treatment. Among its principal contributions are

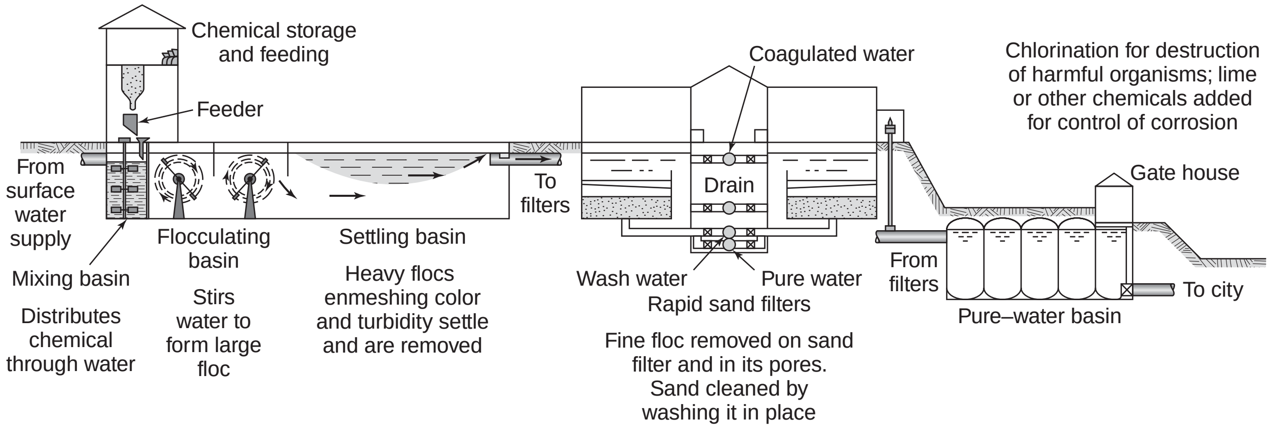


Figure 11.4 Typical surface water treatment plant—filtration plant including coagulation, settling, filtration, chlorination, corrosion control, and treated water storage (After Fair et al., 1971).

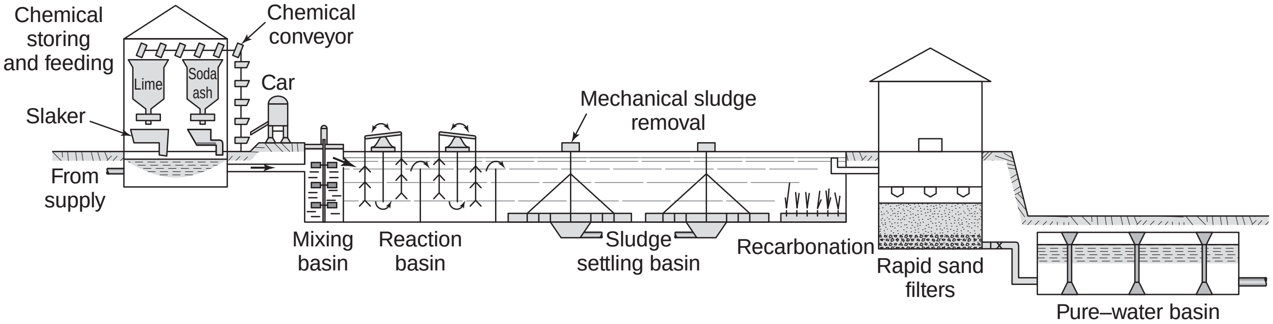


Figure 11.5 Typical groundwater treatment plant—softening plant including addition of softening chemicals, settling, recarbonation, filtration, chlorination, and treated water storage (After Fair et al., 1971).

(a) a better understanding of inherent operations/processes and capabilities in water, (b) the development of mathematical and simple physical models or analogs of treatment mechanisms and their use in identifying the basic components of treatment plant design, and (c) the coordination of effective treatment procedures to attain wanted plant performance and effluent quality.

A general flow diagram of a public WTP for treating groundwater and/or surface water (including GWUDI) is shown in Fig. 11.1. Figure 11.4 shows the vertical view of a typical surface water treatment plant assuming that sedimentation (instead of flotation) is used for clarification. It can be seen from Fig. 11.4 that the water flows through chemical feeders, rapid mixing basin, flocculating basin, settling basin, gravity rapid sand filters and pure-water basin (clearwell) all by gravity. As stated previously, normally a high-quality groundwater is treated by a direct filtration plant involving the use of pressure filters, as shown in Fig. 11.1. If the raw groundwater is classified as “hard water,” then a lime-soda ash softening plant including screening, pumping, chemical feeding (lime and soda ash), rapid mixing basin, flocculating basin (reaction basin), settling basin, gas chemical addition (recarbonation), gravity rapid sand filters, corrosion control, disinfection, and pure-water basin (clearwell) is needed. The lime-soda ash softening plant’s vertical view is shown in Fig. 11.5.

A typical view of a surface water treatment plant is shown in Fig. 11.6 and a general view of a groundwater treatment plant is shown in Fig. 11.7.

Figure 11.6 General view of a surface water treatment plant, http://www.epa.gov/waterinfrastructure/pdfs/brochure_si_sustainingournationswaters.pdf.

Figure 11.7 General view of a groundwater treatment plant, http://www.epa.gov/waterinfrastructure/pdfs/brochure_si_sustainingournationswaters.pdf.

Most water treatment processes bring about changes in the concentration of a specific substance by moving the substance either into or out of the water itself. This is called phase transfer. The principal phases are gas, liquid, and solid, but it is possible to identify other phases as well—the vapor phase, for instance—and to recognize specific states—the dissolved and colloidal states, for example.

The analysis of unit operations/processes is generally approached in one of the following ways: (a) through development of a mathematical model; (b) through construction or conceptualization of a simple physical model producing the wanted reactions; and (c) through batch laboratory tests and continuous pilot plant demonstration tests.

11.4 GAS TRANSFER

In gas transfer gases are released or desorbed from water or absorbed or dissolved by water through its exposure to the air or to other atmospheres under normal, increased, or reduced pressures.

Examples are (1) the addition of oxygen by spray or bubble aeration for removal of iron and manganese; (2) the removal of carbon dioxide, hydrogen sulfide, and volatile, odorous substances by spray or bubble aeration for odor and corrosion control; (3) the addition of ozone, chlorine, or chlorine dioxide in ozone towers or gas chlorinators for disinfection or odor destruction; (4) the removal of oxygen by evacuation in degasifier for corrosion control; and (5) the recarbonation of lime-softened water. The oxygenation of water for removal of iron and manganese is also a form of chemical precipitation.

11.5 ION TRANSFER

Ion transfer can be effected by chemical coagulation, chemical precipitation, ion exchange, adsorption, and biochemical reactions.

11.5.1 Chemical Coagulation

To bring about *chemical coagulation*, floc-forming chemicals are normally added to water for the purpose of enmeshing or combining with settleable or filterable but, more particularly, with otherwise nonsettleable or nonfilterable suspended and colloidal solids to form rapidly settling and readily filterable aggregates, or flocs. The coagulants themselves are soluble, but they are precipitated by transfer of their ions to substances in or added to the water. In water purification the floc formed is subsequently removed by sedimentation, flotation or filtration. The most common coagulants are aluminum and iron salts, which, upon solution, form trivalent aluminum and ferric ions. The precipitating ions are provided by naturally present alkalinity or, more rarely, by alkaline chemical additives such as soda ash.

Examples are (a) the addition of aluminum sulfate, polyaluminum chloride, ferric chloride, or magnesium carbonate to water to coagulate colloids and (b) the addition of polyelectrolytes to balance aggregation. Dosing, mixing, and flocculating or stirring are useful adjunct operations. By-products are chemical sludges and their included impurities. Figures 11.1 and 11.4 show typical water treatment trains in which chemical feeding, mixing, coagulation–flocculation, and subsequent clarification (DAF or sedimentation) and filtration play important roles for water purification.

EXAMPLE 11.3 CHEMICAL REACTIONS AND FLOCS FORMATION IN RAPID MIXING AND COAGULATION–FLOCCULATION BASINS

Write chemical equations for chemical coagulation using aluminum sulfate. Discuss how the raw water containing turbidity, color, microorganisms, suspended fine particles, other colloids, and so on is treated by conventional water treatment process using aluminum sulfate. Discuss the necessity of having a flocculation step and a sedimentation step in a conventional water treatment system.

Solution:

The following is an oversimplified chemical equation for chemical coagulation using aluminum sulfate in the presence of sufficient alkalinity (such as calcium bicarbonate):

$$Al_2(SO_4)_3 \bullet 14.3\ H_2O + 3Ca(HCO_3)_2 \rightarrow 2Al(OH)_3 + 3CaSO_4 + 6CO_2 + 14.3\ H_2O \quad \textbf{(11.1)}$$

Aluminum sulfate is an excellent coagulant, which is originally soluble in water. After it reacts with the natural alkalinity $Ca(HCO_3)_2$ in a rapid mixing basin, insoluble aluminum hydroxide pin flocs are produced. The rapid mix effluent containing raw water and aluminum hydroxide pin flocs flows into a coagulation–flocculation basin (Fig. 11.4) where water and pin flocs are gently mixed. The insoluble pin flocs attract the turbidity, color-causing substances, microorganisms, and other colloids and suspended solids forming bigger flocs. The larger the flocs, the faster the flocs can settle in the subsequent settling tank. After most of the impurities (turbidity, color, undesirable colloids, suspended particles, microorganisms, etc.) have settled together with the chemical flocs (aluminum hydroxide), the settler effluent goes to a gravity filter by which the residual fine flocs are removed by filtration. The filter effluent is disinfected and chemically treated becoming the finished potable water in a clearwell. The flocculation step (gentle mixing) is needed for pin flocs to contact each other for charge neutralization, destabilization, and agglomeration, so the pin flocs will grow bigger and be destabilized. The big chemical flocs need to be settled in a sedimentation clarifier for solid–water separation.

11.5.2 Chemical Precipitation

In *chemical precipitation*, dissolved substances are thrown out of solution. The added chemicals are soluble and the ions released react with ions in the water to form precipitates.

Examples are (1) flocculation of iron by the addition of lime to iron-containing water, the reaction being carried to completion by dissolved oxygen; (2) precipitation of iron and manganese from water by aeration, the reaction being one of oxidation by dissolved oxygen; (3) softening of water by the addition of lime to precipitate carbonate hardness and of soda ash to precipitate noncarbonate hardness; and (4) removal of fluoride ions from water by the addition of tricalcium phosphate, or by their precipitation along with magnesium ions in water softening. Here also, dosing, mixing, and flocculating or stirring are needed adjunct operations, and the by-products are chemical sludges and their included impurities.

11.5.3 Ion Exchange

Traditionally the ion exchange process is mainly used by municipalities to remove hardness and nitrate from groundwater, and is used by hospitals and laboratories to remove dissolved minerals from water, in turn, to produce pure water. In ion exchange operations, specific ions in water are exchanged for complementary ions that are part of the complex of a solid exchange medium.

Examples are (a) the exchange of calcium and magnesium ions for sodium ions by passage of water through a bed of sodium zeolite, which is regenerated by brine (base or cation exchange); (b) the exchange of nitrate by synthetic organic anion exchangers, which are regenerated by sodium carbonate; and (c) the exchange of *natural organic matter*s (NOMs) by anion exchangers as a pretreatment to membrane filtration process units to prevent membrane fouling. The precipitation of iron and manganese on manganese zeolite and the regeneration of the zeolite with potassium permanganate is, in a sense, an example of surface or contact precipitation rather than ion exchange. Ion exchange can also be used to remove arsenic, chromium, excess fluoride, nitrates, radium, cadmium, silver, and uranium.

The ion exchange process unit can be a fixed bed or a fluidized bed in a pressure vessel. For a fluidized ion exchange bed operation, the influent enters the bed from the bottom, and

EXAMPLE 11.4 REMOVAL OF HARDNESS FROM GROUNDWATER BY LIME-SODA ASH SOFTENING PROCESS

Lime is CaO and soda ash is Na_2CO_3. Soluble calcium ions and magnesium ions in groundwater contribute to the hardness. A lime-soda ash softening process removes hardness and some other soluble metals from water by chemical precipitation. In the presence of bicarbonate ions (HCO_3^-), the soluble calcium ions originally in groundwater, the added lime, and the bicarbonate ions react together forming insoluble calcium carbonate ($CaCO_3$), so the calcium hardness is reduced. Magnesium hardness is reduced by lime and bicarbonate at high pH around 10.8, coprecipitating insoluble magnesium hydroxide [$Mg(OH)_2$] and insoluble calcium carbonate ($CaCO_3$). Soda ash and lime together remove permanent calcium hardness ($CaSO_4$) and magnesium hardness ($MgSO_4$) by chemical precipitation, forming insoluble calcium carbonate and magnesium hydroxide. Explain the difference between chemical coagulation and chemical precipitation. Propose a lime-soda ash softening process system to treat a groundwater containing high concentration of hardness (300 mg/L as $CaCO_3$).

Solution:

Using Fig. 11.1 as a guide, the following water treatment train is recommended: well field (groundwater source), chemical feeding (lime addition and soda ash addition), rapid mixing, primary disinfection (chlorination), slow mixing (flocculation for reaction), sedimentation, gas addition (carbon dioxide gas for recarbonation), gravity filtration (rapid sand filtration), corrosion control, secondary disinfection (chlorination), clearwell, and water storage and distribution. The vertical view of the lime-soda ash softening process system is partially shown in Fig. 11.5.

the ion exchange effluent exits from the top. Recently a dual-stage MIEX process system has been developed for NOM removal prior to membrane filtration and/or GAC adsorption. The dual-stage MIEX process system consists of the following process steps: (a) raw water and ion exchange resins are mixed in a big mixing basin for exchange reactions; (b) the mixing basin effluent settles in a sedimentation basin for separation of the ion exchange resins from the water; (c) the supernatant of the sedimentation basin is the treated water, which flows to a subsequent unit process for further treatment; (d) the settled ion exchange resins are the spent ion exchange resins, which are pumped to a smaller mixing vessel to which brine is added for resin regeneration and make-up resin addition, if necessary; (e) the regenerated ion exchange resins are pumped to another small mixing vessel ready for recycling to the big mixing basin for reuse. Wang, Kurylko and Wang (1994) invented the sequencing batch ion exchange (SBR-IX), which can also be used for NOM removal.

11.5.4 Adsorption

In adsorption, interfacial forces remove ions and molecules (*adsorbates*) from solution and concentrate them at the interface of *adsorbents*.

Examples are the adsorption of odor- and taste-producing ions and molecules on beds of GAC or on PAC suspended in water and removed by sedimentation or filtration. GAC beds are regenerated by leaching. Spent PAC

EXAMPLE 11.5 TECHNICAL FEASIBILITY OF ACTIVATED CARBON ADSORPTION FOR ADSORBING ORGANICS AND INORGANICS

Figure 11.8 shows the relationship among the adsorbent (GAC or PAC), adsorbate (impurities to be adsorbed, such as volatile organic compounds (VOCs)), and solvent (such as a water containing the adsorbate). Adsorbate can be attached on the adsorbent's outside surface or its inside macro-channels and micro-channels. What kind of organic and inorganic impurities can be readily adsorbed by a GAC bed or a PAC process? Hint: Search the internet for documenting your answer.

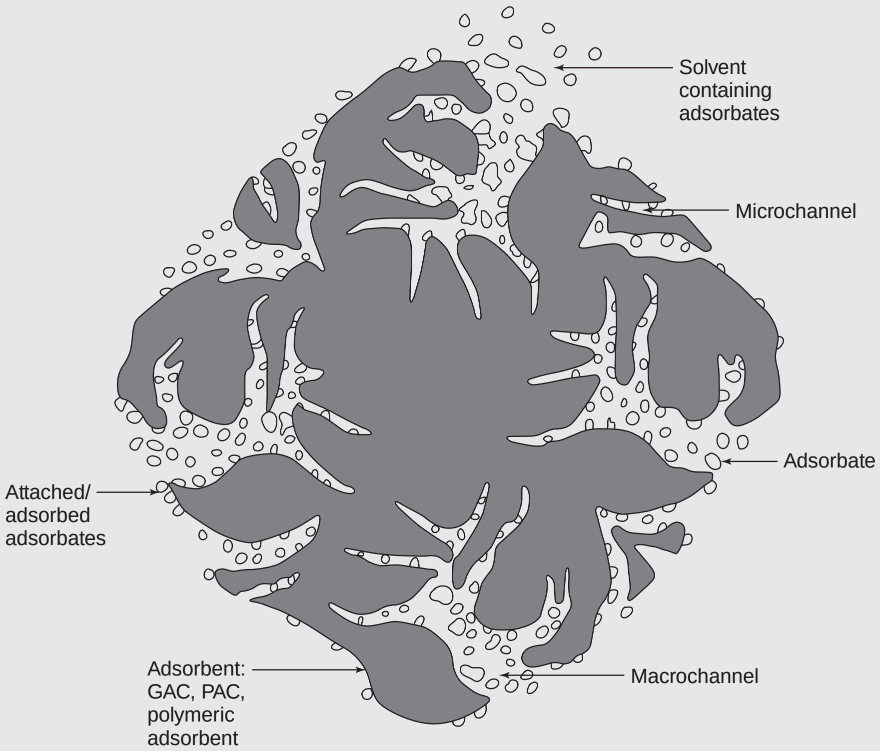


Figure 11.8 Adsorbent (GAC, PAC, polymeric adsorbent), solvent, and adsorbate in an adsorption system.

Solution:

GAC or PAC can readily remove the following organic impurities from surface water or groundwater:

1. Aromatic solvents (benzene, toluene, nitrobenzenes, etc.)
2. Chlorinated aromatics (chlorobenzenes, chloronapthalene, PCBs, etc.)

3. Phenol and chlorophenols
4. Pesticides and herbicides (aldrin, DDT, chlordane, heptachlor, etc.)
5. Polynuclear aromatics (acenaphthene, benzopyrenes, etc.)
6. Chlorinated nonaromatics (chloroalkyl ethers, carbon tetrachloride, etc.)
7. High molecular weight hydrocarbons (gasoline, amines, humics, dyes, etc.)

The following organics are poorly adsorbed by GAC or PAC: (a) alcohols; (b) sugars and starches; (c) low molecular weight ketones, acids, and aldehydes; (d) very high molecular weight or colloidal organics; and (e) low molecular weight aliphatics.

GAC and PAC can effectively adsorb certain inorganic substances, such as mercury and lead.

is normally wasted. By-products are leaching fluids or PAC with contained impurities.

11.6 SOLUTE STABILIZATION

Water is stabilized by a variety of operations in which objectionable solutes are converted into unobjectionable forms without removal.

Examples are (a) the chlorination of water for the oxidation of hydrogen sulfide into sulfate; (b) the liming (addition of quick lime, CaO, or slaked hydrated lime, $Ca(OH)_2$) of water or passage of water through chips of marble (crystalline limestone), limestone (calcium carbonate), or dolomite (calcium and magnesium carbonate) for the conversion of carbon dioxide in excess of equilibrium requirements into soluble bicarbonate; (c) the recarbonation of water softened by excess lime treatment to convert excess lime into bicarbonate; (d) the superchlorination of water or addition of chlorine dioxide for the oxidation of odor-producing substances; (e) the removal of excess chlorine by reducing agents such as sulfur dioxide; (f) the addition to water of complex phosphates to keep iron in solution; and (g) the addition to water of lime, complex phosphates, or sodium silicate to protect metallic surfaces by deposit coatings or otherwise to reduce the corrosive action of water.

11.7 SOLIDS TRANSFER

In order of decreasing size, solids are removed from water by straining, sedimentation, flotation, and filtration.

11.7.1 Straining

Screens and racks strain out floating and suspended solids larger in size than their openings. The rackings and screenings are removed for treatment and disposal. Shredding devices combined with racks and screens convert coarse rackings and screenings into fine solids, which are normally returned to the water to be removed later conjointly with other suspended solids by sedimentation.

Example is the removal from water of leaves, sticks, and other debris by racks and screens and the straining out of algae by microscreens. By-products are the rackings and screenings removed for disposal by burial. In-place comminution of solids by cutting screens is an indirect method of straining. Filtration is, in part, a screening or straining operation.

11.7.2 Sedimentation

To permit the removal of solids from water by sedimentation, the carrying and scouring powers of flowing water, which are functions of its velocity, are reduced until suspended particles settle by gravity to the bottom of holding tanks or basins and are not resuspended by scour.

Examples are (a) the removal of sand and heavy silt from water in settling basins and (b) the removal from water of nonsettleable substances rendered settleable by coagulation or precipitation. The by-products, known as sediment, grit, or sludge, must be removed from the sedimentation devices for treatment and disposal. Figures 11.1 and 11.4 illustrate how and when a conventional continuous sedimentation process unit can be applied to a WTP for separation of flocculated chemical flocs or precipitates. Wang, Kurylko and Wang (1994) have developed sequencing batch sedimentation reactor (SBR-SED) system for potable water treatment, which is very similar to the sequencing batch reactor (SBR), except that sequencing batch sedimentation reactor is a physicochemical process.

An upflow clarifier combines chemical mixing, coagulation–flocculation, and settling in a single clarifier. The flocculation portion of the clarifier is designed to provide the necessary mixing for good floc formation, while the sedimentation portion acts as a true upflow clarification unit with the surface overflow rate controlling suspended solids removal. Sludge or solids blanket is not involved.

A solids-contact clarifier (upflow contact clarifier or solids-contact basin) is another clarification process in which coagulation–flocculation and particle separation occur. Coagulated water from rapid mixing compartment passes upward through a solids blanket, allowing flocculation and particle separation to take place in a single process step. The solids blanket is typically 6–10 ft (2–3 m) below the water surface, and clarified water is collected in launder troughs along the top of the clarifier. Solids (waste sludge) are continually withdrawn from the solids blanket to prevent undesired accumulation.

Microsand-assisted sedimentation is another innovative sedimentation clarification process which relies on improved settling through the addition of microsand and a chemical coagulant to improve coagulation, flocculation, and clarification. In the presence of microsand as a settling aid, the chemical flocs settle faster in comparison with the chemical flocs without microsand. The microsand is separated and recycled through the water system numerous times.

11.7.3 Flotation

In dissolved air flotation (DAF) operations, the transporting power of flowing water is reduced by quiescence or the

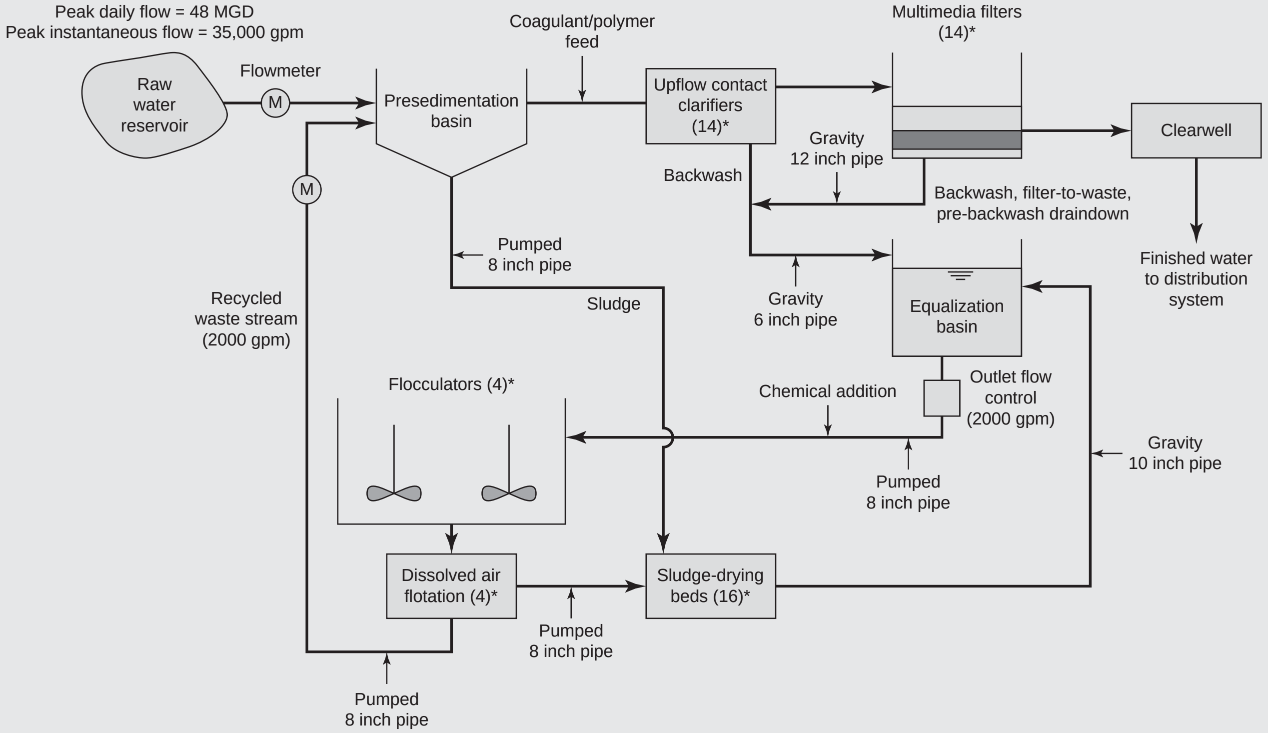
EXAMPLE 11.6 APPLICATION OF UPFLOW CONTACT CLARIFIER FOR WATER TREATMENT AND FLOTATION THICKENER FOR SLUDGE THICKENING

A conventional WTP with a peak design flow of 48 MGD (182 MLD) treats a surface water containing excessive color, turbidity, and coliforms. Upflow contact clarifiers are used for combined chemical mixing, coagulation–flocculation, and sedimentation clarification. The clarifier's settled sludge and the filter's backwash wastewater are treated and only the water portion is recycled to the presedimentation basin. The separated waste sludge is properly disposed of.

Prepare a flow diagram of the water treatment system. Neglect the waste treatment train for the time being, because it will be presented and discussed in Example 11.9.

Solution:

Using Fig. 11.1 as a guide, the surface water treatment train involving the use of an upflow contact clarifier should include presedimentation, screening, pumping, chemical feeding, primary disinfection, upflow contact clarifier (a combination of rapid mixing, coagulation–flocculation, and sedimentation), gravity multimedia granular filtration, corrosion control, secondary disinfection, clearwell, and water storage and distribution. Figure 11.9 shows both the water treatment train and waste treatment train of an upflow contact clarification and filtration plant in which the upflow contact clarifier is used for water treatment, while DAF thickener is used for sludge thickening. The waste treatment train portion will be discussed in Example 11.9.



* Number in parenthesis indicates number of treatment units.

Figure 11.9 Flow diagram of a 48 MGD (181.7 MLD) upflow contact clarification-filtration water treatment plant using equalization, dissolved air flotation, and sludge drying beds for waste treatment (*Source*: US EPA).

EXAMPLE 11.7 APPLICATION OF MICROSAND-ASSISTED SEDIMENTATION IN POTABLE WATER TREATMENT

Figure 11.10 illustrates the flow diagram of a typical water treatment system involving the use of microsand-assisted sedimentation. Based on the description of the microsand-assisted sedimentation in Section 11.7.2, discuss the applications, advantages, and disadvantages of the microsand-assisted sedimentation.

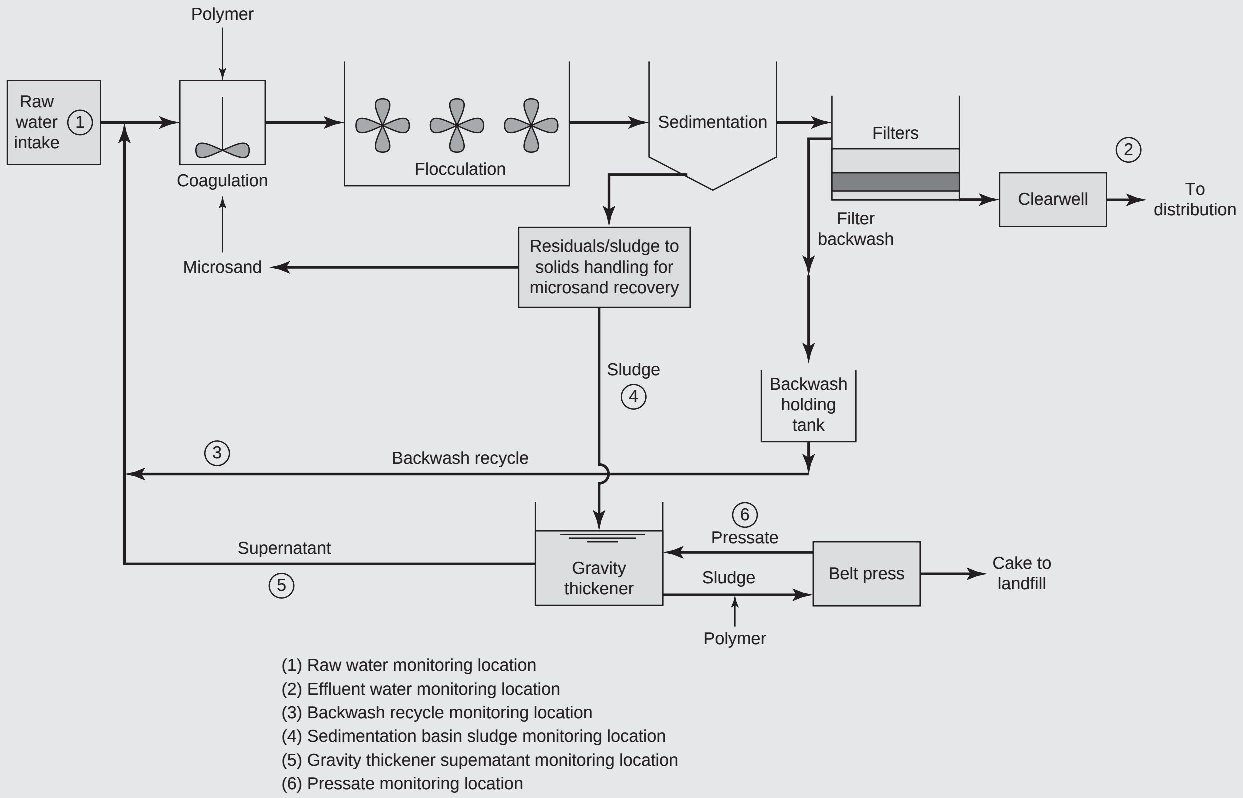


Figure 11.10 Typical flow diagram of a microsand-assisted sedimentation process for water treatment (A modified figure).

Solution:

The process may have application in water treatment facilities that need clarification but that do not have enough space for conventional sedimentation or that need rapid startup clarification ability for variable source water qualities. The advantages of this process are (a) small footprint, (b) good clarification performance, (c) quick process startup time, and (d) possible capital cost saving depending on the flow. The disadvantages include heavy dependence on mechanical equipment and short processing time, dependence upon power, and possible requirement of higher dosage of coagulant.

suspending power of water is overcome by quiescence and sometimes by the addition of *flotation agents*. Substances naturally lighter than water or rendered lighter than water by flotation agents rise to the water surface and are skimmed off. Flotation agents include fine air bubbles and chemical compounds that, singly or in combination, are often hydrophobic wetting and foaming agents.

Examples are (1) the release of fine bubbles of air into water either by diffusion of compressed air or by desorbing air dissolved in the water through reduction of the pressure of the overlying atmosphere, the fine air bubbles attaching themselves to suspended particles, imparting buoyancy to them and lifting them to the surface; and (2) the addition to water of flotation agents that attach themselves to suspended solids or attach suspended solids to bubbles of air and lift the particles to the surface. By-product skimmings or foam must be removed from the top of the flotation devices and disposed of. Examples of flotation agents are the metallic salts of aluminum and iron and the anionic, neutral, or cationic polyelectrolytes. Figure 11.1 shows either DAF clarification or sedimentation clarification can be used for solid–water separation in a potable water treatment train. DAF can be used in potable water treatment train for water clarification (Fig. 11.1) or in waste treatment train for sludge thickening (Fig. 11.9).

EXAMPLE 11.8 APPLICATION OF DISSOLVED AIR FLOTATION IN POTABLE WATER TREATMENT PLANT FOR WATER CLARIFICATION

In a typical water treatment system shown in Fig. 11.1, DAF clarification may replace sedimentation clarification for separation of chemical flocs coming from the flocculation effluent. Figure 11.11 illustrates the flow diagram of a potable water DAF filtration plant. Discuss the difference between a DAF filtration plant and a conventional water filtration plant. What are the first two DAF filtration plants built in North America for potable water treatment?

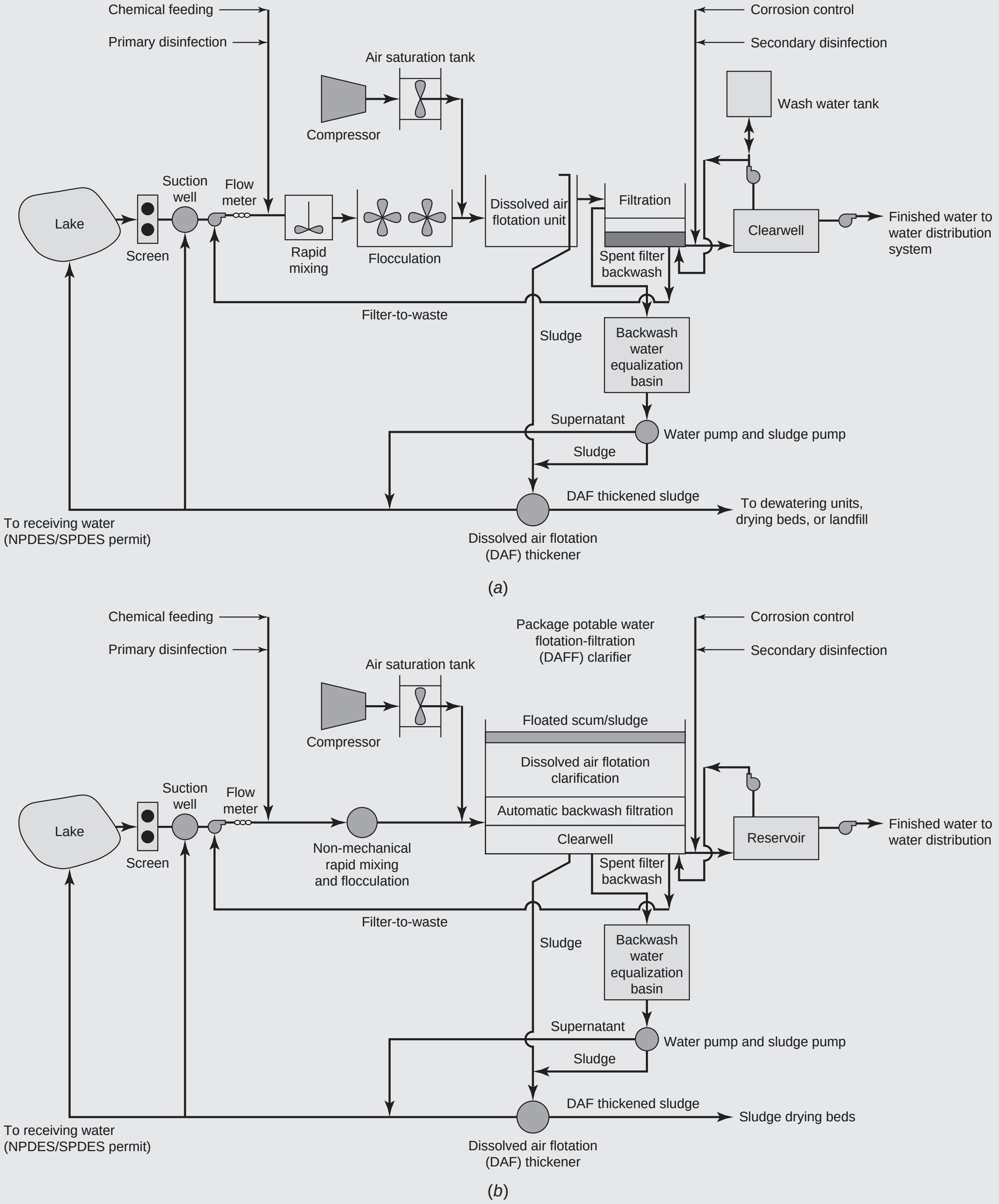


Figure 11.11 (a) Typical flow diagram of a DAF filtration plant using individual unit process units (A modified figure). (b) Typical flow diagram of a DAF filtration plant using package DAFF units (A modified figure).

Solution:

Figures 11.1, 11.11a, and 11.11b are examined for the purpose of comparison and discussion. A complete DAF filtration plant may include screening, pumping, chemical feeding, primary disinfection, rapid mixing, coagulation–flocculation, DAF clarification, gravity filtration, corrosion control, secondary disinfection, clearwell, water storage, and distribution. The flow diagram of a DAF filtration plant is very much similar to that of a conventional WTP, except that the plant adopts DAF for clarification, while conventional WTP adopts sedimentation for clarification. The first DAF filtration plant in North America for potable water treatment was Lenox Water Treatment Plant, Massachusetts, which was built in 1981 to serve 6500 people. One DAF with a capacity of 1 MGD (3.785 MLD) was used in Lenox, Massachusetts, USA, for potable water treatment and a much smaller DAF was used for waste sludge thickening. The second DAF filtration plant was Pittsfield Water Treatment Plant, Massachusetts, USA, which was built in 1986, with a peak capacity of 37.5 MGD (142 MLD), serving the City of Pittsfield and General Electric facilities. Both Lenox WTP and Pittsfield WTP are package flotation-filtration plants or DAFF plants in which rapid mixing, flocculation, DAF, and sand filtration are combined in one single unit, as shown in Fig. 11.11b. The Millwood Water Treatment Plant, New York, USA, was the first DAF filtration plant built in 1993 using individual unit process units. The Millwood WTP has a capacity of 7.5 MGD (28.4 MLD) and includes the individual processes of rapid mixing, three-stage flocculation, DAF clarification, ozone disinfection, and dual-media filtration. Greenville Water Treatment Plant, Greenville, SC, USA, is another flotation-filtration plant using individual unit processes. Greenville WTP has a peak capacity of 75 MGD (284 MLD) and has been in service since 2000.

EXAMPLE 11.9 APPLICATION OF DISSOLVED AIR FLOTATION IN POTABLE WATER TREATMENT PLANT FOR SLUDGE THICKENING

DAF is another form of solids separation and is most commonly used in two applications in water utilities: (a) potable water treatment as a clarification step prior to filtration and (b) waste treatment for sludge thickening.

The bottom portion of Fig. 11.9 or Fig. 11.11 illustrates a waste treatment train consisting of (a) equalization basin, (b) chemical addition and flocculation, (c) DAF for sludge thickening, and (d) sludge drying beds. Discuss the advantages and disadvantages of using DAF for sludge thickening in a WTP.

Solution:

DAF has several advantages over conventional gravity thickener for sludge thickening in a potable WTP: (a) more compact in comparison with conventional gravity thickener; (b) shorter startup time; (c) lower chemical dose when a coagulant is needed; (d) shorter flocculation time; and (e) thicker DAF sludge in comparison with the settled sludge from a conventional gravity thickener. The main disadvantage of DAF compared to gravity thickener is that DAF requires more complex equipment, particularly the air saturation and recycle control equipment. A higher level of skill is needed for operators to operate and maintain the DAF than is needed for equipment associated with gravity thickener. If DAF can provide adequate treatment without pretreatment, then DAF becomes a cost-effective option to treat recycle streams in a potable water plant. DAF for waste alum sludge thickening was first practiced at the Lenox Water Treatment Plant, Massachusetts, USA, in 1981.

11.7.4 Filtration

Conceptually, filtration combines straining, sedimentation, and interfacial contact to transfer suspended solids or flocs onto grains of sand, coal, or other granular materials from which the solids or flocs must generally be removed later.

Examples are (a) bank filtration—the filtration of water through naturally permeable formations in groundwater recharge and of wastewater treated effluents through natural soils in surface or subsurface irrigation; (b) slow sand filtration—the slow filtration of water through beds of sand (either natural deposits or man-made sand beds) that are cleaned by scraping or are allowed to rest and reaerate between dosings; (c) rapid sand filtration—the rapid filtration of water through beds of sand, coal, or other granules singly or in combinations of stratified layers, the accumulated impurities being scoured from the filter by water alone, air followed by water, or water concurrently with mechanical rakes. The wash water is a by-product that may be reclaimed or may have to be treated before discharge into a drainage system, into a receiving body of water, or onto land; (d) membrane filtration—filtration through membranes; (e) cartridge filtration—self-backwashing filters in cartridges; and (f) diatomaceous earth (DE) filtration—filtration of water through relatively thin layers of DE that is normally discarded after each filter run.

Figure 11.12 illustrates the four types of pressure-driven membrane filtration processes based on the pore sizes of membranes: (a) microfiltration; (b) ultrafiltration; (c) nanofiltration; and (d) reverse osmosis, which are introduced below:

Microfiltration (*MF*): It removes ions, salts, other dissolved solids, and nonvolatile organics. In potable water treatment, microfiltration (MF) is operated at low transmembrane pressure (1–30 psi or 7–208 kPa) and is used for separation of bacteria, large microorganisms, fine solids, large flocs, large colloids, and so

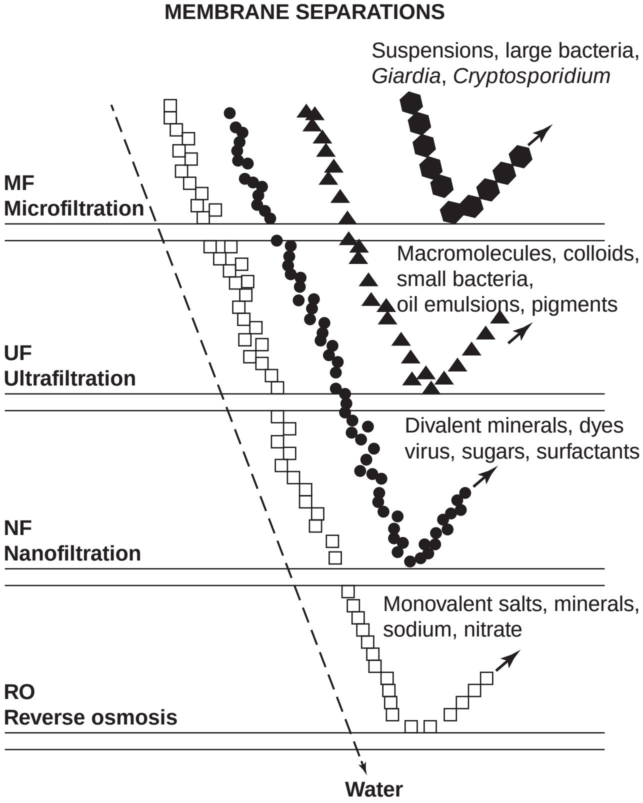


Figure 11.12 Membrane filtration process classifications (A modified figure).

on as well as other bigger substances. MF membrane has a large pore size range (from 0.1 to 10 microns).

Ultrafiltration (*UF*): It removes ions, salts, other dissolved solids, and nonvolatile organics. In potable water treatment, ultrafiltration (UF) is operated at medium transmembrane pressure (30–75 psi or 208–517 kPa) and is used for separation of colloids, flocs, turbidity, suspended solids, large microorganisms, and so on as well as other bigger substances. UF membrane has a medium pore size range (from 0.005 to 0.09 microns).

Nanofiltration (*NF*): It removes ions, salts, other dissolved solids, and nonvolatile organics. In potable water treatment, nanofiltration (NF) is operated at high transmembrane pressure (75–149 psi or 517–1,027 kPa) and mainly used for separation of divalence and trivalence ions, hardness, DBP, NOM, *Giardia* cysts, *Cryptosporidium*, color, surfactants, and so on as well as bigger substances. NF membrane has very small pore size range (from 0.0005 to 0.005 microns).

Reverse osmosis (*RO*): It removes ions, salts, other dissolved solids, and nonvolatile organics. In potable water treatment, reverse osmosis (RO) has the highest transmembrane pressure (150–500 psi or 1,034–3,448 kPa) and is used for desalination or separation of monovalence ions, such as salts, sodium, nitrate, nitrite, chloride, and so on as well as bigger substances. RO membrane has the smallest pore size range (from 0.0001 to 0.001 microns) in comparison with MF, UF, and NF.

Recent potable water filtration developments include continuous backwash sand filtration, automatic backwash filtration (ABF), GAC filtration and biological GAC filtration. Various filtration applications and their roles in water treatment systems are further discussed in Sections 11.8, 11.12 and 11,13.

11.8 NUTRIENT OR MOLECULAR TRANSFER AND INTERFACIAL CONTACT

In the natural purification of water, saprobic organisms convert complex, principally organic, substances into living cell material and simpler or more stable matter, including gases of decomposition; and photo-synthetic organisms convert simple, principally inorganic, substances into cell material with the aid of sunlight, with oxygen being a by-product of their activity.

Examples are (1) the essentially aerobic destruction or stabilization of suspended and dissolved organic matter by saprobic organisms (saprobic organisms feed on dead organic matter; plant-like members are called saprophytes; animal-like members are called saprozoa) multiplying in polluted receiving bodies of water; (2) the essentially anaerobic destruction or stabilization (also by saprobic organisms) of organic substances deposited on the bottom of polluted receiving bodies of water in appreciable thickness; and (3) the production of algae and large aquatic vegetation in the presence of simple plant nutrients and sunlight. These three actions may occur concurrently in polluted waters, and they do so quite generally in stabilization ponds for wastewaters. Bottom deposits and algal or other blooms are by-products.

Interfacial contact is provided by biologically active flocs or slimes of living organisms that have been generated under aerobic conditions. Putrescible, principally finely divided and dissolved, nutrients are transferred to the flocs, film, slimes, or cell interfaces. Some of the nutrients promote the growth of living cells; some provide energy to the living system. Soluble and stable end products of biological activity are returned to the water.

Example is the filtration of water through slow sand filters (only in part). By-products are the biological films unloaded intermittently. Secondary settling, flotation, and membrane filtration are common adjunct operations. Figures 11.13a and 11.13b illustrate the simplest potable water slow sand filtration plant. An innovative biological GAC filtration has been developed for potable water treatment. Biological GAC filtration is similar to slow sand filtration except that GAC media is used instead of sand media.

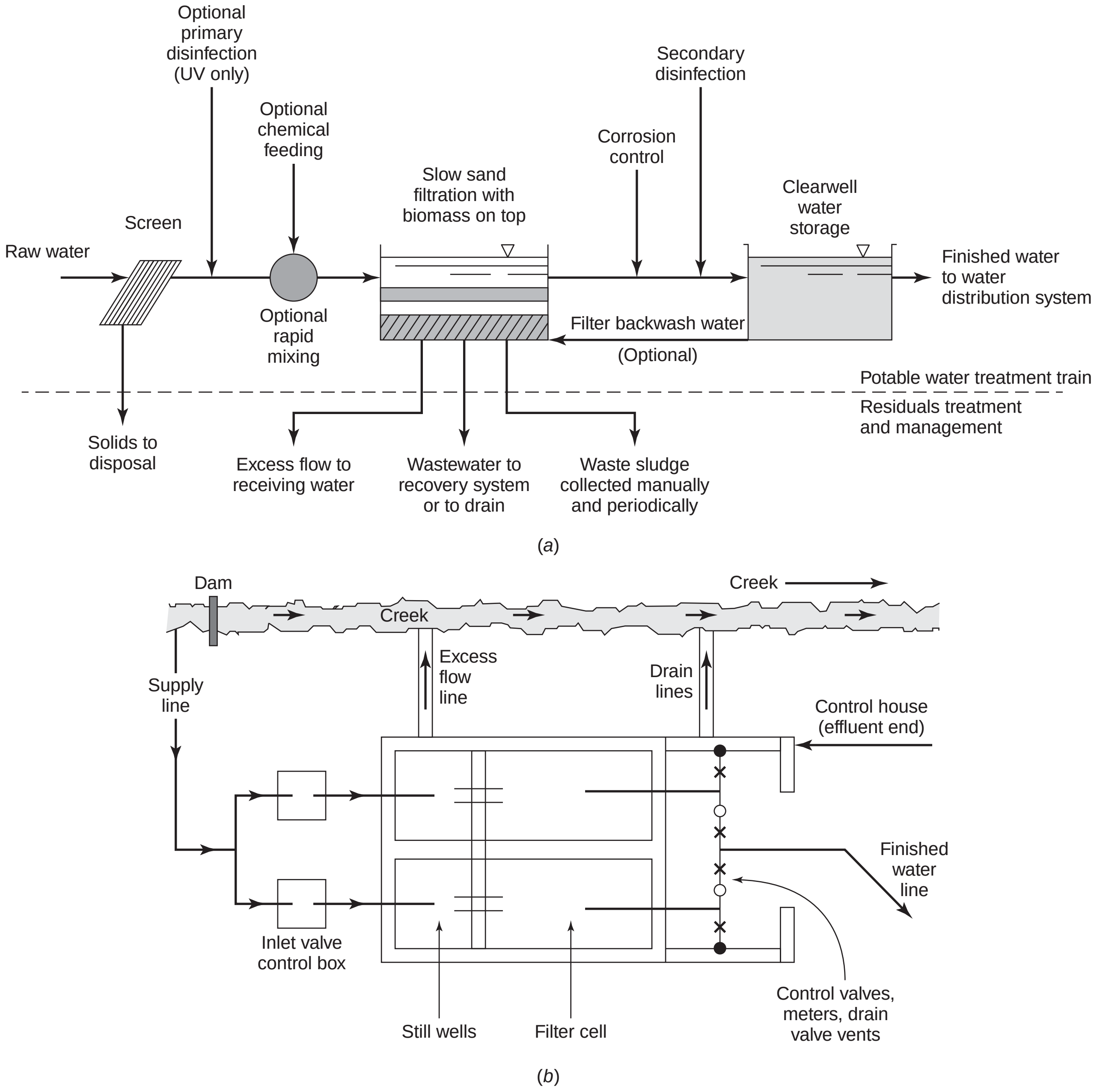


Figure 11.13 (a) Typical flow diagram of a slow sand filtration plant for surface water treatment. (b) Typical layout of a slow sand filtration plant for surface water treatment (*Source*: US EPA).

11.9 DISINFECTION

The recently promulgated SWTR in the United States requires water treatment systems to inactivate 99% of *Cryptosporidium*, 99.9% of *Giardia* cysts and 99.99% of enteric viruses. Disinfection systems alone, or in conjunction with filtration systems, can meet these requirements. At the same time, regulations regarding *DBPs* in the finished water must be met.

Primary disinfectants are those used for the inactivation of *Giardia* cysts, C*ryptosporidium*, viruses, and bacterial contaminants, while secondary disinfectants suppress biological regeneration in the distribution system. Common primary disinfectants are *chlorine*, *chlorine dioxide*, *ozone*, and *UV radiation*. Most are suitable for groundwater and surface water. Secondary disinfectants include chlorine, chlorine dioxide, and *chloramines*.

Disinfection effectiveness is measured in terms of the residual concentration and length of contact time necessary to achieve the desired inactivations. Four chemical disinfectants listed in descending order of their effectiveness are ozone, chlorine dioxide, chlorine, and chloramines. The effectiveness of a particular disinfectant is also influenced by water

EXAMPLE 11.10 APPLICATIONS OF ULTRAVIOLET RADIATION IN WATER TREATMENT

Discuss the applications of UV radiation in potable water treatment. Propose the flow diagram of a potable WTP for treating a groundwater (GWUDI) containing over 3 *Cryptosporidium* oocyst/L, assuming all other water quality parameters meet the Federal and the State drinking water standards.

Solution:

The DBPs of greatest current concern are the trihalomethanes and other halogenated organic compounds. The use of UV for secondary disinfection may reduce the chance of producing DBPs, but cannot provide any chlorine residue for water quality protection in the water distribution system. Therefore, UV is not suitable to be used for secondary disinfection alone. Some communities do use combined UV and chlorine for secondary disinfection, which is technical feasible, but the cost is high because UV is a dechlorination process resulting in higher chlorine dosage requirement. UV light is highly recommended for primary disinfection when the raw water (either groundwater or surface water) is contaminated by *Cryptosporidium* oocyst. Using Fig. 11.1 as a guide, the recommended flow diagram of a WTP for treating GWUDI includes well field, pumping, UV for primary disinfection (mainly for inactivation of *Cryptosporidium*), direct pressure sand filtration, corrosion control, secondary disinfection using chlorine, clearwell, water storage, and water distribution.

Alternatively, combined ozone–UV (instead of UV alone) can be applied to the above flow diagram for primary disinfection. The combination of ozonation and UV is called "*advanced oxidation process* (*AOP*)," which is highly efficient for inactivation of *Cryptosporidium*, *Giardia*, and oxidation of iron and manganese. Other AOPs include (a) the combination of hydrogen peroxide oxidation and UV; (b) the combination of calcium peroxide oxidation and UV; and (c) the combination of magnesium peroxide oxidation and UV.

quality, temperature, and pH. Lower disinfectant dosages may be used when (1) there is filtration or oxidation prior to the disinfection step and (2) the water temperature is high.

The dosage of chlorine required for effective disinfection is reduced as the pH of the water is reduced. A qualitative summary of the advantages and disadvantages of the five disinfectants is provided in Table 18.3.

One of the most important considerations in assessing disinfectants is balancing inactivation or biocidal effectiveness with by-product production. The by-products of greatest current concern are trihalomethanes and other halogenated organic compounds; chlorine has the greatest potential for generating harmful by-products. The amount of these by-products produced by chlorine is affected by (a) chlorine dosage, (b) types and concentrations of organic material in the influent, (c) influent temperature, (d) influent pH, (f) contact time for free chlorine, and (g) nature of residual (free chlorine vs. combined chlorine) and presence of bromide ion.

11.10 MISCELLANEOUS OPERATIONS/PROCESSES

Miscellaneous operations/processes include the following:

- *Disinfection* of water by heat, light, or chemicals that kill living, potentially infectious organisms.
- *Copper sulfating* of water to control algae.
- *Fluoridation* of water. The objective of this process is to attain a concentration of fluoride in the water, which imparts to the population the maximum degree of resistance to tooth decay.
- *Desalination* of water by *thermal* processes, such as evaporation and freezing, and by *diffusion* or *dialytic* processes, such as *reverse osmosis*.

11.11 COORDINATION OF UNIT OPERATIONS/PROCESSES

The unit operations of water purification are introduced into treatment works in many different combinations and sequences to meet, in water purification works, existing conditions of raw water quality and requirements of pure-water quality. The selection and elaboration of the unit operations to be employed constitute the *process* or *systems design* of the treatment works.

In order to direct attention to feasible combinations of water treatment operations, the attributes of water affected by some conventional unit operations and processes of treatment are identified in Table 11.1. There the relative degree of effectiveness of each unit operation is indicated by the number of plus signs (+) up to a limit of four; adverse effects are shown by minus signs (–) also to degree; and indirect effects are shown by parentheses placed around the signs. Limitations and other factors are explained in footnotes. It is noted that the performance (i.e., water treatment efficiency) of a coagulation and DAF combination is slightly better than that of coagulation and sedimentation combination. The performance of a lime-soda softening and DAF combination is also slightly better than that of lime-soda softening and sedimentation combination.

Typical combined operations in water treatment are illustrated in Fig. 11.4 for surface water treatment and Fig. 11.5 for groundwater treatment plant. Figure 11.14 shows a WTP

Table 11.1 Common attributes of water affected by conventional unit operations and water treatment processes

Attribute (*a*)	Aeration (*b*)	Coagulation and sedimentation (*c*)	Lime-soda softening and sedimentation (*d*)	Slow sand filtration without (*c*) (*e*)	Rapid sand filtration preceded by (*c*) (*f*)	Disinfection (chlorination) (*g*)
Bacteria	0	+ +	(+ + +)[1,2]	+ + + +	+ + + +	+ + + +
Color	0	+ + +	0	+ +	+ + + +	0
Turbidity	0	+ + +	(+ +)[2]	+ + + +[3]	+ + + +	0
Odor and taste	+ +[4]	(+)	(+ +)[2]	+ +	(+ +)	+ + + +[5] – –[6]
Hardness	+	(– –)[7]	+ + + +[11]	0	(– –)[7]	0
Corrosiveness	+ + +[8] – – –[9]	(– –)[10]		0	(– –)[10]	0
Iron and manganese	+ + +	+[11]	(+ +)	+ + + + [12]	+ + + + [12]	0

Source: After Fair et al. (1971).

(1) When very high pH values are produced by excess lime treatment; (2) by inclusion in precipitates; (3) but filters clog too rapidly at high turbidities; (4) not including chlorophenol tastes; (5) when break-point chlorination is employed or superchlorination is followed by dechlorination; (6) when (5) is not employed in the presence of intense odors and tastes; (7) some coagulants convert carbonates into sulfates; (8) by removal of carbon dioxide; (9) by addition of oxygen when it is low; (10) some coagulants release carbon dioxide; (11) variable, some metals are attacked at high pH values; (12) after aeration.

for advanced surface water treatment, while Fig. 11.15 shows a WTP for treating either surface water or GWUDI water. Typical groundwater treatment processes are shown in Figs. 11.16 and 11.17.

11.12 SELECTION OF WATER TREATMENT TECHNOLOGIES

Defining treatment objectives and selecting *control technologies* involve eight basic considerations:

1. Treated water requirements
2. Influent characteristics
3. Existing system configuration
4. Water treatment costs
5. Operation requirements
6. Pretreatment and posttreatment components
7. Waste management
8. Future needs of the service area

11.12.1 Treated Water Requirements and Influent Characteristics

Federal drinking water regulations, which include treated water requirements, are the primary factors determining water treatment goals. Supplementary effluent criteria from regulatory recommendations, WHO (World Health Organization) recommendations, professional organizations, state regulations, and consumer preferences are also taken into considerations. For further details on these effluent standards, the readers are referred to Chapter 10 of this book. Comparisons of existing influent characteristics with effluent requirements provide the basis for identifying treatment needs.

11.12.2 Existing System Configuration

The adaptability of an existing water treatment configuration to larger or different processes is an important consideration in assessing treatment options. Chapters 1–9 provide the basic knowledge for assessing the existing water system and predicting the future appropriate water works.

11.12.3 Water Treatment Costs

Total water treatment costs are divided into one-time *capital costs* and annual *operating and maintenance costs*. Each water treatment option has a different combination of capital and operating and maintenance costs, with higher capital costs often being associated with lower operation and maintenance costs.

11.12.4 Operation Requirements

Consistency in the nature and volume of the influent water is a crucial operating parameter. As the consistency of the influent water decreases, the necessity of monitoring and the operating complexity of most systems increase.

Other important operating considerations include energy requirements, chemical availability, chemical consumption rate, instrumentation and automation, preventative maintenance, noise, esthetics, backup/redundant systems, startup phase requirements, and cleaning and backwashing requirements.

11.12.5 Pretreatment and Posttreatment Processes

All water treatment technologies perform differently with different pretreatment and posttreatment processes. The

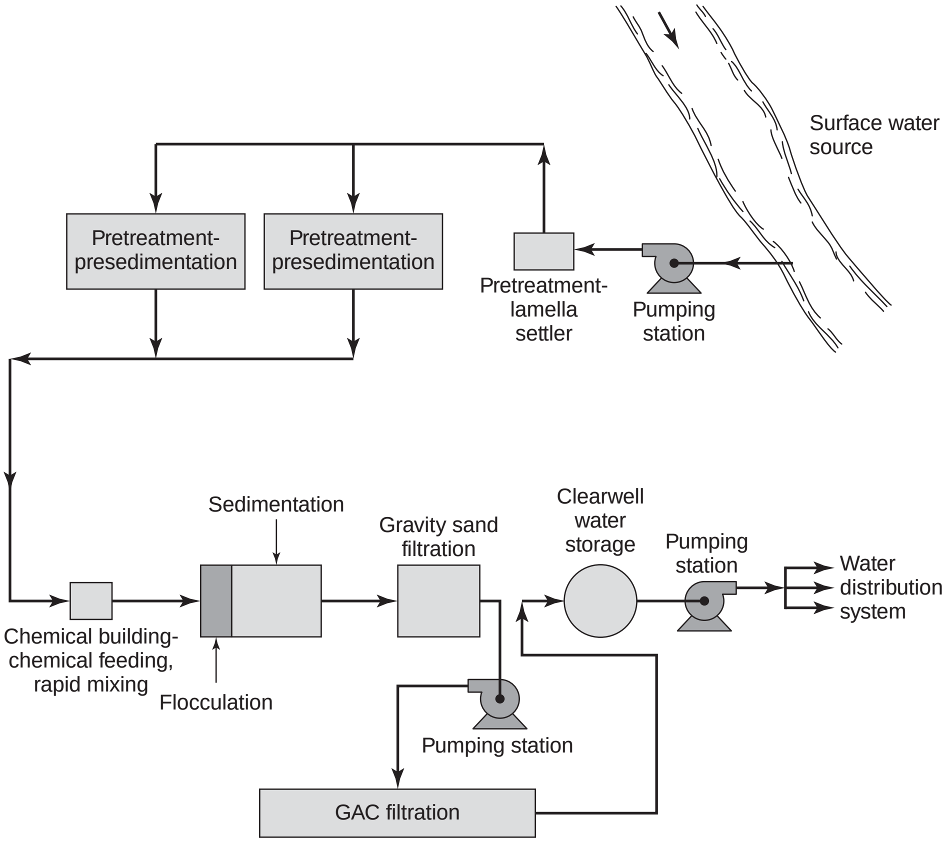


Figure 11.14 Granular activated carbon (GAC) filtration for advanced treatment in a conventional water treatment plant (*Source*: US EPA).

compatibility of all the processes in the treatment train is key to achieving individual treatment goals. Figures 11.1 and 11.14 show that presedimentation lamella settler and conventional presedimentation are common pretreatment and GAC is a common posttreatment. UV and membrane filtration are popular pretreatment and posttreatment, respectively. The combination of anion exchanger, membrane filtration, and GAC is a new posttreatment for removal of residual NOM from water. More pretreatment and posttreatment processes for removal of organic, inorganic and microbial contaminants are discussed in Sections 11.13 and 11.14.

11.12.6 Waste Management

Management of waste solutions, gases, sludges, and solids is a concern associated with the removal of contaminants in drinking water. Most treatment processes concentrate contaminants, sometimes hazardous, into residuals that require special handling. In Massachusetts, the waste alum sludge from the City of Pittsfield's DAF WTP has been discharged to the city's wastewater treatment plant for phosphate removal since 1986. Pretreatment of water plant wastes prior to river discharge or sewer discharge is another waste management option. Figures 11.9 and 11.15 illustrate how a full-scale 48 MGD (181.7 MLD) upflow clarification–filtration WTP and a full-scale 20 MGD (75.7 MLD) conventional WTP, respectively, manage their waste sludges and spent filter backwash wastewaters.

The volumetric flows of waste sludge, spent filter backwash wastewater, thickener effluent, and press effluent (pressate) in each stage are roughly estimated. Figure 11.10 shows the recommended monitoring locations throughout a WTP including waste management.

11.12.7 Future Service Area Needs

Population and economic forecasts of the service area are the basic tools for evaluating future demands. Examination and analyses of the watershed(s) of present and potential water supplies are important to determine the vulnerability of supplies.

11.13 CONTROL OF TURBIDITY, COLOR, AND BIOLOGICAL CONTAMINATION

The three basic regulatory requirements associated with filtration systems are control of turbidity, color, and biological contamination (*Giardia cysts*, *Cryptosporidium*, enteric viruses, and coliform bacteria). Other key considerations in selecting a filtration system include frequency of the cleaning cycle, chemical requirements, operational complexity, sludge volume, and toxicity.

In most conventional filtration systems, screening, mixing, flocculation, and sedimentation processes typically precede actual filtration as shown in Figs. 11.4 and 11.5. While these pretreatment elements are always found in conventional

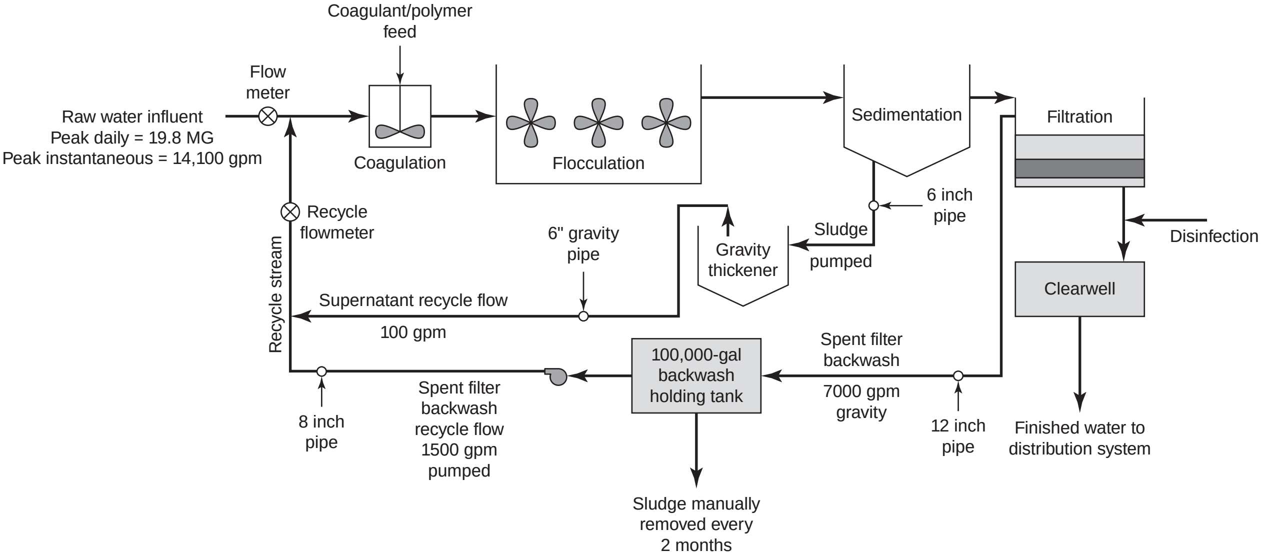


Figure 11.15 Flow diagram and flow balance of a 20 MGD (75 MLD) conventional water filtration plant with wastewater recycle (*Source*: US EPA).

filtration systems they are also sometimes found in nonconventional systems. In addition to the four prefiltration steps, additives, such as chemicals for pH adjustment, *coagulants*, *coagulant aids*, *PAC*, and *polymers*, are commonly used in conjunction with filtration.

The seven filtration options reviewed in this chapter and Chapter 16 include conventional filtration, direct filtration, slow sand filtration, package plant filtration, DE filtration, membrane filtration (reverse osmosis), and cartridge filtration.

The performance of each filter type depends on the quality of the influent and proper design and operation. The range of influent characteristics for which various filters are effective is provided in Table 17.16.

Conventional filtration, with rapid mix, flocculation, and sedimentation, is clearly the most versatile in its effectiveness in treating variable influents. Coagulation/filtration systems are more difficult to operate compared to slow sand or DE filters because they involve adjusting water chemistry for proper coagulation. Slow sand, DE, membrane, and cartridge filtration units do not have a coagulation step. The complexity of operating a package filtration plant varies with the manufacturer and model. Package plants, slow sand, membrane, and cartridge filtration are most applicable to small systems. Figures 11.13a–11.13b and 11.16a–11.16b illustrate slow sand filtration and DE filtration, respectively. Figure 11.11b introduces the Lenox Water Treatment Plant, Massachusetts, which is a 1 MGD (3.785 MLD) package filtration plant consisting of chemical feeding, primary disinfection, rapid mixing, coagulation–flocculation, DAF clarification, continuous automatic backwash sand filtration (ABF), corrosion control, secondary disinfection, and clearwell. Figure 11.17 introduces the City of Cape Coral Reverse Osmosis Water Treatment Facility in Florida, which is a 15 MGD (57 MLD) membrane filtration plant consisting of (a) chemical feeding (H_2SO_4 acidification, anti-scalant addition); (b) cartridge filtration; (c) membrane filtration (RO); (d) degasification (aeration for H_2S gas removal); (e) disinfection (chlorination), (f) corrosion control by neutralization with NaOH; and (g) clearwell.

11.14 ORGANIC CONTAMINANT REMOVAL

The 1986 Amendments to the US EPA's SDWA (*Safe Drinking Water Act*) require the establishment of new maximum contaminant levels (MCLs) for many organic contaminants, including DBPs. The regulations designate BATs (best available technologies) as well as MCLs for organic contaminants. The July 1987 regulations specify *packed column aeration* (PCA) and GAC as the BAT for seven of the eight VOCs. PCA is BAT for vinyl chloride, the eighth VOC. Water utilities with these contaminants will have to provide removals at least equivalent to those achieved by the designated BATs.

The removal capabilities of selected treatment technologies for several organic contaminants and contaminant classes are provided in Table 11.2. The readers are referred to Section 11.20 for the process terminologies introduced in all tables of this chapter. The first listed treatment, coagulation/filtration is widely used primarily for turbidity and microbial contaminant control, but can be somewhat effective in removing certain organic compounds. GAC and PCA are designated as BAT for many of the organic chemicals and are much more effective. Costs for either technology vary depending on the contaminant removed and whether waste management is a potential issue. PAC, the fourth treatment listed in the table, is only suited for application to conventional systems with rapid mix, flocculation, sedimentation, and filtration components. PAC also addresses some taste and odor problems. This

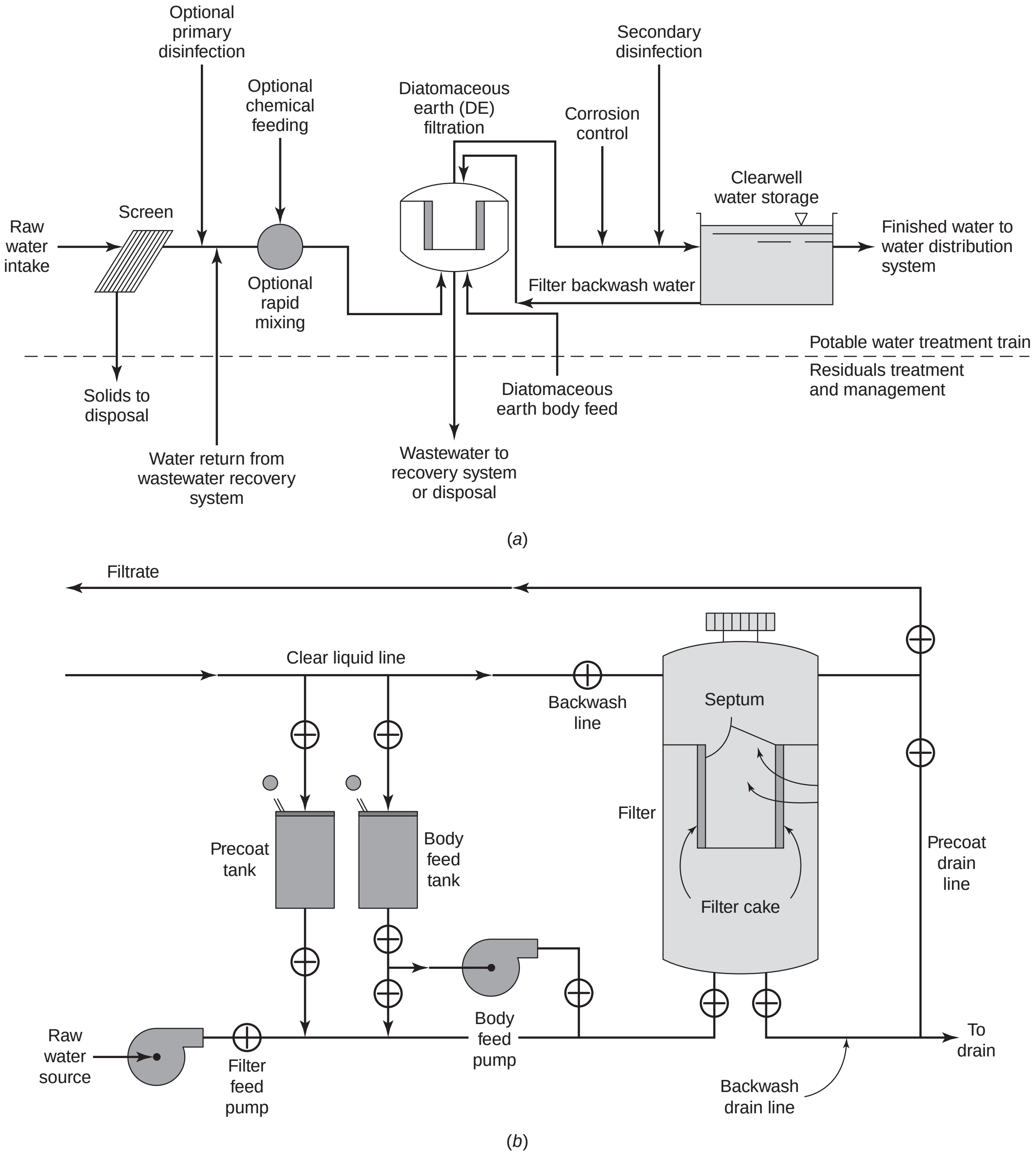


Figure 11.16 (a) Typical flow diagram of a diatomaceous earth filtration process (A modified figure). (b) Vertical view of a diatomaceous earth filtration process (*Source*: Wang et al., 2006).

powdered form of activated carbon is especially suitable for seasonal organic contaminant problems since it can be added as needed. In some instances, however, such high dosages of PAC are required to achieve organic removal that waste management becomes a problem. The readers are referred to Chapters 13 and 20 for details. Diffused aeration, oxidation, reverse osmosis, multiple tray aeration, caternary aeration, Higee aeration, UF, and mechanical aeration are also defined in Section 11.20. Example 11.10 introduces the AOP, which has higher efficiency for removal of organic contaminants, in comparison with common oxidants, such as potassium permanganate, ozone, hydrogen peroxide, and oxygen.

All other listed treatments, including the different aeration configurations, are less established. The aeration

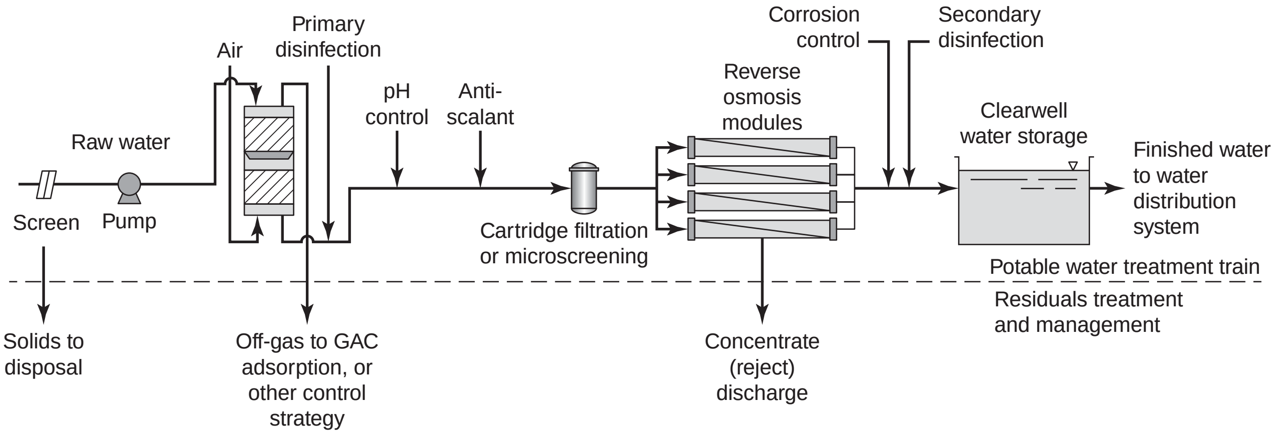


Figure 11.17 Typical flow diagram of a reverse osmosis water treatment plant (A modified figure).

treatments, developed for specific applications, are generally equal to or less effective than PCA, but have higher energy requirements. For some, controlling biological growth in the systems is a problem. The remaining treatments, besides the aeration technologies, all tend to be very effective in removing contaminants, but their application to specific organics is still experimental. The efficiencies of all treatments depend on the type and concentration of the contaminants. While pretreatment is not always required, it can increase the effectiveness of some of the treatments. Table 11.3 illustrates the variation in operating conditions for these treatments.

11.15 INORGANIC CONTAMINANT REMOVAL AND CONTROL

Inorganic contaminant treatments (Chapters 12 and 14–20) are categorized as prevention strategies or removal technologies. Corrosion controls prevent or minimize the presence of corrosion products (inorganic contaminants) at the point of use. Removal technologies treat source water that is contaminated with metals or radionuclides.

11.15.1 Corrosion Controls

Corrosion controls address the two primary aspects of corrosion: (a) water quality characteristics and (b) materials subject to corrosion. The two most significant water quality contaminants that influence corrosion are pH and carbonate/bicarbonate alkalinity. Components of the distribution system subject to corrosion include pipes, valves, meters, plumbing, solder, and flux. The longer the contact time between the water and corrodible materials in the distribution system, the higher the concentration of dissolved metals in the drinking water.

The four general types of corrosion controls are (a) adjustments to water pH, (b) addition of corrosion inhibitors to the water to form protective coatings over the potentially corrodible metal, (c) electronic cathodic protection, and (d) applied coating and linings.

The most commonly used corrosion control is pH adjustment because it is inexpensive and easily applied. Table 11.4 compares corrosion controls.

Corrosion controls involving pH adjustment may conflict with ideal pH conditions for disinfection and control of DBPs. The treatment methods selected for both treatment objectives should be carefully coordinated to avoid diminishing the effectiveness of either process.

11.15.2 Inorganic Contaminant Removal

There are 10 treatment processes for addressing the many inorganic contaminants including radionuclides. Most treatment processes are effective for only a specific set of contaminants under certain circumstances. Fortunately, all of the inorganic contaminants do not often occur simultaneously. The most appropriate applications for each treatment process are shown in Table 11.5. This table distinguishes between groundwater and surface water by the generally higher concentrations of suspended solids in most surface water. Table 11.6 provides removal efficiencies for specific contaminants with the same treatments. For radon removal, which is not included in the table, aeration is highly affective.

The most important factors affecting inorganic contaminant removal are (a) contaminant type and valence, (b) influent contaminant concentration, (c) influent levels of dissolved solids and pH, and (d) desired effluent concentration. The first three factors are site specific. Table 11.7 summarizes the advantages and disadvantages of each of the treatment mechanisms.

11.15.3 Radionuclides Removal and Risk Control

Radionuclides are radioactive, cancer-causing substances that release radiation (alpha, beta, and gamma). Radionuclides include (a) radium-226, radium-228, natural uranium, radon, gross alpha, gross beta, and photon emitters; and (b) strontium-90, lead-210, polonium-210, thorium-230, and

Table 11.2 Treatment technology removal effectiveness for organic contaminants (percent)

Contaminant	Coagulation/ filtration	GAC	PCA	PAC	Diffused aeration	Oxidation[a]	Reverse osmosis
Acrylamide	5	NA	0–29	13	NA	NA	0–97
Alachlor	0–49	70–100	70–100	36–100	NA	70–100	70–100
Aldirarb	NA	NA	0–29	NA	NA	NA	94–99
Benzene	0–29	70–100	70–100	NA	NA	70–100	0–29
Carbofuran	54–79	70–100	0–29	45–75	11–20	70–100	70–100
Carbon tetrachloride	0–29	70–100	70–100	0–25	NA	0–29	70–100
Chlordane	NA	70–100	0–29	NA	NA	NA	NA
Chlorobenzene	0–29	70–100	70–100	NA	NA	30–69	70–100
2,4-D	0–29	70–100	70–100	69–100	NA	W	0–65
1,2-Dichloroethane	0–29	70–100	70–100	NA	42–77	0–29	15–70
1,2-Dibromochloropropane	0–29	70–100	70–100	NA	42–77	0–29	10–100
Dibromochloropropane	0–29	70–100	30–69	NA	NA	0–29	NA
Dichlorobenzene	NA	70–100	NA	NA	NA	NA	NA
o-Dichlorobenzene	0–29	70–100	70–100	38–95	14–72	30–88	30–69
p-Dichlorobenzene	0–29	70–100	70–100	NA	NA	30–69	0–10
1,1-Dichloroethylene	0–29	70–100	70–100	NA	97	70–100	NA
cis-1,2-Dichloroethylene	0–29	70–100	70–100	NA	32–85	70–100	0–30
trans-1,2-Dichloroethylene	0–29	70–100	70–100	NA	37–96	70–100	0–30
Epichlorohydrin	NA	NA	0–29	NA	NA	0–29	NA
Ethylbenzene	0–29	70–100	70–100	33–99	24–89	70–100	0–30
Ethylene dibromide	0–29	70–100	70–100	NA	NA	0–29	37–100
Heptachlor	64	70–100	70–100	53–97	NA	70–100	NA
Heptachlor epoxide	NA	NA	NA	NA	NA	70–100	NA
High molecular weight hydrocarbons (gasoline, dyes, amines, humics)	NA	W	NA	NA	NA	NA	NA
Lindane	0–29	70–100	0–29	82–97	NA	0–100	50–57
Methoxychlor	NA	70–100	NA	NA	NA	NA	>90
Monochlorobenzene	NA	NA	NA	14–99	14–85	86–98	50–100
Natural organic material	P	P	NA	P	NA	W	P
PCBs	NA	70–100	70–100	NA	NA	NA	95
Phenol and chlorophenol	NA	W	NA	NA	NA	W	NA
Pentachlorophenol	NA	70–100	0	NA	NA	70–100	NA
Styrene	0–29	NA	NA	NA	NA	70–100	NA
Tetrachloroethylene	NA	70–100	NA	NA	73–95	W	70–90
Trichloroethylene	0–29	70–100	70–100	NA	53–95	30–69	0–100
Trichloroethane	NA	70–100	NA	NA	NA	NA	95
1,1,1-Trichloroethyane	0–29	70–100	70–100	40–65	58–90	0–29	15–100
Toluene	0–29	70–100	70–100	0–67	22–89	70–100	NA
2,4,5-TP	63	70–100	NA	82–89	NA	30–69	NA
Toxaphene	0–29	70–100	70–100	40–99	NA	NA	NA
Vinyl chloride	0–29	70–100	70–100	NA	NA	70–100	NA
Xylenes	0–29	70–100	70–100	60–99	18–89	70–100	10–85

Source: US EPA.

Note: Little or no specific performance data were available for (1) multiple tray aeration; (2) catenary aeration; (3) Higee aeration; (4) resins; (5) ultrafiltration.

W, well removed; P, poorly removed; NA, not available.

[a] The specifics of the oxidation processes effective in removing each contaminant are provided in later chapters.

thorium-232. The US EPA's MCLGs for "Group a" radionuclides in drinking water are zeros. Radium in drinking water can be effectively removed by (a) lime softening process; (b) membrane filtration (reverse osmosis); (c) iron and manganese oxidation process; (d) cation ion exchange; (e) anion ion exchange; and (f) conventional coagulation–flocculation and filtration process. Radon in drinking water is commonly removed by (a) aeration process; (b) GAC filtration; and (c) PAC in conventional coagulation–flocculation filtration process. Uranium in drinking water is normally removed by (a) anion exchange process; (b) lime softening process at high pH; and (c) membrane filtration (reverse osmosis).

Table 11.3 Operational conditions for organics treatments

Technology	Level of operational skill required	Level of maintenance required	Energy requirements
Coagulation/filtration	High	High	Low
GAG	Medium	Low	Low
PCA	Low	Low	Varies
PAC	Low	Medium	Low
Diffused aeration	Low	Low	Varies
Multiple tray aeration	Low	Low	Low
Oxidation	High	High	Varies
Reverse osmosis	High	High	High
Mechanical aeration	Low	Low	Low
Catenary grid	Low	Low	High
Higee aeration	Low	Medium	High
Resins	Medium	Medium	Low
Ultrafiltration	Medium	High	Medium

Source: US EPA.

Table 11.4 Corrosion control considerations

Corrosion controls	Amount of system covered by the control	Completely agreeable with consumers	Optimum level of additive in the water
Water treatment			
pH adjustment	Total	Yes	>8.0 pH
Inorganic phosphate	Total	Usually	Varies
Silicates	Total	Yes	2–12 mg/L
Cathodic protection	Partial	Yes	NA
Coatings and linings	Partial	Yes	NA

Source: US EPA.
NA, not applicable.

Table 11.5 Most probable treatment application for inorganic contaminants

Treatment	Most probable application	
	Source water	Contaminants
Conventional with coagulation	Surface	Ag, As, Cd, Cr, or Pb
Lime softening	Surface (hard)	As, V, Cd, Cr^{3+}, F, or Pb
	Groundwater	Ba or Ra
Ion exchange: Cation	Groundwater	Ba or Ra
Anion	Groundwater	NO_3, Se
Reverse osmosis	Groundwater	Al
Powdered activated carbon	Surface water	Organic Hg from spills
Granular activated carbon	Surface water	Organic Hg
	Groundwater	Organic Hg
Activated alumina	Groundwater	As, F, or Se

Source: US EPA.

The occurrence of natural radionuclides in drinking water supplies can be interpreted by the *average population weighted concentrations* (APWC) of radionuclides:

$$\mathrm{APWC} = C(P_{\mathrm{exp}}/P_{\mathrm{total}}) \tag{11.2}$$

where APWC is the average population weighted concentration, pCi/L; C is the radioactivity concentration, pCi/L; P_{exp} is the number of people exposed to that concentration, #; and P_{total} is total number of people, #. Population risk of radionuclides in drinking water for a country or an area can be calculated by the following equation:

$$\mathrm{RISK} = \mathrm{APWC} \times \mathrm{RATE} \times P_{\mathrm{area}} \tag{11.3}$$

where RISK is the population risk, excess cases/lifetime in an area; APWC is the average population weighted concentration, pCi/L; RATE is the cancer risk rate, excess cases/lifetime/person/(pCi/L); and P_{area} is the total population for a country or an area to be investigated.

Table 11.6 Removal effectiveness of inorganic contaminants by nine processes

Treatment	Contaminant																				
	Ag	As	As^{3+}	As^{5+}	Ba	Cd	Cr	Cr^{3+}	Cr^{6+}	F	Hg	$Hg^{(O)}$	Hg^{3+}	NO_3	Pb	Ra	Rn	Se	Se^{5+}	Se^{3+}	U
Conventional treatment	H	–	M	H	L	H	–	H	H	L	–	M	M	L	H	L	–	–	M	L	M
Coagulation aluminum	H	–	–	H	–	M	–	H	–	–	M	–	–	–	H	–	–	–	–	–	–
Coagulation iron	M	–	–	H	–	–	–	H	H	–	–	–	–	–	–	–	–	–	–	–	–
Lime softening	–	–	M	H	H	H	–	H	L	M	–	L	M	L	H	H	–	–	M	L	H
Reverse osmosis and electrodialysis	H	–	M	H	H	H	H	–	–	H	H	–	–	M	H	H	–	H	–	–	H
Cation exchange	–	L	–	–	H	H	–	H	L	L	–	–	–	L	H	H	–	L	–	–	H
Anion exchange	–	–	–	–	M	M	–	M	H	–	–	–	–	H	M	M	–	H	–	–	H
Activated alumina	–	–	H	–	L	L	–	–	–	H	–	–	–	–	–	L	–	H	–	–	–
Powdered activated carbons	L	–	–	–	L	M	–	L	–	L	–	M	M	L	–	L	–	–	–	–	–
Granular activated carbons	–	–	–	–	L	M	–	L	–	L	–	H	H	L	–	L	H	–	–	–	–

Source: US EPA.
H, high = >80% removal; M, medium = 20–80% removal; L, low = <20% removal; "–", indicate no data provided.

Table 11.7 Advantages and disadvantages of inorganic contaminants removal processes

Precipitation and coprecipitation used in coagulation/filtration

Advantages

- Low cost for high volume
- Often improved by high ionic strength
- Reliable process well suited to automatic control

Disadvantages

- Stoichiometric chemical additions required
- High-water-content sludge must be disposed of
- Part-per-billion effluent contaminant levels may require two-stage precipitation
- Not readily applied to small, intermittent flows
- Coprecipitation efficiency depends on initial contaminant concentration and surface area of primary floc

Ion exchange

Advantages

- Operates on demand
- Relatively insensitive to flow variations
- Essentially zero level of effluent contamination possible
- Large variety of specific resins available
- Beneficial selectivity reversal commonly occurs upon regeneration

Disadvantages

- Potential for chromatographic effluent peaking
- Spent regenerant must be disposed of
- Variable effluent quality with respect to background ions
- Usually not feasible at high levels of total dissolved solids

Activated alumina

Advantages

- Operates on demand
- Insensitive to flow and total dissolved solids background
- Low effluent contaminant level possible
- Highly selective for fluoride and arsenic

Disadvantages

- Both acid and base are required for regeneration
- Media tend to dissolve, producing fine particles
- Slow adsorption kinetics
- Spent regenerant must be disposed of

Membranes (*reverse osmosis and electrolysis*)

Advantages

- All contaminant ions and most dissolved non-ions are removed
- Relatively insensitive to flow and total dissolved solids level
- Low effluent concentration possible
- In reverse osmosis, bacteria and particles are removed as well

Disadvantages

- High capital and operating costs
- High level of pretreatment required
- Membranes are prone to fouling
- Reject stream is 20–90% of feed flow

Source: US EPA.

11.16 WATER RENOVATION

There is growing evidence that the need for water by the municipalities, industries, and agriculture of some areas is outstripping the supply of natural waters, that is, waters that have passed through the evaporative and precipitative phases of the hydrological cycle. For this reason the profession is being encouraged to discover ways and means for a fuller purification of all waters that have completed their use cycle in the service of man. On the one hand, pursuance of nature's way of purification by evaporation of all needed waters appears to remain outside the economic reach of present-day society. On the other hand, neither conventional wastewater treatment nor conventional water purification processes promise to accomplish 100% the equivalent of water evaporation and condensation. The removal of 90–95% of the suspended solids, biochemical oxygen demand (BOD), and *chemical oxygen demand* (COD) of wastewaters before the discharge of effluents into receiving waters is not considered enough, nor is the subsequent natural purification of receiving waters for reuse by man accepted as sufficiently rigorous to assure the safety and palatability as well as the general usefulness of such waters. There are new pollutants, among them the wastes from the synthesis of organic chemicals and residues from the dissemination of biocidal chemicals, in the environment. New, also, is the lengthening life span of man, which subjects him to exotic chemicals of this kind for many more years than before and aggravates, in time and concentration, the pollutional insults to which he is exposed.

In these circumstances, departure from conventional treatment processes is considered to be in order, together with intensification of compensatory action within the use cycle of water. A general water renovation technology will be concerned with (a) the addition of a tertiary step to the primary and secondary treatment of all wastewaters, (b) the enhancement of the natural purification of receiving waters and their better protection against stormwater flows and overflows as well as agricultural drainage, (c) more stringent quality management of regional waters as part of the total natural resource, and (d) the addition of highly selective processes to available purification methods for water supply.

Component actions include, besides the exhaustive use, exploration, and intensification of established treatment processes, some of the following: (a) greater emphasis on adsorptive processes because of their ability to remove substances selectively and when present in minute concentrations, the adsorption of toxicants as well as odors and tastes from water by activated carbons being an example; (b) introduction of a broader spectrum of oxidants, the destruction of odors and tastes as well as color by ozone being an example; (c) exploration of foam separation and foam harvesting, including the removal of foaming impurities such as the synthetic detergents and the purposeful injection of foaming agents in wide variety for lifting unwanted solutes and suspensions from water; and (d) expanded demineralization

EXAMPLE 11.11 UNDERSTANDING THE UNITS AND OCCURRENCE OF NATURAL RADIONUCLIDES IN DRINKING WATER

Define *curie* (Ci), *picocurie* (pCi), *becquerel* (Bq), *rem*, *sievert*, *working level*, *organ weighting factors*, and *average population weighted concentration*. What is the occurrence of natural radionuclides in the US drinking water supplies?

Solution:

Curie (Ci) is a unit of radioactivity mainly used in the United States, representing the rate at which nuclear radiations are emitted. One curie equals 0.037 billion disintegrations per second, or approximately the radioactivity of 1 g of radium-226. 1 Ci = 0.037×10^{12} Bq. *Picocurie* (pCi) is also a unit of radioactivity mainly used in the United States representing the rate at which nuclear radiations are emitted. One picocurie (1 pCi) represents a quantity of radioactive material with an activity equal to one billionth of a curie, that is, 10^{-12} curie.

Becquerel (Bq) is an international unit of radioactivity representing the rate at which nuclear radiations are emitted. The becquerel is the activity of a radionuclide decaying at the rate of one disintegration (one spontaneous nuclear transition) per second.

Rem is a dose equivalent unit, representing the effect of ionizing radiation on tissue. Specifically rem is a unit of dose equivalent from ionizing radiation to total body or any internal organ or organ system. Sievert is also a dose equivalent unit, representing the effect of ionizing radiation on tissue. One sievert equals 100 rem.

Working level (WL) is a unit used to describe the dose of radionuclides due to progeny of radon. *Organ weighting factor* is a factor representing relative sensitivities of organs to ionizing radiation and yielding effective dose equivalent that can be summed for all organs.

APWC can be calculated by the equation APWC = $C\,(P_{\mathrm{exp}}/P_{\mathrm{total}})$, where APWC is the population weighted average, pCi/L; C is the radioactivity concentration, pCi/L; P_{exp} is the number of people exposed to that concentration, #; and P_{total} is the total number of people, #.

The occurrence of natural radionuclides in drinking water supplies (average of surface water and groundwater supplies) can be represented by the calculated APWC values. The US EPA has reported the occurrence of natural radionuclides in the US drinking water supplies as follows: (a) radium-226 = 0.3–0.8 pCi/L; (b) radium-228 = 0.4–1.0 pCi/L; (c) uranium-natural = 0.3–2.0 pCi/L; (d) radon-222 = 50–300 pCi/L; (e) lead-210 = less than 0.11 pCi/L; (f) polonium-210 = less than 0.13 pCi/L; (g) thorium-230 = less than 0.04 pCi/L; and (h) thorium-232 = less than 0.01 pCi/L.

EXAMPLE 11.12 UNDERSTANDING THE RISK OF RADIUM IN DRINKING WATER AND PROPOSING A WATER TREATMENT SYSTEM FOR RADIUM REMOVAL

Assume the occurrence in drinking water in terms of APWC is 0.3–0.8 pCi/L for radium-226, the cancer risk rate of radium is about 2.2–35 excess cases per pCi/L, per lifetime and per 1 million people, and the US population is 314 million (2012 population), determine the population risk for radium in the United States. Assume the raw groundwater contains 15 pCi/L of combined radium-226 and radium-228, and all other water quality parameters meet the drinking water standards. Recommend some appropriate processes for treatment of the raw groundwater and indicate their treatment efficiencies. Discuss your recommended water systems.

Solution:

Population risk of radionuclides in drinking water for a country or area can be calculated by Eq. (11.3):

$$\mathrm{RISK} = \mathrm{APWC} \times \mathrm{RATE} \times P_{\mathrm{area}} \tag{11.3}$$

APWC = the average population weighted concentration = 0.3–0.8 pCi/L.

RATE = the cancer risk rate = $2.2\text{–}35/10^6$ excess cases/lifetime/person/(pCi/L).

P_{area} = the total population for a country or an area to be investigated = 314×10^6.

$$\begin{aligned}\textbf{RISK} &= \text{population risk}\\ &= [(0.3 \text{ to } 0.8)\text{pCi/L}][(2.2 \text{ to } 35) \times 10^{-6}/(\text{pCi/L})][314 \times 10^6]\\ &= \textbf{207 to 8,792 excess cases/lifetime in the United States.}\end{aligned}$$

The treatment efficiencies of feasible processes for radium removal are (a) greater than 75% radium removal by conventional coagulation–flocculation filtration process; (b) greater than 85% radium removal by the lime softening process; (c) greater than 95% radium removal by anion ion exchange process; (d) greater than 90% radium removal by cation ion exchange process; (e) less than 40% radium removal by iron and manganese oxidation process; and (f) greater than 90% radium removal by membrane filtration (reverse osmosis) process.

Discussion: All water quality parameters, except radium, of the raw groundwater meet the drinking water standards. Since the US Primary Drinking Water Standards for combined radium-226 + radium-228 is 5 pCi/L, and the groundwater contains 15 pCi/L of combined radium, a treatment efficiency of greater than 66.7% is required. The feasible processes for radium removal include conventional coagulation–flocculation filtration process, lime softening process, anion ion exchange process, cation ion exchange process, and membrane filtration (reverse osmosis) process. Selection of the conventional coagulation–flocculation filtration process for radium removal will give the lowest costs. Selection of cation ion exchange process will have the second lowest costs, and the ion exchange process will be the simplest one in comparison with other feasible processes. Accordingly the recommended process for radium removal is cation ion exchange process. A complete municipal water system (Fig. 11.18) for radium removal, corrosion control, and disinfection includes screening, pumping, optional primary disinfection, cartridge filtration, cation ion exchange, corrosion control, secondary disinfection, clearwell, water storage, and water distribution.

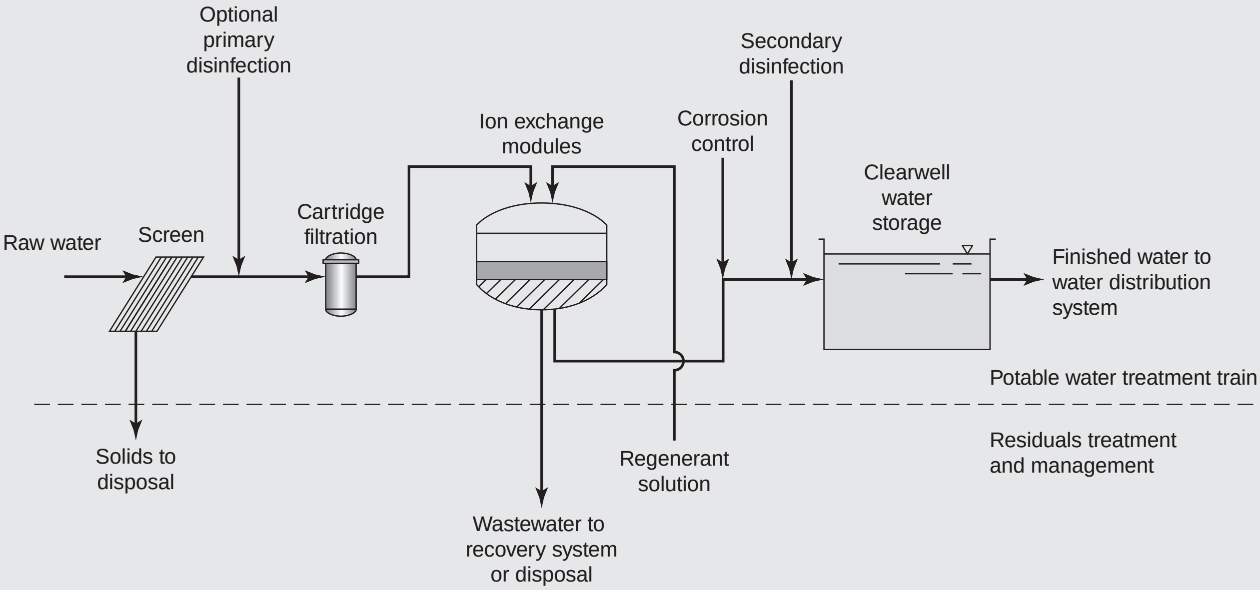


Figure 11.18 Typical process train of an ion exchange plant for treating groundwater (A modified figure).

For the point of use by single families, the ion exchange column (cation) alone will be sufficient for further treatment of the tap water aiming at radium removal only.

The treatment efficiencies of feasible processes for radon removal are (a) 99% radon removal by aeration process and (b) 78–99% radon removal (depending on the initial radon concentration) by GAC filtration.

of water by electrodialysis (ED), electrodialysis reversal (EDR), ion exchange (IX), reverse osmosis (RO), distillation, and freezing. Most of these processes have already found their way in some measure into water purification and wastewater tertiary treatment practices. They are placed in a single category here to draw attention to the trend in water needs and the contemplated action being stimulated in the management of large-scale water quality programs.

11.17 TREATMENT KINETICS

The principal operations and systems of water treatment are generally slow—so slow, in fact, that the rates at which they proceed generally govern the technical equilibrium or purification efficiency that can be attained with justifiable economy in a given treatment operation or system of operations. Whatever can be done, therefore, to speed the changes to be wrought bears directly on the space requirements of necessary treatment systems. Spatial needs, in turn, determine the kind and costs of required structures.

Studies of the time dependence of purification processes and of observed variations under different conditions of exposure point to the following kinetic concepts: (1) reaction kinetics, governing not only simple, normally rapid chemical reactions but also complex, often slow, and encumbered biochemical reactions and changes in the population of associated living organisms; and (2) transfer kinetics, governing the transport of substances to and across phase boundaries or interfaces between adjacent phases, and further distinguishable as the kinetics of diffusion, sorption, ion exchange, and conjunction, as expressions of transfer that takes place in water treatment.

Within limits, the effectiveness of reaction kinetics can generally be enhanced by raising the temperature of the mix

EXAMPLE 11.13 UNDERSTANDING THE RISK OF RADON IN DRINKING WATER AND PROPOSING A WATER TREATMENT SYSTEM FOR RADON REMOVAL

Assume the occurrence in drinking water in terms of APWC is 50–300 pCi/L for radon, the cancer risk rate of radon is about 0.2–6 excess cases per pCi/L, per lifetime and per 1 million people in North America, and Canada's population is 34.5 million (2011 population), determine the population risk for radon in Canada. Assume the raw groundwater of a Canadian community contains 150 pCi/L of radon, and all other water quality parameters meet the drinking water standards. Recommend some appropriate processes for treatment of the raw groundwater and indicate their treatment efficiencies. Discuss your recommended water system.

Solution:

APWC = 50–300 pCi/L.

RATE = $0.2\text{–}0.6/10^6$ excess cases/lifetime/person/(pCi/L).

$P_{\text{area}} = 34.5 \times 10^6$.

$$\begin{aligned}\textbf{RISK} &= \text{APWC} \times \text{RATE} \times P_{\text{area}} \qquad \textbf{(11.3)}\\ &= [(50\text{ to }300)\text{ pCi/L}][(0.2\text{ to }0.6) \times 10^{-6}/(\text{pCi/L})][34.5 \times 10^6]\\ &= \textbf{345 to 6,210 excess cases/lifetime in Canada.}\end{aligned}$$

Discussion: The low cost aeration process is recommended for removal of radon from drinking water by a municipal water system. Using Fig. 11.1 as a guide, the recommended municipal water system includes well field, pumping, aeration, corrosion control, disinfection, clearwell, water storage, and water distribution.

GAC filtration, however, is recommended as a point of use unit by individual families due to its simplicity in installation and operation.

and the effectiveness of transfer kinetics in ways such as the following: (1) enlargement of the phase boundary either statically through expansion of the interfacial area relative to the volume of water undergoing treatment or dynamically through rapid renewal or clearance of the interface by useful power dissipation and (2) steepening the transfer gradient and with it the driving force across the phase boundary either directly by changing the relative concentrations or concentration gradients within the system or indirectly through the introduction of countercurrent flow, flow recirculation, or similar methods.

Ordinarily both reaction kinetics and transfer kinetics need to be supported by interlocking formulations that measure responses to a wide range of physical conditions. Some of these responses can be expressed directly in terms of ambient temperatures or indirectly in terms of their influence on the properties of water substance, for instance; others must account, in suitable terms, for complex longitudinal, that is, time-dependent and process-dependent, environmental changes.

Examples of progressive natural purification and of purposeful treatment that can be formulated in terms of reaction kinetics are the BOD exerted in the natural purification of streams; the natural die-away of pathogens and pollutional indicator organisms in clean and polluted waters; and induced and accelerated die-away of pathogens and other living things by heat, chemicals, and destructive radiations.

Categories of progressive natural purification and of purposeful treatment that can be formulated in terms of transfer kinetics are exemplified by gas transfer to and from water; nutrient transfer to populations of living organisms; adsorption of odor- and taste-producing substances on activated carbon; removal of suspended matter by sedimentation, upflow, and filtration; flocculation for coagulation and precipitation; leaching or washing of flocs or sludges; and ion exchange in the removal of salinity and hardness.

11.18 MONITORING WATER QUALITY

Water systems monitor for a wide variety of contaminants to verify that the water they provide to the public meets all federal and state standards. Currently, the nation's community water systems (CWSs) and *nontransient noncommunity water systems* (NTNCWSs) must monitor for more than 83 contaminants. The major classes of contaminants include VOCs, *synthetic organic compounds* (SOCs), inorganic compounds (IOCs), radionuclides, and microbial organisms (including bacteria). Testing for these contaminants takes place on varying schedules and at different locations throughout the water system. Transient noncommunity water systems may monitor less frequently and for fewer contaminants than CWSs. Because these types of systems serve an ever-changing population, it is most important for them to monitor for contaminants such as microorganisms and nitrate that can cause an immediate, acute public health effect. Water systems also monitor for a number of contaminants that are currently not regulated. These monitoring data provide the basis for identifying contaminants to be regulated in the future.

11.19 DISTRIBUTION TO CUSTOMERS

An underground network of pipes typically delivers drinking water to the homes and businesses served by the water system. Small systems serving just a handful of households may be relatively simple. Large metropolitan water systems can be extremely complex sometimes with thousands of miles of piping serving millions of people. Although water may be safe when leaving the WTP it is important to ensure that this water does not become contaminated in the distribution system because of such things as water main breaks, pressure problems, or growth of microorganisms. Much of the existing drinking water infrastructure was built many years ago. The US EPA Infrastructure Needs Survey, released in 2001, estimated that drinking water systems will need to invest USD 150.9 billion over a 20-year period to ensure the continued source development, storage, treatment, and distribution of safe drinking water. Many agree this is a very conservative low estimate.

11.20 GLOSSARY OF WATER TREATMENT SYSTEMS

Activated alumina (AA) – A charged form of aluminum substance which is commonly used in combination with a synthetic, porous media in an ion exchange process, for removing charged impurities (such as fluoride) from water.

Activated carbon – A carbon activated by high-temperature steam or carbon dioxide to form a material of high adsorptive capacity.

Adsorbate – The materials, such as color bodies, taste and odor compounds, volatile organic compounds, and soluble lead, which are adsorbed on the surface of an adsorbent (activated carbon or polymeric adsorbent).

Adsorbent – A solid material which adsorbs such as clay, carbon, polymeric adsorbent.

Advanced oxidation process (AOP) – A disinfection and oxidation process that involves the use of (a) ozone plus hydrogen peroxide; (b) ozone plus UV; (c) hydrogen peroxide plus UV; (d) calcium peroxide plus UV; (e) magnesium peroxide plus UV; (f) potassium permanganate plus UV; and (g) any other strong oxidizing agent (such as chlorine and chlorine dioxide) plus UV.

Automatic backwash filtration (ABF) – A filtration system in which its filter bed is divided into multiple identical cells for continuous filtration and backwash without shutdowns. A traveling bridge with a hood moves to the top of one filter cell when it is due for backwash. The hood which is identical to the filter cell in shape can move up when the backwash is not needed, or move down to cover the filter cell for backwash, one cell at a time. An ABF can be a single medium filter, such as sand filter, or GAC filter, or a dual-media filter, such as anthracite over sand bed. The shape of ABF can be circular or rectangular.

Anion exchange – An ion exchange process involving the use of anion exchanger (anionic ion exchange resin), mainly for removal of negatively charged ions, such as sulfate, phosphate, and chloride, from water.

Average population weighted concentration (APWC) – A value representing the occurrence of natural radionuclides in drinking water supplies (average of surface water and groundwater supplies) which can be calculated by the equation $\mathrm{APWC} = C\,(P_{\mathrm{exp}}/P_{\mathrm{total}})$, where APWC is the population weighted average, pCi/L; C is the radioactivity concentration, pCi/L; P_{exp} is the number of people exposed to that concentration, #; and P_{total} is the total number of people, #.

Backwash – The process of reversing the flow of water back through the filter media to remove the entrapped solids.

Batch-flow sedimentation – One or more basins sized to receive a volume of flow, such as spent filter backwash water, in a specific period of time. The flow is detained for a specific period of time to allow sedimentation, and then the tank is emptied.

Becquerel (Bq) – An international unit of radioactivity representing the rate at which nuclear radiations are emitted. The becquerel is the activity of a radionuclide decaying at the rate of one disintegration (one spontaneous nuclear transition) per second.

Best available technology (BAT) – The best technology, treatment techniques, or other means which the US EPA Administrator finds, after examination for efficacy under field conditions and not solely under laboratory conditions, are available (taking cost into consideration).

Breakthrough – A condition whereby filter effluent water quality deteriorates (as measured by an increase in turbidity, particle count, or other contaminant). This may occur due to excessive filter run time or hydraulic surge.

Cartridge filtration – A filtration process that has a pressure vessel containing one or more filter cartridges of a specified nominal (or absolute) pore size rating used for removal of particles from water.

Catenary grid aeration – A unit process designed to transfer impurities such as volatile organic compounds and color-, taste-, and odor-causing substances from the liquid phase to the gaseous phase or to provide air (oxygen) to the liquid for oxidation. The process equipment consists of multiple, porous catenary grids in a column. There are fluidized zones between the catenary grids. Inside of the process unit, the forced air flows upward through the centenary grids, but the liquid to be treated flows downward through the catenary grids and fluidized zones forming mists. The countercurrent flow mechanism promotes the mass transfer and/or oxidation.

Cathodic protection – A corrosion control method that involves reduction or elimination of corrosion by making the metal a cathode by means of an impressed direct current or attachment to a sacrificial anode (such as magnesium, aluminum, and zinc).

Cation exchange – An ion exchange process involving the use of cation exchanger (cationic ion exchange resin), mainly for removal of calcium, magnesium, iron, and other positively charged metal ions.

Centrate – Water separated from the solids by a centrifuge.

Chemical precipitation – A chemical process in which one chemical agent precipitates another soluble chemical.

Chloramination – A disinfection process that involves the mixing of chlorine and ammonia for production of mainly monochloramine and a small amount of dichloramine, in turn, for disinfection of water storage tank and distribution system.

Chlorination – A disinfection or oxidation process involving the use of chlorine.

Chlorine (Cl_2) – A disinfectant and oxidizing agent widely used in chlorination/oxidation process for water and wastewater treatment.

Chlorine dioxide (ClO_2) – A red yellow gas that is a very strong oxidizing agent and disinfectant.

Clarifier, flotation – A large circular or rectangular tank or basin in which water is held for a period of time, during which the air bubbles with suspended solids attached float to the top by flotation. Flotation clarifiers are also called flotation basins or dissolved air flotation basins.

Clarifier, sedimentation – A large circular or rectangular tank or basin in which water is held for a period of time, during which the heavier suspended solids settle to the bottom by gravity. Sedimentation clarifiers are also called settling basins and sedimentation basins.

Coagulant – A chemical added to water that has suspended and colloidal solids to destabilize particles, allowing subsequent floc formation and removal by sedimentation, filtration, or both.

Coagulation – A process using coagulant chemicals and mixing by which colloidal and suspended materials are destabilized and agglomerated into flocs.

Coliform – A group of related bacteria whose presence in drinking water may indicate contamination by disease-causing microorganisms.

Community water system (CWS) – A water system which supplies drinking water to 25 or more of the same people year-round in their residences.

Compliance – The act of meeting all state and federal drinking water regulations.

Contaminant – Anything found in water (including microorganisms, minerals, chemicals, and radionuclides) which may be harmful to human health.

Contact clarification – A water treatment process in which flocculation and clarification (and often the rapid mix) are combined in one unit, such as an upflow solids contactor or contact clarifier.

Continuous flow sedimentation – A process by which flow is received on a continuous basis at its normal flow rate and solids are allowed to settle.

Conventional filtration treatment plant – A series of processes including coagulation, flocculation, sedimentation, and filtration resulting in substantial particulate removal.

Coprecipitation – A chemical precipitation process in which two chemical agents precipitate each other producing two precipitates.

Corrosion control – A water treatment method that keeps the metallic ions of a material, typically a pipe or tank from going into solution. Corrosion control method includes pH adjustment, addition of inorganic phosphates, addition of silicates, cathodic protection, and coatings and linings.

Corrosion inhibitors – A substance that slows corrosion by forming a protective film on the interior surface of water tanks, pipes, and process equipment.

Cryptosporidium – A disease-causing protozoan widely found in surface water sources. *Cryptosporidium* is spread as a dormant oocyst from human and animal feces to surface water. In its dormant stage, *Cryptosporidium* is housed in a very small, hard-shelled oocyst form that is resistant to chlorine and chloramine disinfectants. When water containing these cysts is ingested, the protozoan causes a severe gastrointestinal disease called cryptosporidiosis.

Curie (Ci) – A unit of radioactivity mainly used in the United States, representing the rate at which nuclear radiations are emitted. One curie equals 0.037 billion disintegrations per second or approximately the radioactivity of 1 g of radium-226. $1\ \text{Ci} = 0.037 \times 10^{12}\ \text{Bq}$.

Decant – To draw off the liquid from a basin or tank without stirring up the sediment in the bottom.

Dewatering processes – Mechanical and nonmechanical methods used to remove excess liquids from residual solids in order to concentrate the solids. These methods include belt presses, centrifuges, filter presses, vacuum presses, lagoons, and monofill.

Diatomaceous earth filtration (DEF) – A process resulting in substantial particulate removal in which (a) a precoat cake of diatomaceous earth (DE) filter media is deposited on a support membrane (septum) and (b) while the water is filtered by passing through the cake on the septum, additional filter media known as body feed is continuously added to the feed water to maintain the permeability of the filter cake.

Diffused aeration – A process involving the dissolution of air or oxygen in water by the use of coarse bubble diffusers or bubble porous disks or tubes. Typical examples of diffused aeration are (a) removing volatile organic compounds and taste-, odor-, and color-causing substances from water and (b) increasing the oxygen concentration in water for oxidation of iron and manganese.

Direct filtration – A series of processes including coagulation and filtration but excluding sedimentation resulting in substantial cost-saving but acceptable particulate removal.

Direct recycle – The return of recycle flow within the treatment process without first passing the recycle flow through treatment or equalization.

Disinfectant – Any oxidant, including but not limited to chlorine, chlorine dioxide, chloramines, and ozone added to water in any part of the treatment or distribution process, that is intended to kill or inactivate pathogenic microorganisms. A chemical (commonly chlorine, chloramine, or ozone) or physical process (e.g., ultraviolet light) that kills microorganisms such as bacteria, viruses, and protozoa.

Disinfection – A process which inactivates pathogenic organisms in water by chemical oxidants or equivalent agents.

Distribution system – A network of pipes leading from a treatment plant to customers' plumbing systems.

Disinfection by-products (DBPs) – Organic compounds formed by the reaction of the disinfectant, natural organic matter, and the bromide ion during water disinfection process. Regulated DBPs include TTHMs, HAA5s, bromate, and chlorite.

Dissolved air flotation (DAF) – A method of solids separation, whereby a side stream is saturated with air at high pressure and then injected into the flotation tank to mix with the incoming water stream. As the air bubbles rise to the surface they attach to floc particles and create a sludge layer at the surface of the tank, which is then removed for disposal.

Dual-media filtration – A water filtration system which is designed to operate at a filtration rate higher than conventional rapid granular filtration by using two different types of filter media, such as granulated black anthracite on the top and silica sand at the bottom.

Dual-membrane filtration – A membrane filtration system, also known as integrated membrane system, is composed of two different types of membrane filtration modules in series. For instance, a surface water treatment plant may use a microfiltration (MF) or ultrafiltration (UF) membrane module for particle and microorganisms removal followed by a nanofiltration (NF) or reverse osmosis (RO) module for dissolved solids removal.

Electrodialysis (ED) – A non-pressure-driven membrane process which is driven by an electrical potential difference between oppositely charged electrodes. Ions are transferred by direct electric current flow through cation and anion membranes depending on ion charges, from a less concentrated solution to a more concentrated one, producing demineralized water.

Electrodialysis reversal (EDR) – An electrodialysis process which is not driven by pressure, but driven by an electrical potential difference between oppositely charged electrodes. In the electrodialysis reversal process, the electrical polarity of the electrodes is reversed on a set time cycle thereby reversing the direction of ions in the process system for control of membrane scaling and fouling problems.

Equalization – A method used to control the flow of water or residual stream by providing storage and detention time between the point of origin and the return location of the water or residual stream. The water or residual stream is then removed from the storage unit at a controlled, uniform rate.

Filter-to-waste – The practice of discarding filter effluent that is produced during the "filter ripening" period immediately after backwash due to its impaired quality.

Filtrate – The water separated from the solids by a belt filter press or the liquid that has passed through a filter.

Filtration – A process for removing particulate matter from water by passage through porous media.

Finished water – Water that has been treated and is ready to be delivered to customers.

Floc – Collections of smaller particles that have come together (agglomerated) into larger, more settleable particles as a result of the coagulation–flocculation process.

Flocculation – A process to enhance agglomeration or collection of smaller floc particles into larger, more easily settleable particles by sedimentation (or floatable particles by flotation) through gentle stirring by hydraulic or mechanical means.

Flotation thickener – A dissolved air flotation unit that is used for thickening (concentration) of waste sludge using air bubbles. Since the combined density of air bubbles and the attached sludge is lighter than water, the sludge floats to the water surface as scum (floats) which are collected for either further dewatering treatment or disposal.

Foam separation – A dispersed air flotation process in which mechanical aeration or diffused aeration devices are used for generation of fine air bubbles, and surfactants are separated from water or wastewater by air bubbles, producing foam/scum on the water surface. The foam separation effluent water (subnatant) is withdrawn from the bottom.

Giardia lamblia – Flagellated protozoan, which is shed during its cyst stage with the feces of man and animals. When water containing these cysts is ingested, the protozoan causes a severe gastrointestinal disease called giardiasis.

Granular activated carbon (GAC) – An activated carbon which is porous and in granular form with many micro- and macro-channels and surface area, suitable for adsorption of many soluble organics and some inorganics.

Granular activated carbon adsorption – A unit process used to remove mainly dissolved organics from water, wastewater, or air. A receptacle is filled with granular carbon, and the fluid (water, wastewater, or air) is passed through either fixed or moving beds.

Gravity thickener – A sedimentation tank that is used for thickening (concentration) of waste sludge by gravity force due to the fact that the sludge solids are heavier than water. The settled sludge solids are collected for further dewatering treatment or disposal.

Greensand – A naturally occurring mineral that consists largely of greenish, granular glauconite and possesses cationic ion exchange (cation exchanger) capability. It is the base product for manufacturing manganese greensand zeolite products. Greensands are the sand media coated with manganese dioxide.

Greensand filtration – A filter bed that adsorbs soluble iron and manganese through the use of granular greensands. The greensand filter bed can be regenerated with potassium permanganate or chlorine.

Ground water (GW) – The water that systems pump and treat from aquifers (natural reservoirs below the earth's surface).

Ground water under the direct influence of surface water (GWUDI) – Any water beneath the surface of the ground with significant occurrence of insects or other macroorganisms, algae, or large-diameter pathogens such as *Giardia lamblia* or *Cryptosporidium*, or significant and relatively rapid shifts in water characteristics such as turbidity, temperature, conductivity, or pH which closely correlate to climatological or surface water conditions. Direct influence must be determined for individual sources in accordance with criteria established by the State. The State determination of direct influence must be based on site-specific measurements of water quality and/or documentation of well construction characteristics and geology with field evaluation.

Haloacetic acids (HAA5) or (HAAs) – The sum of the concentrations in milligrams per liter of the haloacetic acid compounds (monochloroacetic acid, dichloroacetic acid, trichloroacetic acid, monobromoacetic acid, and dibromoacetic acid), rounded to two significant figures after addition.

Higee aeration – It is a commercial aeration process involving the use of an air–liquid mixing device commercially known as "Higee" through which the water and the forced air are mixed and contacted, so the impurities (such as volatile organic compounds, color substances, and taste- and odor-causing substances) can be stripped from the liquid phase to the gaseous phase and/or the iron and manganese can be oxidized by the oxygen in the air. The process equipment consists of an air blower, a water pump, a water filter, and a Higee device.

Influent water – Raw water plus recycle streams.

Ion exchange (IX) – A reversible physicochemical water treatment process in which ions from an ion exchanger (i.e., an insoluble permanent solid medium or a resin) are exchanged for ions in a solution or fluid mixture surrounding the ion exchanger.

Ion exchange regenerant – A chemical solution used to restore an exhausted bed of ion exchange resins to the fully ionic (regenerated) form necessary for the desired ion exchange to again take place effectively.

Jar test – A laboratory procedure that simulates a water treatment plant's coagulation, rapid mix, flocculation, and sedimentation processes. Differing chemical doses, energy of rapid mix, energy of slow mix, and settling time can be examined. The purpose of this procedure is to estimate the minimum or optimal coagulant dose required to achieve certain water quality goals. Samples of water to be treated are commonly placed in six jars. Various amounts of a single chemical are added to each jar while holding all other chemicals at a consistent dose and observing the formation of floc, settling of solids, and resulting water quality.

Lagooning – The placement of solid or liquid material in a basin, reservoir, or artificial impoundment for purposes of storage, treatment, or disposal.

Leachate – The underflow from a dewatering unit such as a sludge drying bed or monofill.

Liquids from dewatering processes – A stream containing liquids generated from a unit used to concentrate solids for disposal.

Lime-soda softening process – A process for softening water by the addition of lime and soda ash to convert soluble calcium hardness to insoluble calcium carbonate and convert soluble magnesium hardness to insoluble magnesium hydroxide. The hardness is removed when the insoluble calcium carbonate and magnesium hydroxide are removed by clarification and filtration.

Mechanical aeration – A process of introducing air and oxygen into a liquid (water) by the mechanical action of paddle, spray, paddle wheel, or turbine mechanism.

Maximum contaminant level (MCL) – The highest level of a contaminant that is allowed in drinking water as delineated by the National Primary Drinking Water Regulations. MCLs ensure that drinking water does not pose either a short-term or long-term health risk. US EPA sets MCLs at levels that are economically and technologically feasible. Some states set MCLs which are stricter than US EPA's.

Maximum contaminant level goal (MCLG) – The level of a contaminant at which there would be no risk to human health. This goal is not always economically or technologically feasible, and the goal is not legally enforceable in places other than their residences. Some examples are schools, factories, office buildings, and hospitals which have their own water systems.

Membrane concentrate – The reject stream generated when the source water is passed through a membrane for treatment.

Membrane filtration – A filtration process (e.g., reverse osmosis, nanofiltration, ultrafiltration, and microfiltration) using tubular or spiral-wound elements that exhibits the ability to mechanically separate water from other ions and solids by creating a pressure differential and flow across a membrane with an absolute pore size <1 micron.

Microfiltration (MF) – It is one of the four pressure-driven membrane filtration processes (microfiltration, ultrafiltration, nanofiltration, and reverse osmosis) that removes ions, salts, other dissolved solids, and nonvolatile organics. In potable water treatment, microfiltration (MF) is operated at low transmembrane pressure (1–30 psi or 7–208 kPa) and is used for separation of bacteria, large microorganisms, fine solids, large flocs, large colloids, and so on as well as other bigger substances. MF member has a large pore size range (from 0.1 to 10 micron).

Microorganisms – Tiny living organisms that can be seen only with the aid of a microscope. Some microorganisms can cause acute health problems when consumed in drinking water. Also known as microbes.

Micron – A unit of length equal to 1 micrometer (μm). One millionth of a meter or one thousandth of a millimeter. One micron equals 0.00004 of an inch.

Microsand – A small-grain sand used to improve settling.

Minor streams – Waste streams that result due to spills, laboratory analyses, washdown of plant facilities, leaks, and other similar streams that are small in volume.

Monitoring – Testing that water systems must perform to detect and measure contaminants. A water system that does not follow US EPA's monitoring methodology or schedule is in violation and may be subject to legal action.

Monochloramine (NH_2Cl) – A chloramine chemical species that is produced from the reaction of chlorine and ammonia in a chloramination process. Typically monochloramine and a small amount of dichloramine ($NHCl_2$) are formed in the chloramination process as the secondary disinfectant for protection of water distribution system.

Monofill – An ultimate disposal technique for water treatment plant sludge in which the sludge is applied to a landfill for sludge only.

Multiple tray aeration – An unit process designed to transfer impurities from the liquid phase to the gaseous phase and/or to provide oxygen for oxidation reaction in the liquid. The process equipment consists of multiple staggered slat or porous trays which allow liquid to be treated to flow downward through the trays as rain drops and allow contact between a countercurrent airflow and the liquid, and thereby promoting mass transfer and/or oxidation.

Nanofiltration (NF) – It is one of the four pressure-driven membrane filtration processes (microfiltration, ultrafiltration, nanofiltration, and reverse osmosis) that removes ions, salts, other dissolved solids, and nonvolatile organics. In potable water treatment, nanofiltration (NF) is operated at high transmembrane pressure (75–149 psi or 517–1,027 kPa) and mainly used for separation of divalence and trivalence ions, hardness, DBP, NOM, *Giardia* cysts, *Cryptosporidium*, color, surfactants, and so on as well as bigger substances. NF member has very small pore size range (from 0.0005 to 0.005 micron).

Nontransient, noncommunity water system – A water system which supplies water to 25 or more of the same people at least 6 months per year.

Operating capacity – The maximum finished water production rate approved by the State drinking water program.

Ozonation – A disinfection and oxidation process for water or waste treatment using ozone.

Ozone – A gas that results from complex chemical reactions between nitrogen dioxide and volatile organic compounds; the major component of smog. Ozone at the ground level is one of the six "criteria" pollutants for which EPA has established national ambient air quality standards.

Package plant – A single compact process plant that contains all required unit processes and unit operations for water or waste treatment.

Package plant filtration – A single compact filtration plant that contains all supporting unit processes and unit operations for water filtration treatment.

Packed column aeration (PCA) – A unit process designed to transfer impurities (such as volatile organic compounds, color substances, and taste- and odor-causing substances) from the liquid phase to the gaseous phase and/or to provide air (oxygen) for oxidation reaction in the liquid. The process equipment is a hollow vertical column containing packing materials which provide surface area over which the liquid to be treated flows through. There is a contact between a countercurrent airflow and the liquid on the surface of the packing materials and the contact promotes mass transfer and/or oxidation. Also referred to as packed tower aeration.

pH – pH is an expression of the intensity of the basic or acidic condition of a solution. Mathematically, pH is the negative logarithm (base 10) of the hydrogen ion concentration, $[H^+]$. $[pH = \log (1/H^+)]$. The pH may range from 0 to 14, where 0 is most acidic, 14 most basic, and 7 neutral. Natural waters usually have a pH between 6.5 and 8.5.

Picocurie (pCi) – A unit of radioactivity mainly used in the United States representing the rate at which nuclear radiations are emitted. One picocurie (1 pCi) represents a quantity of radioactive material with an activity equal to one billionth of a curie, that is, 10^{-12} curie.

Pilot plant – A small-scale water treatment plant set up on a raw water source to determine the feasibility and impacts of a treatment scheme for a given water supply. Pilot plants are used to test alternative technologies and experiment with chemical dosages for new water treatment plants or upgrades to existing plants.

Polymer – A synthetic organic compound with high molecular weight and composed of repeating chemical units (monomers). Polymers may be polyelectrolytes (such as water-soluble flocculants), water-insoluble ion exchange resins, or insoluble uncharged materials (such as those used for plastic or plastic-lined pipe).

Polyelectrolyte – Long chained, ionic, high molecular weight, synthetic, water-soluble, organic coagulants. Also referred to as polymers.

Posttreatment – An additional treatment process following a prior treatment process. For instance, granular activated carbon filtration can be a posttreatment to rapid sand filtration.

Powdered activated carbon (PAC) – An activated carbon which is porous and in powdered form with many micro- and macro-channels and surface area, suitable for adsorption of many soluble organics and some soluble inorganics from a fluid.

Powdered activated carbon adsorption – A unit process involving the addition of powdered activated carbon to water for removal of impurities, such as taste- and color-causing substances, odor-causing substances, and volatile organic compounds.

Precipitation – (a) A chemical process in which one chemical agent precipitates another soluble chemical; (b) the total measurable supply of water received directly from clouds as rain, sleet, hail, snow, and so on.

Precoat filtration – A process that is designed to remove particulates by applying the water to be treated to a fabric or membrane module coated with very fine granular medium, such as diatomaceous earth. Precoat filtration is also called diatomaceous earth filtration.

Pressate – The water separated from the solids by a filter press.

Presedimentation – A water treatment process in which solid particles are settled out of the water in a clarifier or sedimentation basin prior to entering the treatment plant.

Pretreatment – One or more process steps occurring immediately before water enters a downstream process unit.

Public water system (PWS) – Any water system which provides water to at least 25 people for at least 60 days annually. There are more than 170,000 PWSs providing water from wells, rivers, and other sources to about 250 million Americans. The others drink water from private wells. There are differing standards for PWSs of different sizes and types.

Raw water – Water in its natural state, prior to any treatment for drinking.

Radionuclides – Radioactive, cancer-causing substances that release radiation (i.e., alpha, beta, and gamma which are radioactive isotopes or unstable forms of elements). Radionuclides include (a) radium-226, radium-228, natural uranium, radon, gross alpha, gross beta, and photon emitters; and (b) strontium-90, lead-210, polonium-210, thorium-230, and thorium-232.

Radium (Ra) – One of radionuclides regulated by the US EPA. Radium is a naturally occurring radioactive element in the form of radium-226 or radium-228, which are produced in the decay of uranium and thorium series.

Radon (Rn) – A gaseous radioactive element that derives from the radioactive decay of radium. Radon is one of radionuclides regulated by the US EPA.

Rapid sand filtration – A type of granular filter for which the granular material is sand, and the filtration rate is at least 2 gpm/ft^2 (5 m/h).

Rem – A dose equivalent unit, representing the effect of ionizing radiation on tissue. Specifically rem is a unit of dose equivalent from ionizing radiation to total body or any internal organ or organ system.

Resin – (a) An ion exchange resin product or (b) a nonionic polymeric resin product (usually in the form of specifically manufactured organic polymer beads).

Reverse osmosis (RO) – It is one of the four pressure-driven membrane filtration processes (microfiltration, ultrafiltration, nanofiltration, and reverse osmosis) that removes ions, salts, other dissolved solids, and nonvolatile organics. In potable water treatment, reverse osmosis (RO) has the highest transmembrane pressure (150–500 psi or 1,034–3,448 kPa) and is used for desalination or separation of monovalence ions, such as salts, sodium, nitrate, nitrite, and chloride, as well as bigger substances. RO member has the smallest pore size range (from 0.0001 to 0.001 micron) in comparison with MF, UF, and NF.

Recycle – The act of returning a residual stream to a plant's primary treatment process.

Recycle stream – Any water, solid, or semisolid generated by a plant's treatment processes, operational processes, and residual treatment processes that is returned to the plant's primary treatment process.

Recycle notification – Information on recycling practices that must be provided to the State by conventional and direct filtration water treatment plants that recycle spent filter backwash, thickener supernatant, or liquids from dewatering processes.

Schmutzdecke – The surface dirt cake of accumulated particulates, including a variety of living and nonliving micro- and macroorganisms, on top of a slow sand filter, that assists in turbidity removal.

Sedimentation – A process for removal of solids before filtration by gravity or separation. (Note: The Federal definition refers to the sedimentation process used in the main treatment train, but sedimentation can also be used for recycle streams.)

Sequencing batch flotation reactor (SBR-DAF) – A batch dissolved air flotation process.

Sequencing batch ion exchange reactor (SBR-IX) – A batch physicochemical process using ion exchange resins.

Sequencing batch sedimentation reactor (SBR-SED) – A batch coagulation-sedimentation process.

Sievert – A dose equivalent unit, representing the effect of ionizing radiation on tissue. One sievert equals 100 rem.

Slow sand filtration – A process involving passage of raw water through a bed of sand at low velocity (generally less than 0.4 m/h) resulting in substantial particulate removal by physical and biological mechanisms.

Sludge thickener – A tank or other piece of equipment designed to concentrate water treatment sludges.

Spent filter backwash water – A stream containing particles that are dislodged from filter media when water is forced back through a filter (backwashed) to clean the filter.

Spray aeration – An aeration process involving the use of nozzles for generation of water streams similar to a water fountain. The volatile organic compounds (VOCs) in water streams can be stripped off by air, and soluble iron and manganese can be oxidized by the oxygen in air, forming insoluble iron oxides and manganese oxides.

Streaming current – A current gradient generated when a solution or suspension containing electrolytes, polyelectrolytes, or charged particles passes through a capillary space, as influenced by adsorption and electrical double layers. This phenomenon is used in monitoring and controlling coagulation and flocculation processes.

Surface water – The water that systems pump and treat from sources open to the atmosphere, such as rivers, lakes, and reservoirs.

Suspended solids (SS) – Solid organic and inorganic particles that are held in suspension by the action of flowing water and are not dissolved.

Thickener subnatant – A stream containing the floating scum from the top of a flotation thickener that is used to treat water, solids, or semisolids from the water or wastewater treatment processes. The flotation clarified water that exits the units after particles have been allowed to float to the top is flotation thickener subnatant.

Thickener supernatant – A stream containing the decant from a gravity thickener or other unit that is used to treat water, solids, or semisolids from the water or wastewater treatment processes. The clarified water that exits the units after particles have been allowed to settle out is thickener supernatant.

Total organic carbon (TOC) – Total organic carbon in milligrams per liter measured using heat, oxygen, ultraviolet irradiation, chemical oxidants, or combinations of these oxidants that convert organic carbon to carbon dioxide, rounded to two significant figures.

Total trihalomethane precursors – Organic materials in the raw water that promote the formation of trihalomethanes.

Total trihalomethanes (TTHM) – The sum of the concentration in milligrams per liter of the trihalomethane compounds (trichloromethane [chloroform], dibromochloromethane, bromodichloromethane, and tribromomethane [bromoform]), rounded to two significant figures.

Total trihalomethanes formation potential (TTHMFP) – A measure of the ability of a water to create trihalomethanes.

Transient, noncommunity water system **–** A water system which provides water in a place such as a gas station or campground where people do not remain for long periods of time. These systems do not have to test or treat their water for contaminants which pose long-term health risks because fewer than 25 people drink the water over a long period. They still must test their water for microbes and several chemicals.

Trihalomethane (THM) – One of a family of organic compounds, named as derivatives of methane, wherein three of the four hydrogen atoms in methane are each substituted by a halogen atom in the molecular structure.

Tube settlers – Bundles of small-bore (2–3 inches or 50–75 mm) tubes installed on an incline as an aid to sedimentation. As water rises in the tubes, settling solids fall to the tube surface. As the sludge (from the settled solids) in the tube gains weight, it moves down the tubes and settles to the bottom of the basin for removal by conventional sludge collection means. Tube settlers are sometimes installed in sedimentation basins and clarifiers to improve settling of particles.

Turbidimeter – A device that measures the amount of light scattered by suspended particles in a liquid under specified conditions.

Turbidity – The cloudy appearance of water caused by the presence of suspended and colloidal matter which causes the scattering and adsorption of light. In the waterworks field, a turbidity measurement is used to indicate the clarity of water. Technically, turbidity is an optical property of the water based on the amount of light reflected by suspended particles. Turbidity cannot be directly equated to suspended solids because white particles will reflect more light than dark-colored particles and many small particles will reflect more light than an equivalent large particle.

Ultrafiltration (UF) – It is one of the four pressure-driven membrane filtration processes (microfiltration, ultrafiltration, nanofiltration, and reverse osmosis) that removes ions, salts, other dissolved solids, and nonvolatile organics. In potable water treatment, ultrafiltration (UF) is operated at medium transmembrane pressure (30–75 psi or 208–517 kPa) and is used for separation of colloids, flocs, turbidity, suspended solids, large microorganisms, and so on as well as other bigger substances. UF member has a medium pore size range (from 0.005 to 0.09 micron).

Ultraviolet (UV) – UV light has a wavelength range of 10 nanometers (the longest wavelength of X-ray) to 390 nanometers (the shortest wavelength of visible light); therefore, UV is at the invisible violet end of the light spectrum. UV light may be used as a disinfectant or to create hydroxyl radicals (OH).

Ultraviolet radiation – A disinfection process that involves the use of UV light.

Underground source of drinking water (USDW) - An underground source of drinking water is an aquifer or part of an aquifer which (1) supplies any public water system or contains a sufficient quantity of ground water to supply a public water system and currently supplies drinking water for human consumption or contains fewer than 10,000 milligrams/liter of total dissolved solids (TDS); and (2) is not an exempted aquifer. An “exempted aquifer” is part or all of an aquifer which meets the definition of a USDW but which has been exempted by the government that the aquifer does not currently serve as a source of drinking water and the aquifer cannot now and will not in the future serve as a source of drinking water

Uranium (U) – A metallic element with three natural radioactive isotopes, U-234, U-235, and U-238.

Volatile organic compounds (VOCs) – Carbon-containing compounds that easily go from a solid to a gaseous form at normal temperatures. Sources include household products such as paints, paint strippers, and other solvents; wood preservatives; aerosol sprays; cleansers and disinfectants; moth repellents and air fresheners; stored fuels and automotive products; hobby supplies; dry-cleaned clothing.

Zeta potential – The electric potential arising due to the difference in the electrical charge between the dense layer of ions surrounding a particle and the net charge of the bulk of the suspended fluid surrounding the particle. The zeta potential, also known as the electrokinetic potential, is usually measured in millivolts and provides a means of assessing particle destabilization or charge neutralization in coagulation and flocculation procedures.

Watershed – The land area from which water drains into a stream, river, or reservoir.

Wellhead protection area – The area surrounding a drinking water well or well field which is protected to prevent contamination of the well(s).

Working level (WL) – A unit used to describe the dose of radionuclides due to progeny of radon. Organ weighting factor is a factor representing relative sensitivities of organs to ionizing radiation and yielding effective dose equivalent that can be summed for all organs.

PROBLEMS/QUESTIONS

11.1 In gas transfer gases are released or desorbed from water or absorbed or dissolved by water through its exposure to the air or to other atmospheres under normal, increased, or reduced pressures. Give four water treatment examples, which depend on gas transfer.

11.2 To bring about *chemical coagulation*, floc-forming chemicals are normally added to water for the purpose of enmeshing or combining with settleable or filterable but, more particularly, with otherwise nonsettleable or nonfilterable suspended and colloidal solids to form rapidly settling and readily filterable aggregates, or flocs. Give two water treatment examples, which depend on coagulation.

11.3 In *chemical precipitation*, dissolved substances are thrown out of solution. The added chemicals are soluble and the ions released react with ions in the water to form precipitates. Give three water treatment examples, which depend on chemical precipitation.

11.4 Water is stabilized by a variety of operations in which objectionable solutes are converted into unobjectionable forms without removal. Give five water treatment examples, which depend on solute stabilization.

11.5 To permit the removal of solids from water by sedimentation, the carrying and scouring powers of flowing water, which are functions of its velocity, are reduced until suspended particles settle by gravity to the bottom of holding tanks or basins and are not resuspended by scour. Give two water treatment examples, which depend on sedimentation.

11.6 Conceptually, filtration combines straining, sedimentation, and interfacial contact to transfer suspended solids or flocs onto grains of sand, coal, or other granular materials from which the solids or flocs must generally be removed later. Give three water treatment examples, which depend on filtration.

11.7 Does the safety of water when leaving the water treatment plant guarantee its quality at the tap?

11.8 Draw a schematic of a conventional public water treatment system for treating regular surface water aiming at removal of coliforms, color and turbidity, and corrosion control. The water distribution network has two significant different pressure zones.

11.9 Recommend a complete flow diagram of a groundwater treatment plant to treat raw groundwater containing low turbidity (2 NTU) and coliforms and to treat its wastewater for recycle. All other water quality parameters of the raw groundwater meet the drinking water standards.

11.10 Write chemical equations for chemical coagulation using ferric chloride as the coagulant. Assume that natural alkalinity is available in sufficient amount as $Ca(HCO_3)_2$.

11.11 Based on your knowledge in inorganic chemistry, and the description and flow diagram of the lime-soda ash softening process in Example 11.4, write the chemical equations for the process.

11.12 Based on the description of a dual-stage MIEX process system in Section 11.6.3, draw a flow diagram for the process system.

11.13 Discuss the applications, advantages, and disadvantages of dissolved air flotation in comparison with sedimentation for water treatment and sludge thickening.

11.14 A groundwater treatment plant in the State of New York, USA, treats its raw groundwater mainly for iron and manganese removal, disinfection, and corrosion control. The water treatment plant consists of well field, pumping, chemical feeding (potassium permanganate), primary disinfection (chlorination), pressure greensand filtration, corrosion control, secondary disinfection (chlorination), clearwell, and water storage and distribution. It is discovered that its well field has been contaminated by PCBs from the Hudson River water due to infiltration through the river banks. Recommend an emergency measure for protection of public health and a permanent solution to the problems.

11.15 Figure 11.11b shows the flow diagram of a package flotation filtration plant (Lenox Water treatment Plant, Massachusetts) consisting of pumping, chemical feeding, primary disinfection, rapid mixing, flocculation, dissolved air flotation clarification, circular automatic backwash filtration (ABF), corrosion control, secondary disinfection, and clearwell. The circular filtration bed of the ABF is divided into multiple identical pie-shape cells for continuous filtration and backwash without shutdowns. A traveling bridge with a hood moves to the top of one filter cell when it is due for backwash. The hood, which is identical to the filter cell in pie shape, can move up when the backwash is not needed, or move down to cover the pie-shape filter cell for backwash, one cell at a time. ABF can be in circular or rectangular shape. Conduct an Internet search to find out the municipal water treatment plants that use circular and rectangular ABF. Draw a diagram to show how a rectangular ABF works.

11.16 Figure 11.14 illustrates the flow diagram of a conventional water treatment plant in which (a) the spent filter backwash wastewater is discharged to a holding tank for sludge settling and supernatant recycling and (b) the settled waste sludge is thickened in a gravity thickener. The thickener's supernatant is also recycled to the head of the water treatment plant, and the thickener's settled sludge is hauled away for disposal. If the volume of the thickener's settled sludge is too high to be handled and disposed of economically, further sludge dewatering may become necessary. Propose a flow diagram of a conventional water treatment plant which is similar to Fig. 11.14, except that the thickened sludge is further dewatered by a filter press or centrifuge.

11.17 What is the most efficient process system for treating raw surface water containing soluble Cd, Cr, As, Ag, and Pb? Assume all other water quality parameters of the raw water meet the drinking water standards.

11.18 What is the most efficient process system for treating a raw groundwater or raw hard surface water containing soluble Ba, Cd, Cr^{3+}, F, As, V, Ra, and Pb? Assume all other water quality parameters of the raw water meet the drinking water standards.

11.19 What is the most efficient municipal process system for treating a groundwater containing soluble barium? Assume all other water quality parameters of the raw water meet the drinking water standards.

11.20 What is the most efficient municipal process system for treating a groundwater containing soluble F, As, and Se. Assume all other water quality parameters of the raw water meet the drinking water standards.

11.21 What is the most efficient municipal process system for treating a groundwater or surface water containing soluble mercury? Assume all other water quality parameters of the raw water meet the drinking water standards.

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Chapter 12

Chemicals Feeding, Mixing, and Flocculation

12.1 INTRODUCTION

Conventional treatment is the most widely used train of processes to control microbial and turbidity levels in surface supplied drinking water. The precise order and composition of conventional treatment elements are not invariably the same, but commonly include chemical feed, rapid mix, flocculation, sedimentation, filtration, and disinfection.

A deficiency or a set of deficiencies in one or more components of conventional treatment will typically prompt an evaluation of the potential for improvement. Table 12.1 summarizes the causes of potential problems for each of the first three treatment elements: chemical feed, rapid mix, and flocculation.

The treatment processes described in this book are, in a sense, both physical and chemical operations, because they affect both the physical and the chemical composition of the waters subjected to them. There is *flocculation*, for example, when small, often colloidal, particles conjoin as a result of interparticle contact while the suspending fluid is being stirred or otherwise submitted to hydraulic shear; there is *softening* when lime and other chemicals are added to groundwater for removing the soluble hardness-contributing metals of calcium and magnesium by *precipitation*; and there is the addition of chemicals for the removal of iron and manganese also by precipitation.

12.2 HANDLING, STORING, AND FEEDING CHEMICALS

Feeding of coagulants and coagulant aids includes chemical handling, storage, measurement, and transport of the required quantities to the mixing equipment. Chemicals can be purchased and fed either in a dry form or in solution.

The physicochemical properties of the chemical will necessarily have an impact on the type of equipment needed for feeding it into the water treatment stream. Solution feeds are usually less costly and easier to maintain; while dry feeders have less equipment. The dry-feed system consists of a hopper, a measuring or proportioning system—volumetric or gravimetric, a dissolving basin, and chemical conveying lines to the proper point of application. For solution feeders, a solution of a predetermined concentration of the coagulant is prepared in dedicated storage tanks. A metering liquid feeder is used to deliver the required flow rate of the chemical solution to the point of application.

The properties of chemicals commonly used in the treatment of water and wastewater are listed in Table 12.2. Information such as this is basic to the design of handling, storage, and feeding facilities. Size and location of plant as well as available sources of supply and shipping facilities enter into decisions. In small plants with a capacity of 2.5 MGD (10 MLD) or less, chemicals may be handled satisfactorily by simple hoisting equipment and two-wheeled trucks to be (1) stored on open floors in their shipping containers and (2) moved to feeding devices. Receiving and feeding weights may be determined by simple beam scales with platforms placed at floor level for convenience. Large plants with a capacity of 2.5 MGD (10 MLD) or greater generally require mechanical or pneumatic material-handling equipment to unload dry bulk chemicals from freight cars or automotive trucks and to transport them to storage bins, similar to grain-storage bins, whence they can flow by gravity to weighing and feeding machines. In such plants liquids are pumped to storage and to feeding devices. Liquefied gases such as chlorine and ammonia are kept in factory-filled containers from which they flow under their container pressure.

With rare exceptions, the chemicals are dissolved or suspended in water before they are introduced into feed lines. Feeding devices regulate the amounts of chemicals to be added to the water or wastewater. Dry-feed machines control the dosage by the rate of volumetric or gravimetric displacement of dry chemicals. *Volumetric* and *gravimetric* machines generally plow, push, or shake the chemical off a receiving table, onto which the chemical flows from a hopper-shaped supply bin, as in a chicken-feed device. *Solution feed* depends on the regulated displacement of liquid or dry chemicals that have been dissolved or suspended in water to produce solutions of known strength or slurries of known concentration. Measurement is by constant-head orifices, swinging pipes, or regulated pumps. Pumped flow may be proportioned automatically to the rate of flow of the water or wastewater to be

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Water Engineering: Hydraulics, Distribution and Treatment, First Edition. Nazih K. Shammas and Lawrence K. Wang.

Table 12.1 Summary of operating issues in chemical feed, rapid mix, and flocculation

Treatment element	Operating issues
Chemical feed	Choice of chemical(s)
	Choice of chemical dose and pH
	Control of chemical addition; performance of chemical pumping equipment
	Maintenance of chemical feed lines
	Flexibility in feed system to allow for changing the point of addition, adding chemicals at more than one point, etc.
	Sequence of adding different chemicals
	Degree of dilution of chemicals before injection
Rapid mix	Type of rapid mix; in-line versus mechanical mix
	Number of rapid mixers
	Method of chemical addition
	Mixing speed/detention time
Flocculation	Optimum detention time
	Optimum mixing intensity
	Number of stages
	Adequate baffling to approximate plug flow conditions

(Source: U.S. EPA, 1990).

treated. *Gas feed* is generally through pressure regulators and controlled orifices as a gas or through solution-feed devices after solution in water.

The *dissolved* or *suspended chemical* is conveyed to the point of application through pressure or gravity pipelines. The lines must be resistant to attack by the chemical transported. Suitable materials are indicated in the list of chemicals commonly used in water treatment (Table 12.2). To avoid clogging and permit necessary cleaning, pipelines are liberally dimensioned, laid out in straight runs, and provided with cleanouts.

For the *carbonation* of water, CO_2 is commonly generated at the plant in a coke, oil, or gas burner. The gas may be cooled, scrubbed, and dried by passage through limestone chips and steel turnings before it is compressed and delivered to diffusers or bubblers in the carbonation chamber. A gas-flow meter measures the amount added. Carbon dioxide, ozone, chlorine dioxide, and chlorinated copperas are the only chemicals that must be produced at the site or that can ordinarily be produced there more economically than in commercial chemical plants.

Figure 12.1 shows a typical dry chemical feeder, a typical liquid chemical feed pump and a typical gas chemical feed basin. Figures 12.2 and 12.3 introduce a lime feed system and a polymer feed system, respectively. Two common liquid chemical feed systems are illustrated in Fig. 12.4. Although chlorine gas can be fed directly into a water pipe, ozone gas needs to be fed through porous diffusers (Fig. 12.1c). Figure 12.5 shows some common ozonation systems. The porous diffusers (Fig. 12.1c) can also be used for feeding carbon dioxide gas.

In plants of less than about 2.5 MGD (10 MLD), *polymer application* can be performed using a basic chemical feed system. In small plants, a polymer feed may include two tanks of equal size, a mixer, and a metering pump. One tank is used for mixing and one for storage of the polymer feed solution. The polymer mixing tank is elevated above the storage tank to permit gravity feed of the polymer solution. Optionally, a single tank suitable for water treatment operations can be interrupted to prepare the polymer solutions. A preparation of 0.25% stock polymer solution is generally used for all plant sizes.

Plants greater than 2.5 MGD (10 MLD) generally require two tanks of equal size and a mixer, but, in addition, require a dry chemical storage hopper and a dry chemical feeder.

12.2.1 Points of Chemical Addition

Flow sheets for chemicals can be elaborated in wide variety, depending on the treatment processes involved. The points of application should be kept as flexible as possible. The best situation is to have the capability to add chemicals at several points, so that the plant operator can perform tests to determine the ideal application point. Treatment methods and, with them, the points best suited to the introduction of chemicals are subject to change in step with the development of new chemicals and treatment processes, varying quality of the water or wastewater, seasonal requirements, and demands for higher standards of performance. The point of application of any chemical will have a great impact on its effectiveness. Chemicals are added at seven basic points in the water treatment process:

1. To raw water before rapid mixing
2. During rapid mixing
3. Before flocculation
4. Before sedimentation
5. Before filtration
6. After filtration
7. Prior to filter backwash
8. Prior to water storage tank and distribution system

12.2.2 Chemical Metering Equipment

Chemical metering equipment must be able to maintain accurate feed rates that can be easily adjusted according to demand. Hence, most chemical feeders used in water/wastewater treatment plants are of the positive displacement type. There are several varieties of these in the

Table 12.2 Water and wastewater treatment chemicals

Chemical	Properties
Activated carbon, C	Available as powder and granules in 35 lb (15.9 kg) bags, 5 and 25 lb (2.3 and 11.4 kg) drums, and carloads; dusty, arches in hoppers; weighs 8–28 lb/ft^3 (128–449 kg/m^3) averaging 12 lb/ft^3 (192 kg/m^3); fed dry or as slurry of 1 lb/gal (0.12 kg/L) maximum; handled dry in iron or steel, wet in stainless steel, rubber, and bronze
Activated silica, SiO_2	Silica sol, produced at site from sodium silicate, Na_2SiO_3, and alum, ammonium sulfate, chlorine, carbon dioxide, sodium bicarbonate, or sulfuric acid; sodium silicate is diluted to 1.5% SiO_2 before activation; insoluble, will gel at high concentrations; fed as 0.6% solution to prevent gel formation; handled in iron, steel, rubber, and stainless steel
Alum, aluminum sulfate, $Al_2(SO_4)_3{\cdot}18H_2O$	Alum, filter alum, sulfate of alumina; available in ground, rice, powder, and lump form in 100 and 200 lb (45.4 and 90.8 kg) bags, 325 and 400 lb (147.6 and 181.6 kg) barrels, 25, 100, and 250 lb (11.4, 45.4, and 113.5 kg) drums, and carloads; dusty, astringent, only slightly hygroscopic; weighs 60–75 lb/ft^3 (961–1,202 kg/m^3); should contain at least 17% Al_2O_3; fed dry in ground and rice form; maximum concentration 0.5 lb/gal (0.06 kg/L); handled dry in iron, steel, and concrete, wet in lead, rubber, asphalt, cypress, and stainless steel, etc.; available also as 50% solution
Anhydrous ammonia, NH_3	Available in steel cylinders containing 50, 100, and 150 lb (22.7, 45.4, and 68.1 kg), also in 50,000 lb (22,700 kg) tank cars; pungent, irritating odor, liquid causes burns; 99–100% NH_3; fed as dry gas and aqueous solution through gas feeder or ammoniator; handled in iron, steel, glass, and nickel
Calcium oxide, CaO	Quicklime, burnt lime, chemical lime, unslaked lime; available as lumps, pebbles, crushed or ground, in 100 lb (45.4 kg), moisture-proof bags, wooden barrels, and carloads; unstable, caustic, and irritating; slakes to calcium hydroxide with evolution of heat when water is added; weighs 55–70 lb/ft^3 (881–1,121 kg/m^3); should contain 70–90% CaO; best fed dry as 3/4 in. (19 mm) pebbles or crushed to pass 1 in. (25.4 mm) ring; requires from 0.4 to 0.7 gal (1.5–2.6 L) of water for continuous solution; final dilution should be 10%; should not be stored for more than 60 days even in tight container; handled wet in iron, steel, rubber hose, and concrete; clogs pipes
Calcium hydroxide, $Ca(OH)_2$	Hydrated lime, slaked lime; available as powder in 50 lb (22.7 kg) bags, 100 lb (45.4 kg) barrels, and carloads; must be stored in dry place; caustic, dusty, and irritant; weighs 35–50 lb/ft^3 (567–801 kg/m^3); should contain 62–74% CaO: fed dry, 0.5 lb/gal (0.06 kg/L) maximum, and as slurry 0.93 lb/gal (0.11 kg/L) maximum; handled in rubber hose, iron, steel, asphalt, and concrete; clogs pipes
Calcium hypochlorite, $Ca(OC1)_2{\cdot}4H_2O$	Available as powder, granules, and pellets in 115 lb (52.2 kg) barrels, 5, 15, 100, and 300 lb (2.27, 6.81 and 45.4 and 136.2 kg) cans, and 800 lb (363.2 kg) drums; corrosive and odorous; must be stored dry; weighs 50–55 lb/ft^3 (801–881 kg/m^3); should contain 70% available $C1_2$; fed as solution up to 2% strength (0.25 lb/gal or 0.03 kg/L); handled in ceramics, glass, plastics, and rubber-lined tanks.
Chlorine, $C1_2$	Chlorine gas, liquid chlorine; available as liquefied gas under pressure in 100 and 150 lb (45.4 and 68.1 kg) steel cylinders, ton containers, cars with 15 ton (13.6 metric ton) containers, and tank cars of 16, 30, and 55 ton (14.5, 27.2, and 49.9 metric ton) capacity; corrosive, poisonous gas; should contain 99.8% $C1_2$; fed as gas vaporized from liquid and as aqueous solution through gas feeder or chlorinator; dry liquid or gas handled in black iron, copper, and steel, wet gas in glass, silver, hard rubber, and tantalum
Copper sulfate, $CuSO_4{\cdot}5H_2O$	Available ground and as powder or lumps in 100 lb (45.4 kg) bags and 450 lb (204.3 kg) barrels or drums; poisonous; weighs 75–90 lb/ft^3 (1,202–1,442 kg/m^3) ground, 73–80 lb/ft^3 (1,169–1,282 kg/m^3) as powder, and 60–64 lb/ft^3 (972–1,025 kg/m^3) as lumps; should be 99% pure; best fed ground and as powder; maximum concentration 0.25 lb/gal (0.03 kg/L); handled in stainless steel, asphalt, rubber, plastics, and ceramics
Ferric chloride, $FeCl_3$ (anhydrous and as solution)	$FeC1_3{\cdot}6H_2O$ (crystal); chloride of iron, ferrichlor; available as solution, lumps, and granules in 5 and 13 gal (18.9 and 49.2 L) carboys and in tank trucks; hygroscopic; solution weighs 11.2–12.4 lb/gal (1.3–1.5 kg/L), crystals 60– 64 lb/ft^3 (961–1,025 kg/m^3), anhydrous chemical 85–90 lb/ft^3 (1,362–1,442 kg/m^3); solution should contain 35–45%, crystals 60%, and anhydrous chemical 96–97% $FeC1_3$; fed as solution containing up to 45% $FeC1_3$; handled in rubber, glass, ceramics, and plastics
Ferric sulfate, $Fe_2(SO_4)_3{\cdot}3H_2O$ and $Fe_2(SO_4)_3{\cdot}2H_2O$	Ferrifloc, Ferriclear, iron sulfate; available as granules in 100 lb (45.4 kg) bags, 400 and 425 lb (181.6 and 192.9 kg) drums, and carloads; hygroscopic, must be stored in tight containers; weighs 70–72 lb/ft^3 (1,121–1,153 kg/m^3); $Fe_2(SO_4)_3{\cdot}3H_2O$ should contain 18.5% Fe, $Fe_2(SO_4)_3{\cdot}2H_2O$ should contain 21% Fe; best fed dry, 1.4–2.4 lb/gal, detention time 20 min; handled in stainless steel, rubber, lead, and ceramics

(*continued*)

Table 12.2 (*Continued*)

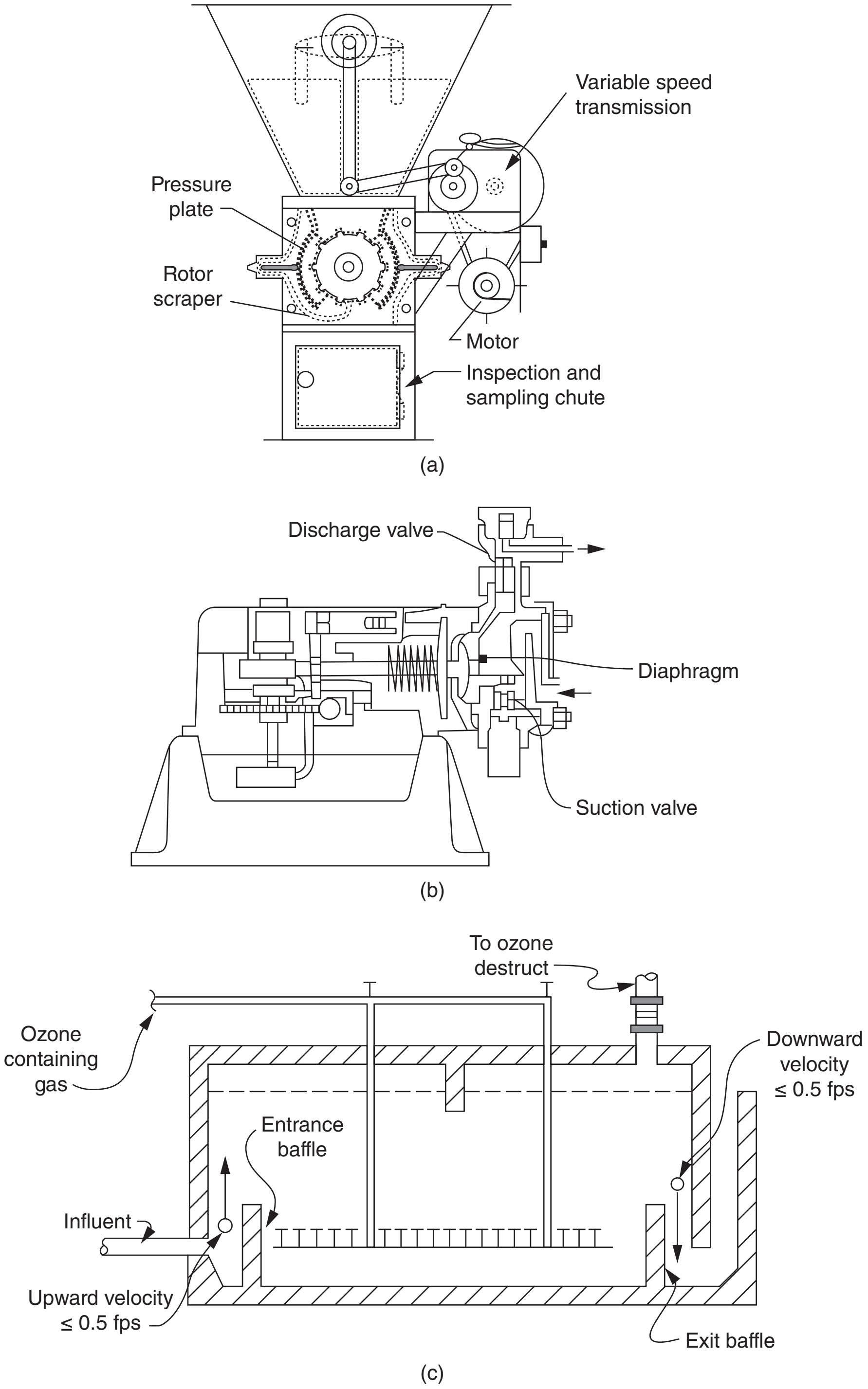
Chemical	Properties
Ferrous sulfate, $FeSO_4 \cdot 7H_2O$	Copperas, iron sulfate, sugar sulfate, green vitriol; available as granules, crystals, powder, and lumps in 100 lb (45.4 kg) bags, 400 lb (181.6 kg) barrels, and bulk; weighs 63–66 lb/ft^3 (1,009–1,057 kg/m^3); should contain 20% Fe; best fed as dry granules, 0.5 lb/gal (0.02 kg/L), detention time 5 min; handled dry in iron, steel, and concrete, wet in lead, rubber, iron, asphalt, cypress, and stainless steel
Sodium carbonate, Na_2CO_3	Soda ash; available as crystals and powder in 100 lb (45.4 kg) bags, 100 lb (45.4 kg) barrels; 25 and 100 lb (11.4 and 45.4 kg) drums, and carloads; weighs 30–65 lb/ft^3 (481–1,041 kg/m^3); extra light to dense; should contain 58% Na_2O; best fed as dense crystals 0.25 lb/gal (0.03 kg/L), detention time 10 min, more for higher concentrations; handled in iron, steel, and rubber
Sodium chloride, NaCl	Common salt, salt; available as rock, powder, crystals, and granules in 100 lb (45.4 kg) bags, barrels, 25 lb (11.4 kg) drums, and carloads; rock weighs 50– 60 lb/ft^3 (801–961 kg/m^3), fine weighs 58–70 lb/ft^3 (929–1,121 kg/m^3); should contain 98% NaCl; fed as saturated brine; handled in galvanized iron and rubber
Sodium fluoride, NaF	Fluoride; available as granules (crystals) and powder in 100 lb (45.4 kg) bags, 25, 125, and 375 lb (11.4, 56.8, and 170.3 kg) drums; powder weighs 66 –100 lb/ft^3 (1,057–1,602 kg/m^3); granules weigh 90–106 lb/ft^3 (1,442–1,698 kg/m^3); should contain 43–44% F; best fed as granules 1 lb (0.454 kg) to 12 gal (45.4 L); handled dry in iron and steel, wet in rubber, plastics, stainless steel, asphalt, and cypress
Sodium hypochlorite, NaOCl	A 12% solution is widely used in waterworks for the chlorination of water, and a 15% solution is more commonly used for disinfection of wastewater in treatment plants. Sodium hypochlorite can also be used for point-of-use disinfection of drinking water
Sodium silicate, $Na_2O(SiO_2)_{3.25\ \mathrm{approx}}$	Water glass; available as liquid in 1, 5, and 55 gal (3.8, 18.9, and 211.8 L) drums, ton trucks, and tank cars; weighs about 11.7 lb/gal (1.4 kg/L); should contain about 9% Na_2O and 29% SiO_2; fed in solution as received; handled in cast iron, steel, and rubber
Sulfur dioxide, SO_2	Available as liquefied gas under pressure in 100, 150, and 200 lb (45.4, 68.1, and 90.8 kg) steel cylinders; 100% SO_2; fed as gas, handled dry in steel, wet in glass, rubber, and ceramics

market. Positive displacement pumps are used for solution feeders while screw, vibrating trough, rotary, and belt-type gravimetric feeders are used for dry-feed systems.

1. *Positive displacement pumps*. These include the plunger pump, the gear pump, and the diaphragm pump. All three varieties produce a constant chemical flow rate for a predetermined specific pump setting. However, the plunger pump is most widely used because of its accuracy and ease of adjusting the piston stroke. The rate of chemical output is easily calculated from the fixed volume of discharge per stroke and the number of strokes per minute.
2. *Screw feeder*. The screw feeder is located directly below the hopper. The unit maintains the desired chemical dosage by adjusting the speed and the duration of time the screw rotates as it delivers the chemical to the discharge point.
3. *Vibrating trough feeder*. The vibrating trough controls its rate of chemical delivery by the magnitude and the time interval of vibration.
4. *Rotary feeder*. The rotary feeder receives its chemical input from a hopper located above the rotating gear of the feeder. The gear has teeth that maintain a fixed amount of chemical between them. The chemical discharge rate is controlled by the speed of the rotor and its running duration.
5. *Belt-type gravimetric feeder*. A balance and a vibrating trough allow this feeder to maintain a constant weight of chemical on a moving belt. The rate of chemical feed is controlled by the amount of chemical on the belt and the speed and duration of belt travel.

When selecting a chemical feeder, the following factors should be taken into consideration:

1. Sufficient capacity of operating range for present and future expected feeding rates
2. Accuracy of the unit in maintaining uniform feeding rates
3. Repeatability of the unit when reverting to a previous setting
4. Ease and difficulty of calibration, operation, and maintenance
5. Resistance of the system to corrosion
6. Provision for dust suppression
7. Availability of reasonably priced spare parts
8. Safety consideration in operation and maintenance
9. Length of unit useful life
10. The fixed initial cost and the yearly cost of operation and maintenance



(a)

(b)

(c)

Figure 12.1 (a) Typical positive displacement rotary feeder for feeding dry chemicals; (b) typical positive displacement pumps for feeding liquid chemicals; and (c) typical contact basin using porous diffusers for feeding gaseous chemicals (Source: U.S. EPA). Conversion factor: 1 fps = 1 ft/s = 0.3048 m/s

12.3 RAPID MIXING

Rapid mixing is generally the first stage of the treatment process (see Fig. 12.6). The purpose of this step is to achieve instantaneous, uniform dispersion of the chemicals through the water body. Instantaneous flash mixing is not only sufficient, but is desirable because

1. The production of effective coagulant species depends greatly on being able to achieve a uniform dispersion of the added chemicals,
2. Rates for the formation of monohydroxocomplexes and other hydroxometal polymers are very rapid, and
3. The adsorption rate for the various coagulant products is also very fast.

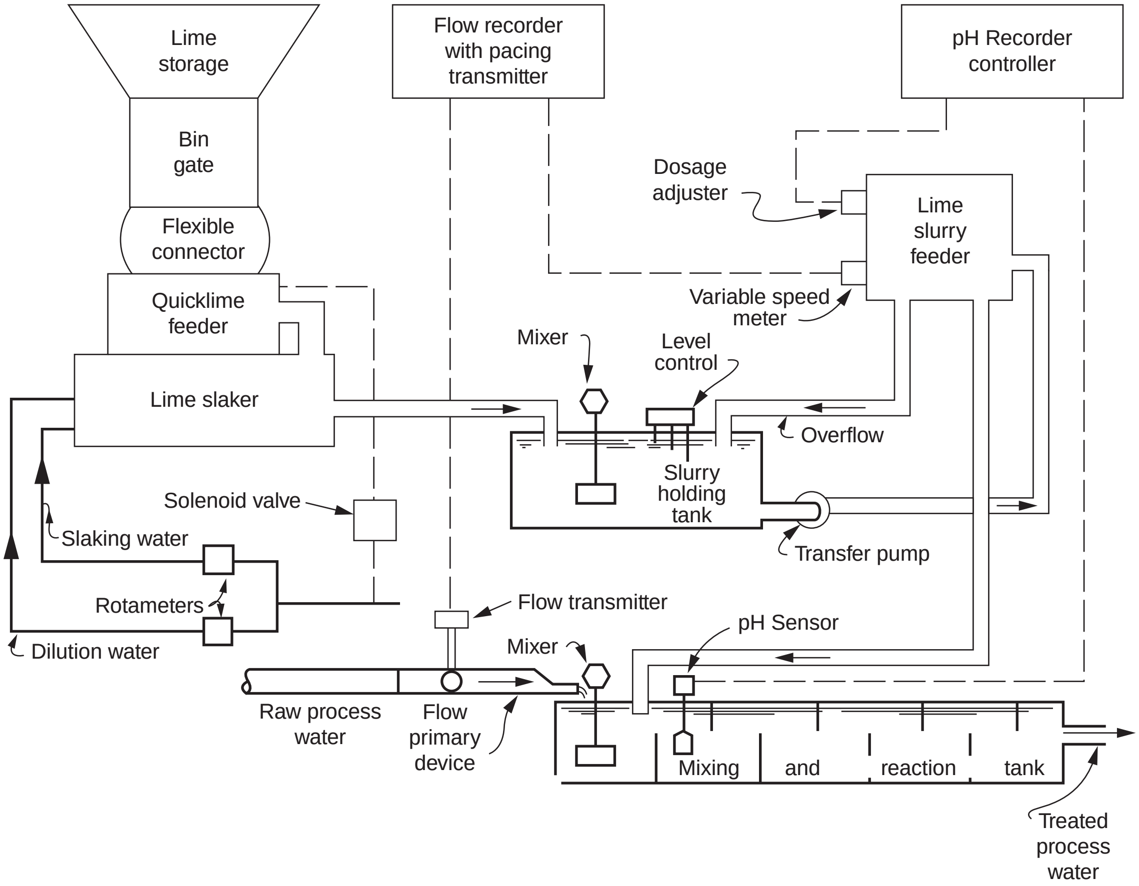


Figure 12.2 Typical complete feed system for storage, preparation, and feeding of dry chemicals (Source: U.S. EPA).

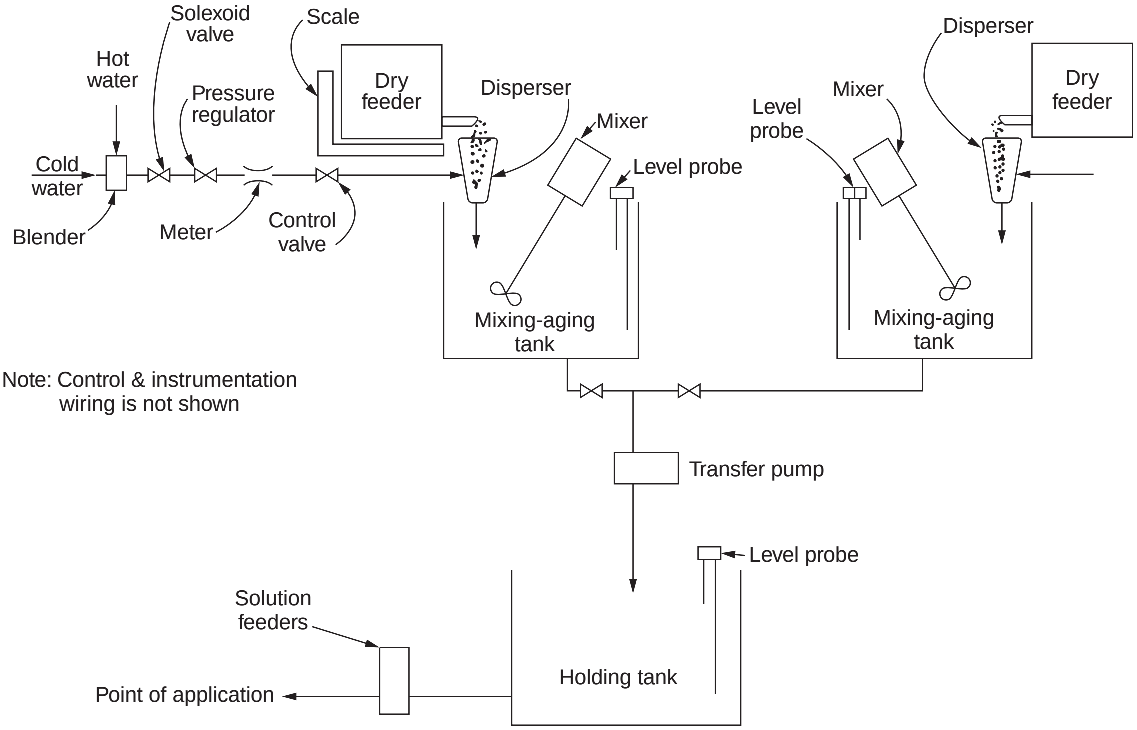


Figure 12.3 Typical complete feed system for preparation and feeding of polymers (Source: U.S. EPA).

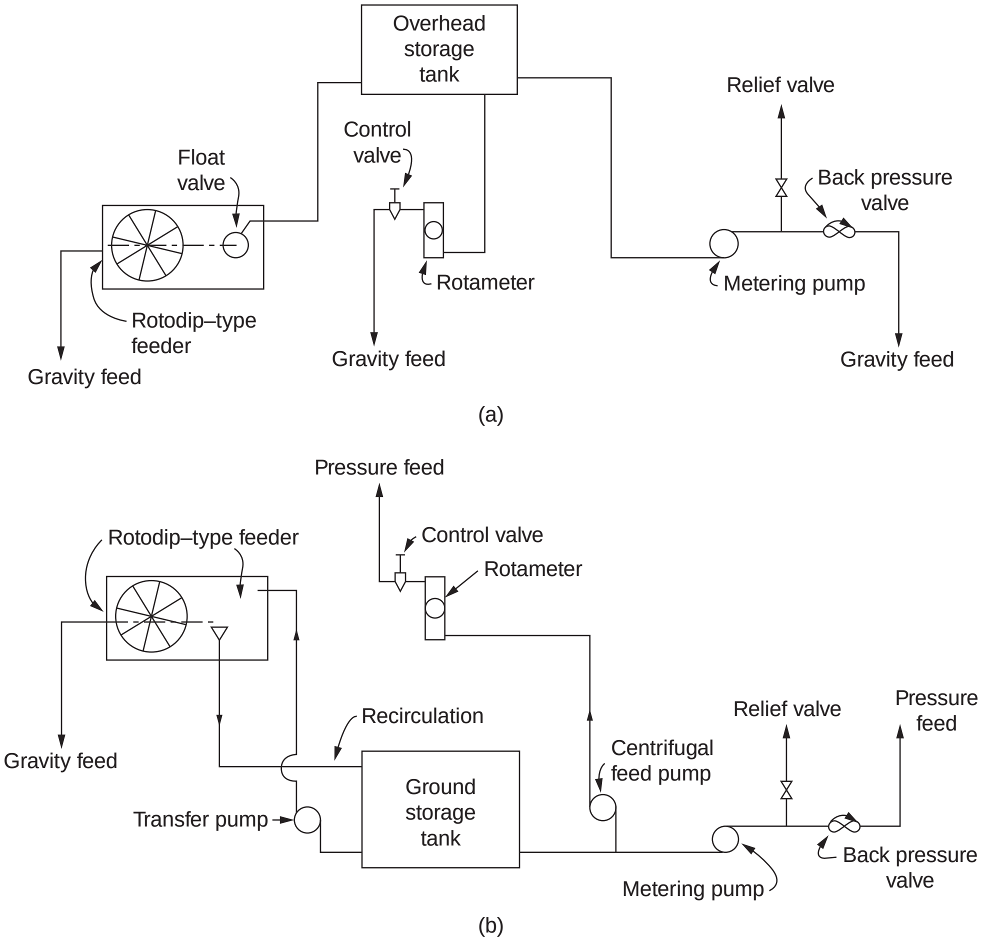


Figure 12.4 (a) Alternative liquid feed systems for overhead storage and (b) alternative liquid feed systems for ground storage (Source: U.S. EPA).

Extended mixing times are generally unwarranted, since these reactions will be completed in less than 1 second. However, when dealing with fragile colloidal particles, it may be prudent to achieve the required dispersion through a less intense mixing over a longer time interval.

The efficiency of a rapid mix is based on the power imparted to the water, which is measured in terms of the velocity gradient G and the contact time t. Recommended detention times range from less than a second up to 1 minute and G values from 700 to 4,000 s^{-1}. Table 12.3 shows typical design values for G and t. In many instances the product of G and t, Gt, is used for the design of rapid-mix units. Recommended Gt values can range from 10,000 to 40,000. Furthermore, some researchers have recommended an empirical relationship that relates G to the coagulant concentration in addition to contact time:

$$GtC^{1.46} = 5.9 \times 10^6 \tag{12.1}$$

where

C = coagulant concentration, mg/L

The types of devices used for rapid mixing include mechanical mixers, static mixers, jet injection mixers, and coagulant diffusers.

12.3.1 Mechanical Mixers

The most common mixing devices are mechanical mixing tanks, also termed completely mixed or back-mixed units. They use turbines or impellers to mix coagulant with water. The three advantages of mechanical mixers are that they (1) are effective, (2) impart little head loss (head loss being loss of pressure or the reduction of the water velocity within the plant), and (3) are unaffected by the volume of water and flow variations. Conventional rapid-mixing tanks are usually designed for a detention time of 10–60 s with velocity gradients of 700–1,000 s^{-1}. Power requirement ranges between 0.9 and 1.2 hp/MGD (0.18–0.24 kW/MLD).

A variable-speed drive can alter mixer speeds to achieve different velocity gradients. Lower velocity gradients are used following the application of polyelectrolyte coagulant aids. Labor is required for daily jar testing operations and

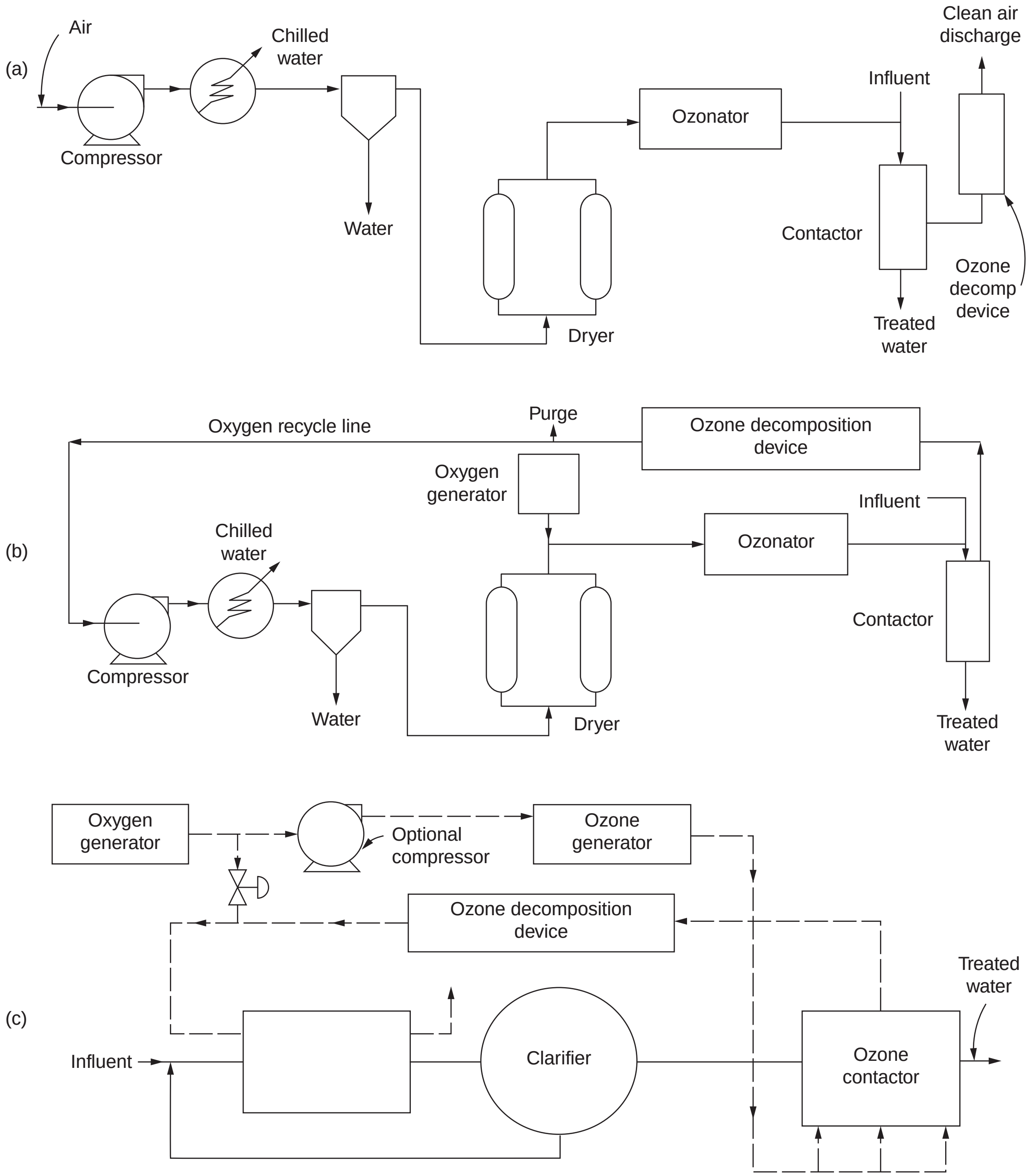


Figure 12.5 Alternative ozone feed systems: (a) once-through air process system; (b) oxygen recycle process system; and (c) once-through oxygen process system (Source: U.S. EPA).

routine inspections, as well as annual inspections, cleaning, tank drainage, and oil changes.

12.3.2 In-Line Static Mixers

In-line static mixers consist of a series of baffle elements placed in a pipe section to impart alternating changes in flow direction and intense mixing action as water flows through the device. Head loss ranging from 1 to 6 ft (0.3–1.8 m) occurs as the flow passes through the static mixer. This significant head loss requires an extra power input of 0.5–1.0 hp/MGD (0.10–0.20 kW/MLD). Static mixers achieve virtually instantaneous mixing, are relatively maintenance free, and are less expensive than other rapid-mixing processes. The only

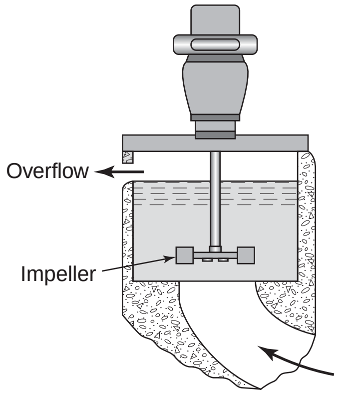


Figure 12.6 Rapid mixer, Blacksburg Christiansburg, VPI water authority, http://www.h2o4u.org/tour/flashmixer.shtml (After Fair et al., 1971).

disadvantage of in-line static mixers is that mixing intensity depends upon the plant's water flow rate. As rates decrease, the mixing intensity slows.

12.3.3 In-Line Mechanical Blenders

In-line mechanical blenders provide rapid mixing of treatment chemicals with water flowing in a pipeline under pressure. These devices consist of a housed propeller driven by an electrical motor. Typical velocity gradients generated in these devices are 3,000–5,000 s^{-1} with residence times of 0.5–1.0 s. They have the advantage over static mixers of imparting considerably less head loss and not being affected by change in flow. An in-line mechanical blender with residential time of 0.5 s requires 0.5 water hp (0.37 kW) per MGD (43.8 L/s) of water flow. Head losses in the manufactured in-line mechanical blenders range from 1 to 3 ft (0.3 to 0.9 m). In addition, in-line blenders offer the following advantages:

1. Virtually instantaneous mixing with a minimum of short-circuiting
2. Minimal head loss
3. Less expense than more conventional rapid-mix units

Table 12.3 Rapid mixing contact time and velocity gradient

Contact time *t* (s)	Velocity gradient, *G* (s^{-1})
0.5–1.0 (in-line blenders)	4,000
10–20	1,000
21–30	900
31–40	800
41–60	700

12.3.4 Jet Injection Blending

Jet injection mixers, a type of in-line blender, are used for attaining nearly instantaneous dispersion of coagulant with raw water, usually at larger treatment plants. Jet injectors can avoid the backmixing inefficiencies of turbine or impeller mixers, and the recommended detention times are shorter than for mechanical mixers. The primary disadvantages are that the orifices in the injection pipe tend to become plugged and the mixing intensity cannot be varied. Figure 12.7 illustrates a jet injection mixer.

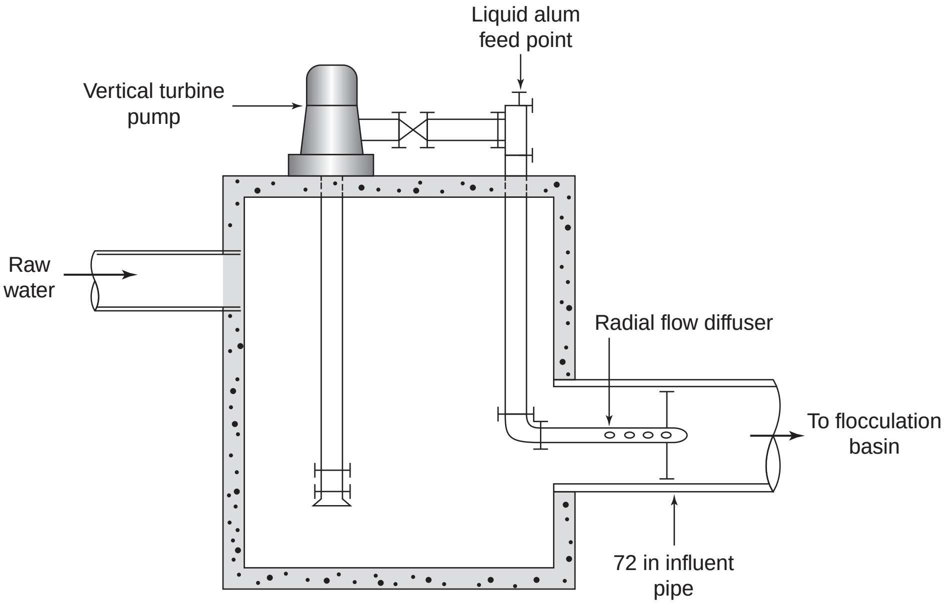


Figure 12.7 In-line jet injection mixer (After U.S. EPA, 1990). Conversion factor: 1 in. = 2.54 cm.

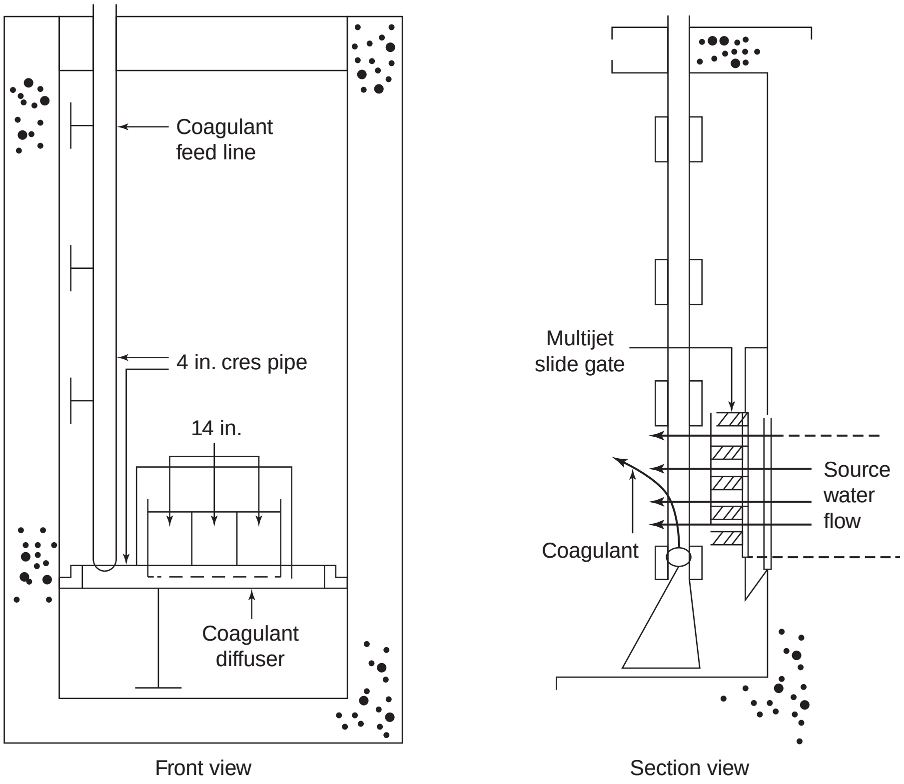


Figure 12.8 Coagulant diffuser (After U.S. EPA, 1990). Conversion factor: 1 in. = 2.54 cm.

12.3.5 Coagulant Diffusers

Coagulant diffusers are also used to improve the rapid-mixing process. Figure 12.8 is a schematic diagram of a coagulant diffuser. Coagulant diffusers are similar to jet injection mixers in design, except that jet injectors are usually used in a pipe setting and coagulant diffusers are typically used in basin settings. Coagulant diffusers are designed to apply the coagulant at the point of maximum turbulence. In some plants, multiple application points spaced uniformly across the flow cross-section permit rapid and thorough contact of chemical solution with the entire incoming water flow. This rapid and uniform dispersion of coagulant prevents its hydrolysis, which is a common problem in systems with single application points. Where turbulence in incoming channels or pipelines provides good mixing, the simple addition of a coagulant diffuser could improve coagulation and lead to improved filtered water quality.

12.3.6 Hydraulic Jumps

A hydraulic jump can be easily formed downstream of the *Parshall flume* (a device for flow rate measurement) at the inlet to the water treatment plant. This can be done with very low expense by a sudden drop in the bottom of the flume effluent channel. Although the operational flexibility is rather limited, it has the advantage of not using any mechanical equipment and economy. Typical velocity gradient, G, and contact time values are 800 s^{-1} and 2 s, respectively.

12.4 RAPID MIXING AND SLOW FLOCCULATION

The rapid mixing and flocculator systems, while similar in their elements, have distinct functions that lead to specific design parameters for each system. The goal of the rapid mixer is to, as the name implies, rapidly, uniformly, and efficiently distribute the chemicals into the water. "G" (velocity gradient) values from 700 to 1,000 s^{-1} are typical with relatively short hydraulic residence times, typically 30–45 s.

The goal of the flocculator mixer is to uniformly mix the coagulated water to promote the formation of a floc that can be subsequently removed through filtration, flotation, or settling. The flocculator design of choice today is the conventional vertical impeller (see Fig. 12.9). Horizontal systems with their inherent maintenance problems are being phased out and retrofitted. During the early stages of flocculation the objective is to capture colloidal particles with the added chemicals introduced in rapid mixing. This will typically be accomplished at G values of between 70 and 100 s^{-1}. Table 12.4 presents contact times and Gt values for flocculation. During the latter stages of the process, the goal is to "grow" the floc size to enhance ultimate removal

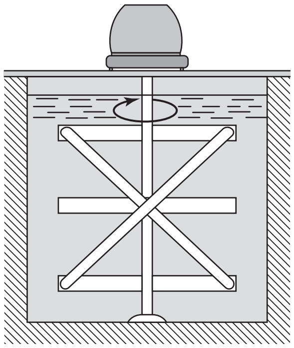


Figure 12.9 Flocculators, Blacksburg Christiansburg, VPI water authority, http://www.h2o4u.org/tour/flocculators.shtml (After Fair et al., 1971).

downstream. This requires the delicate balance between mixing and floc formation.

12.5 FLOCCULATION

The agglomeration of particles is a function of their rate of collisions. The function of flocculation is to optimize the rate of contact between the destabilized particles, hence increasing their rate of collision and bringing about the attachment and aggregation of the particles into larger floc. The floc may then float to the top of the liquid, settle to the bottom of the liquid, or can be readily filtered from the liquid.

A combination of mixing and stirring or agitation that produces aggregation is called *flocculation,* even though more specific terms such as coagulation and thickening may be preferred by physical chemists and chemical engineers to identify the origin of the particulates concerned. *Mixing* is the specific blending, mingling, or commingling of coagulating chemicals or materials with water or wastewater in order to create a more or less homogeneous single- or multiple-phase system, whereas *stirring* describes the disturbing of the flow pattern of a fluid in a mechanically orderly way for the purpose of effecting a dynamic redistribution of particles. Random rather than orderly turbulence is distinguished by the term *agitation.*

Generally speaking, mixing is a brief operation seeking a quick response, often in advance of stirring or agitation, whereas stirring and agitation are more protracted operations normally aiming at the conjunction of suspended particles or flocs, but sometimes intended to break up large flocs in order to maintain particle numbers. Aggregation of particles is referred to as *floc growth*; breakup of flocs, as *floc shear*.

Table 12.4 Typical G and Gt values for flocculation

Type	G (s^{-1})	Gt
Low turbidity, color removal coagulation	20–70	60,000–200,000
High turbidity, solids removal coagulation	50–150	90,000–180,000
Softening, 10% solids	130–200	200,000–250,000
Softening, 30% solids		300,000–400,000

Flocculation is a time-dependent, slow process that directly affects clarification efficiency by providing multiple opportunities for particles suspended in water to collide through gentle and prolonged agitation. The process takes place in a basin equipped with a mixer that provides gentle agitation. This agitation must be thorough enough to encourage interparticle contact but gentle enough to prevent disintegration of existing flocculated particles. Effective flocculation is important for the successful operation of the sedimentation process.

Proper flocculation requires long, gentle mixing. Mixing energy must be high enough to bring coagulated particles constantly into contact with each other, but not so high as to break up those particles already flocculated. Induced velocity in floc chambers should vary from 2 ft/s (0.6 m/s) in the first stage to 0.25 ft/s (0.076 m/s) in the last stage. Velocity through openings in the baffle should be slightly less than the induced velocities.

If water passes through the flocculation basin in much less time than the volumetric residence time, the influent stream has *short-circuited.* Inlet and outlet turbulence is often the major source of destructive energy in flocculation basins that contributes to short-circuiting. Baffles are used in flocculator basins to direct the movement of water through the basin. Baffling near the basin inlet and outlets improves basin circulation and achieves more uniform circulation.

12.6 MIXING AND STIRRING DEVICES

The sources of power for flocculating devices are gravitational, pneumatic, or mechanical. Generally speaking, mechanical and pneumatic devices are relatively flexible in power input; gravitational devices, relatively inflexible. Because of this, gravitational devices are seldom included in large plants, even though they may possess quite useful features.

12.6.1 Baffled Channels

Baffled channels are prime examples of gravitational mixing and stirring devices. Most other gravitational devices are relatively inefficient. Baffled channels differ from unobstructed open channels and, for that matter, from pipelines in that shear gradients or turbulence are not merely functions of frictional resistance to flow. Velocity gradients (G values) are purposely intensified by induced changes in the direction of flow (Fig. 12.10). For baffled channels of capacity V, in

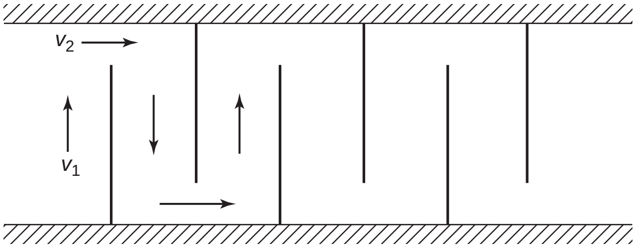


Figure 12.10 Schematic diagram of baffled channel—plan of round-the-end baffles or vertical section of over-and-under baffles (After Fair et al., 1971).

which a loss of head h is incurred when the rate of flow is Q, the useful power input P is

$$P = Q(\rho g)h \tag{12.2}$$

where

ρg is the weight density or specific weight of water = 62.4 lb/ft^3 (9,800 N/m^3) $\rho g = \gamma$.
Q = flow, ft^3/s (m^3/s)
h = loss of head, ft (m)
P = power, ft-lb/s (N-m/s)

Velocity gradient, G, is an important parameter for designing a mixing unit:

$$G^2 = Q\gamma h/\mu V = \gamma h/\mu t \tag{12.3a}$$

$$G = \sqrt[2]{P/\mu V} \tag{12.3b}$$

where

G = velocity gradient, s^{-1}
P = power input, W or J/s or N-m/s (ft-lb/s)
V = volume of mixing basin, m^3 (ft^3)
μ = dynamic (absolute) viscosity of water, Pa-s, N-s/m^2 (lb-s/ft^2)
t = time (s)

Each foot (0.3048 m) of lost head is 62.4 × 1.547/550 = 0.175 hp/MGD or 62.4 × 1.547/737.6 = 0.131 kW/MGD (0.035 kW/MLD). In practice, head losses commonly lie between 0.5 and 2 ft (0.15 and 0.61 m), velocities vary from 0.5 to 1.5 ft/s (0.15–0.46 m/s), and detention times run from 10 to 60 min. For $(n - 1)$ equally spaced *over-and-under* or *around-the-end* baffles and for velocities v_1 and v_2 in the channels and baffle slots, respectively, the loss of head approaches h as in Eq. (12.4) in addition to normal channel friction:

$$h = nv_1^2/2g + (n - 1)v_2^2/2g \tag{12.4a}$$

where

g = the gravity constant, m/s^2 (ft/s^2)
v_1 = velocity 1 shown in Fig. 12.10, m/s (ft/s)
v_2 = velocity 2 shown in Fig. 12.10, m/s (ft/s)

In arriving at this estimate the assumption is made that necessary velocities must be redeveloped at each change in direction of flow.

The head loss can also be approximated by the following relationship, where n is equal to the number of partitions and $K = 2$ (common values of K are 2–4 depending on type of bend):

$$h = (n - 1)Kv^2/2g \tag{12.4b}$$

12.6.2 Pneumatic Mixing and Stirring

When air is injected or diffused into water after suitable compression, it normally expands isothermally. Accordingly, the work done by the air is $\int p\,\mathrm{d}V$, where p is the absolute pressure intensity and V is the volume of air. Because pV is constant (e.g., $p_\mathrm{a}V_\mathrm{a}$),

$$V_\mathrm{a}\int_{V_\mathrm{c}}^{V_\mathrm{a}} \mathrm{d}V/V \; = p_\mathrm{a}V_\mathrm{c}\ln(V_\mathrm{a}/V_\mathrm{c}) = p_\mathrm{c}V_\mathrm{a}\ln(p_\mathrm{c}/p_\mathrm{a})$$

where the subscripts a and c denote free or atmospheric conditions and compressed conditions, respectively. If, for example, Q_a ft^3/min of free air is injected into water from a diffuser situated h ft below the water surface, the power dissipated usefully by the rising air bubbles is essentially P, ft-lb/s:

$$P = (14.7 \times 144 \times 2.303/60)\,Q_\mathrm{a}\log[(h + 34)/34]\text{ft-lb/s}$$

$$P = 81.5Q_\mathrm{a}\log[(h + 34)/34] \quad \text{(US customary units)} \tag{12.5a}$$

The equivalent SI equation is

$$P = 3{,}905Q_\mathrm{a}\log[(3.28h + 34)/34] \quad \text{(SI units)} \tag{12.5b}$$

where P = power dissipated, N-m/s or W; Q_a = free air flow, m^3/min; and h = loss of head, m.

The allowable loading Q/V of an aerated flocculating tank or channel is found by substituting Eq. (12.5) in Eq. (12.2) and simplifying the resultant mathematical expression.

Compressed air is diffused into treatment units also in aeration for gas exchange, in cleaning granular filters by air scour, and in aerating and stirring activated-sludge units.

12.6.3 Mechanical Mixing and Stirring

The impellers employed in mechanical mixing and stirring generate both mass flow and turbulence. Three types are in common use: (1) paddles, (2) turbines, and (3) propellers.

Paddles consist of blades attached directly to vertical or horizontal shafts (Figs. 12.11c and 12.11d). The moving blades (rotors) may be complemented by stationary blades (stators) that oppose rotational movement of the entire mass of water within the treatment unit and help to suppress vortex formation. However, stators are not often used in water or wastewater treatment practice. Paddles are rotated at slow

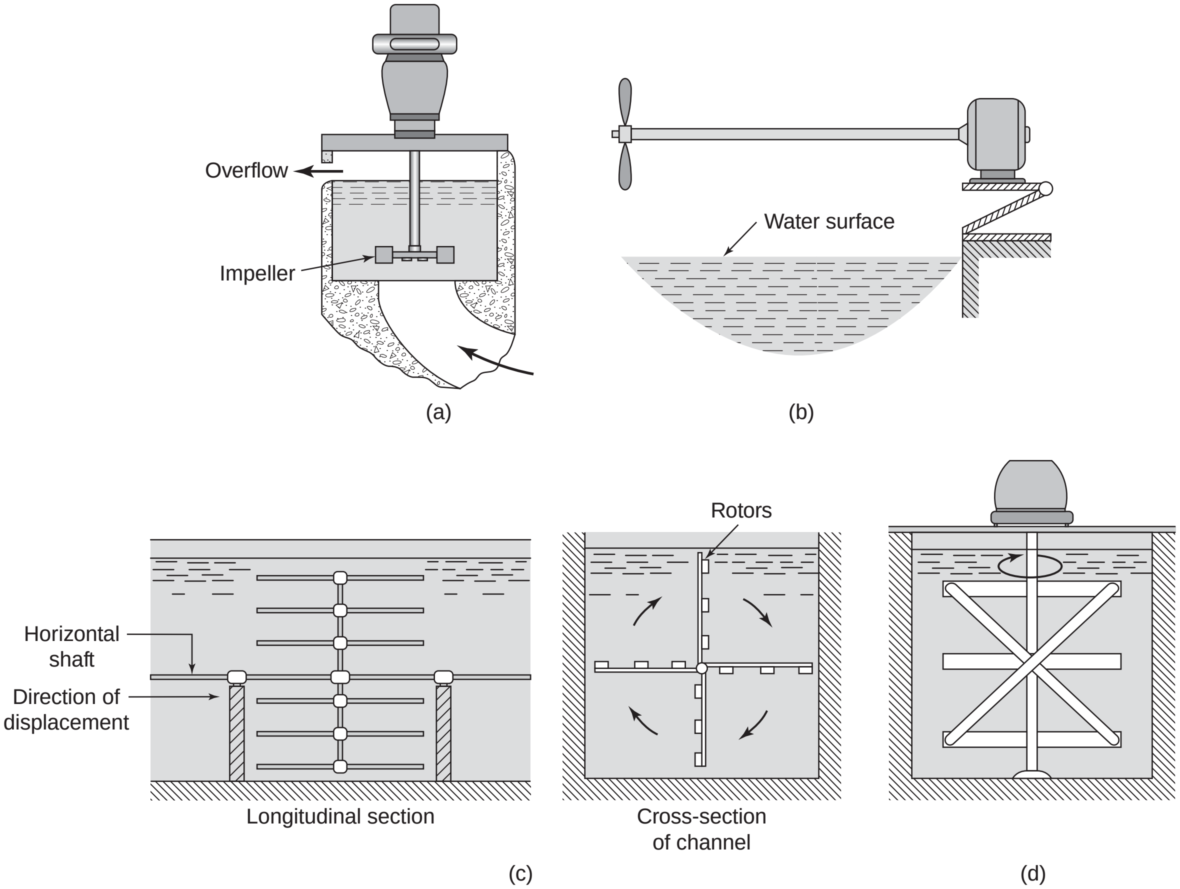


Figure 12.11 Mixing and stirring impellers: (a) flash mixer; (b) mixing propeller tilted into horizontal position; (c) paddle or blade mixer with horizontal shaft; (d) paddle mixer with vertical shaft (After Fair et al., 1971).

to moderate speeds of 2–15 rpm. The currents generated by them are both radial and tangential.

Turbines comprise flat or curved blades attached by a connecting radius arm to a vertical or horizontal shaft. Operating in the middle range of speeds (10–150 rpm), they generate much the same kind of currents as do paddles (Fig. 12.11a).

Propellers are shaped like ships' screws. The blades are mounted on a vertical or inclined shaft and generate strong axial currents. Their speed is high—150 to 1,500 rpm or more—and they may be placed off center in the treatment unit (Fig. 12.11b). Propellers are employed primarily in flash mixers.

Paddle flocculators may be arranged so that the flow is parallel to horizontal shafts installed lengthwise in parallel or sequential basins, or at right angles to vertical shafts or horizontal shafts installed across the width of one or more tanks or compartments. Alternative installations are illustrated in AWWA *Water Treatment Plant Design*.

The useful power input for mixing and stirring is lowered by the rotational movement of water masses as a whole and by vortex formation. Shear gradients become smaller because the velocity differential between the impeller and the water is narrowed, and power expended in changing water levels by vortex formation is not put to use for mixing purposes. Stators are useful adjuncts to all types of impellers.

Rearranging Eq. (12.3) gives a power equation.

$$P = G^2 V \mu \tag{12.3c}$$

Another expression for the power imparted to the water is given by

$$P = N_{\mathrm{P}} \rho n^3 d^5 \tag{12.3d}$$

where

N_{P} = power number of the impeller (power numbers for different types of impellers are given by the manufacturers)

d = impeller diameter, ft (m)

ρ = mass density of water, slug/ft^3 (kg/m^3) = γ/g

μ = dynamic (absolute) viscosity of water, lb-s/ft^2 (N-s/m^2)

n = impeller speed, revolutions per second, rps

P = power imparted to water, lb-ft/s (N-m/s or kg-m^2/s^3)

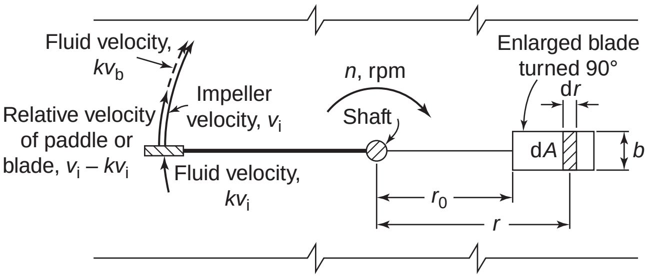


Figure 12.12 Schematic diagram of velocity and power relationships of mechanical mixers or stirrers (After Fair et al., 1971).

Table 12.5 Typical data for design of flocculation tanks

Parameter	Typical value
Gt_d value	20,000–150,000
Velocity gradient, G	20–80 s^{-1}
Detention time, t	20–30 min
Length to width ratio	4:1
Maximum stage volume	12,500 ft^3 (300 m^3)
Depth	12 ft (3.6 m)
Variable-speed motors	60% efficient
Freeboard and mixing apparatus	Require 20% of tank volume
Vertical mixers	Three-blade propeller impeller with R_N max of 10^4
Horizontal paddles mixers	8 (4 arms with 2 paddles)
Total paddle-blade area	Less than 20% of tank cross-sectional area
Paddle tip velocity	Less than 2 ft/s (0.61 m/s) for weak floc, and 4 ft/s (1.22 m/s) for strong floc

The useful power input of an impeller is a function of the impelling force F_I and coefficient of drag C_D, that is, the drag force, $F_D = C_D F_I$ of the paddle, blade, or propeller, the stator, if any, and the tank wall; the relative velocity v of the impeller and the fluid; and the area A of the impeller blade (Fig. 12.12). The useful power input $P = F_D v = C_D F_I v$, where $F_I = \rho A v^2/2$. Therefore,

$$P = {}^1\!/\!_2\, C_D \rho A v^3 \qquad \textbf{(12.6)}$$

where P = power input, ft-lb/s, C_D = drag coefficient, dimensionless, ρ = water density, 62.4/32.2 = 1.94 slug/ft^3, A = area, ft^2, and v = relative velocity, ft/s.

When Eq. (12.6) is applied as such using the SI units, then P = power input, N-m/s or W; C_D = drag coefficient, dimensionless; ρ = water density, 1 kg/L = 1,000 kg/m^3; A = area, m^2; and v = relative velocity, m/s.

If k is the ratio of the fluid velocity to the impeller velocity (v_i), the relative velocity of the blade $v = v_i - kv_i = (1 - k)\, v_i = 2\pi\,(1 - k)\, rn/60$. Here r is the effective radius arm of the blade and n is the number of revolutions per minute. It follows that the useful power expended by a single blade is

$$P = 5.74 \times 10^{-4} C_D \rho [(1 - k)n]^3 r^3 A \qquad \textbf{(12.7)}$$

If the dimension of A in the direction of the radius arm is substantial, but the width, b, of the blade is constant, Fig. 12.10 shows that $r^3 A = \int_{r_0}^{r} r^3 \mathrm{d}A = b \int_{r_0}^{r} r^3 \mathrm{d}r = \frac{1}{4}\, b\, (r^4 - r_0{}^4)$ because dA = b dr. Moreover, if the impeller includes a series of blades, $r^3 A$ is replaced by $\Sigma r^3 A$. In these terms, therefore,

$$P = 1.44 \times 10^{-4} C_D \rho [(1 - k)n]^3 b \sum (r^4 - r_0^4) \qquad \textbf{(12.8)}$$

The allowable loading Q/V is found by substituting Eq. (12.6), (12.7), or (12.8) in Eq. (12.2).

In flocculation practice, peripheral speeds of paddles range from 0.3 to 3 ft/s (0.09–0.91 m/s), k is about 0.25 in. (6.4 mm) the absence of stators, and C_D is approximately 1.8 for flat plates. At n = 1–5 rpm or more, paddles 8 ft (2.44 m) in diameter have Reynolds numbers of 7.57×10^4–3.79×10^5, and the power number is directly proportional to G^2. Accordingly, G becomes as meaningful as P, and G and Gt_d values have been tapered from high to low for best results in specific cases. Wherever possible the power dissipation function P/V should be determined by measuring the actual torque and rotor speed. Table 12.5 shows some typical data used in the design of flocculation tanks. Furthermore, some researchers have reported an empirical relationship that relates G (s^{-1}) to the coagulant concentration C (mg/L) in addition to contact time (min):

$$G^{2.8} Ct = 4.4 \times 10^6 \qquad \textbf{(12.9)}$$

EXAMPLE 12.1 CHEMICAL FEED CALCULATIONS FOR LIQUID ALUM

The following is a liquid alum's specification table or load slip provided by an alum manufacturer:

Specific gravity	Solution density (lb/gal)	Percent as Al_2O_3	Percent as dry alum	Dry alum content (lb/gal)	Dry alum content (g/L)
1.0069	8.40	0.19	1.12	0.09	11.277
1.0140	8.46	0.39	2.29	0.19	23.221
1.0211	8.52	0.59	3.47	0.30	35.432
1.0284	8.58	0.80	4.71	0.40	48.438

Specific gravity	Solution density (lb/gal)	Percent as Al_2O_3	Percent as dry alum	Dry alum content (lb/gal)	Dry alum content (g/L)
1.0357	8.64	1.01	5.94	0.51	61.521
1.0432	8.70	1.22	7.18	0.62	74.902
1.0507	8.76	1.43	8.41	0.74	88.364
1.0584	8.83	1.64	9.65	0.85	102.136
1.0662	8.89	1.85	10.88	0.97	116.003
1.0741	8.96	2.07	12.18	1.09	130.825
1.0821	9.02	2.28	13.41	1.21	145.110
1.0902	9.09	2.50	14.71	1.34	160.368
1.0985	9.16	2.72	16.00	1.47	175.760
1.1069	9.23	2.93	17.24	1.59	190.830
1.1154	9.30	3.15	18.53	1.72	206.684
1.1240	9.37	3.38	19.88	1.86	223.451
1.1328	9.45	3.60	21.18	2.00	239.927
1.1417	9.52	3.82	22.47	2.14	256.540
1.1508	9.60	4.04	23.76	2.28	273.430
1.1600	9.67	4.27	25.12	2.43	291.392
1.1694	9.57	4.50	26.47	2.58	309.540
1.1789	9.83	4.73	27.82	2.74	327.970
1.1885	9.91	4.96	29.18	2.89	346.804
1.1983	9.99	5.19	30.53	3.05	365.841
1.2083	10.08	5.43	31.94	3.22	385.931
1.2185	10.16	5.67	33.35	3.39	406.370
1.2288	10.25	5.91	34.76	3.56	427.131
1.2393	10.34	6.16	36.24	3.74	449.122
1.2500	10.43	6.42	37.76	3.93	472.000
1.2609	10.52	6.67	39.24	4.12	494.777
1.2719	10.61	6.91	40.65	4.31	517.027
1.2832	10.70	7.16	42.12	4.51	540.484
1.2946	10.80	7.40	43.53	4.71	563.539
1.3063	10.89	7.66	45.06	4.91	588.619
1.3182	10.99	7.92	46.59	5.12	614.149
1.3303	11.09	8.19	48.18	5.34	640.938
1.3426	11.20	8.46	49.76	5.57	668.078
1.3551	11.30	8.74	51.41	5.81	696.657
1.3679	11.41	9.01	53.00	6.05	724.987

The manufacturer ships a truck load of liquid alum to Springfield Village Water Treatment Plant, and its Material Safety Data Sheet (MSDS) specifies the liquid alum's concentration being 48.18% as dry alum. A verification is needed.

The plant engineer weights 3 10-mL samples of liquid alum on an analytical balance, and calculates the average weight to be 13.303 g/10 mL. Determine the liquid alum's (a) specific gravity; (b) the percent of dry alum; (c) weight of dry alum in liquid alum, lb/gal (g/L); and (d) the percent content as Al_2O_3.

Solution 1:

1. 13.303 g/10 mL = **1.3303 g/mL**

   Specific gravity = **1.3303**

2. From the specification table, the percent dry alum content is verified to be **48.18%**
3. The weight of dry alum in liquid alum = **5.34 lb/gal = 640.938 g/L**
4. The percent content as Al_2O_3 = **8.19%**

Note: Chemical feed calculations for liquid polymer or other liquid chemicals can be done in a similar manner if the chemical manufacturer provides correct chemical load ship, or MSDS.

EXAMPLE 12.2 PREPARATION OF DRY CHEMICAL'S STOCK SOLUTION OR CHEMICAL FEED SOLUTION

The following simple formula can be used for dry chemical's stock solution preparation:

$$W_{dc} = C_{ss}(V_{ss}),$$

where W_{dc} = weight of dry chemical, mg; C_{ss} = desired concentration of stock solution, mg/L; and V_{ss} = desired volume of stock solution, L. The dry chemical can be either inorganic or organic. Determine the amount of dry alum required to make up: (a) 1 L (1,000 mL) of 1% stock solution for a jar test and (b) 200 gal (757 L) of 10% stock solution for chemical feeding.

Solution 1 (US Customary System):

1. $C_{ss} = 1\% = 10{,}000\ \text{mg/L};\ V_{ss} = 1\ \text{L}$

$$W_{dc} = (10{,}000\ \text{mg/L})(1\ \text{L}) = 10{,}000\ \text{mg} = \mathbf{10\ g}$$

2. $C_{ss} = 10\% = 100{,}000\ \text{mg/L};\ V_{ss} = 200\ \text{gal} = 757\ \text{L}$

$$\begin{aligned} W_{dc} &= (100{,}000\ \text{mg/L})(757\ \text{L}) = 75{,}700{,}000\ \text{mg} \\ &= (75{,}700\ \text{g})(1\ \text{lb}/454\ \text{g}) \\ &= \mathbf{166.74\ lb} \end{aligned}$$

Solution 2 (SI System):

1. $C_{ss} = 1\% = 10{,}000\ \text{mg/L};\ V_{ss} = 1\ \text{L}$

$$W_{dc} = (10{,}000\ \text{mg/L})(1\ \text{L}) = 10{,}000\ \text{mg} = \mathbf{10\ g}$$

2. $C_{ss} = 10\% = 100{,}000\ \text{mg/L};\ V_{ss} = 757\ \text{L}$

$$\begin{aligned} W_{dc} &= (100{,}000\ \text{mg/L})(757\ \text{L}) = 75{,}700{,}000\ \text{mg} \\ &= 75{,}700\ \text{g} \\ &= \mathbf{75.7\ kg} \end{aligned}$$

EXAMPLE 12.3 PREPARATION OF LIQUID CHEMICAL'S STOCK SOLUTION OR CHEMICAL FEED SOLUTION USING VOLUMETRIC SOLUTION METHOD

The volumetric solution method is represented by the following formula:

$$(V_{lc})(C_{lc}) = (V_{ss})(C_{ss}),$$

where

V_{lc} = volume of bulk liquid chemical solution that must be added to V_{ss} to form stock solution or feed solution
C_{lc} = concentration of bulk liquid chemical solution
V_{ss} = volume of the liquid chemical target stock solution or feed solution
C_{ss} = concentration of liquid chemical's target stock solution or feed solution

The units of V_{lc} and V_{ss} should be identical, while the units of C_{lc} and C_{ss} should be identical.

Assume a 0.2% (2,000 mg/L) alum stock solution is desired, so every mL of stock solution added to a 2-L jar test unit equals 1 mg/L chemical feed dosage. Also assume that the commercial liquid alum available has a sp. gr. of 1.3303 g/mL, and a concentration of 5.34 lb/gal (640.94 g/L). Determine the volume of liquid alum required to produce one liter of 0.2% alum stock solution, using the volume solution method.

Solution 1 (US Customary System):

$$\begin{aligned} 1\ \text{mg/L} &= 8.345\ \text{lb/MG} = 8.345 \times 10^{-6}\ \text{lb/gal} \\ (V_{lc})(C_{lc}) &= (V_{ss})(C_{ss}) \\ (V_{lc})(5.34\ \text{lb/gal}) &= (1\ \text{L})(2{,}000 \times 8.345 \times 10^{-6}\ \text{lb/gal}) \\ V_{lc} &= 0.00312\ \text{L} \\ &= \mathbf{3.12\ mL} \end{aligned}$$

Therefore, the 0.2% alum stock solution is made up by placing 3.12 mL of liquid alum into a 1,000-mL flask and filling the flask with water to the 1,000-mL mark.

Solution 2 (SI System):

$$\begin{aligned}(V_{\mathrm{lc}})(C_{\mathrm{lc}}) &= (V_{\mathrm{ss}})(C_{\mathrm{ss}})\\(V_{\mathrm{lc}})(640.94\ \mathrm{g/L}) &= (1\ \mathrm{L})(2\ \mathrm{g/L})\\V_{\mathrm{lc}} &= 0.00312\ \mathrm{L}\\&= \mathbf{3.12}\ \mathrm{mL}\end{aligned}$$

Therefore, the 0.2% alum stock solution is made up by placing 3.12 mL of liquid alum into a 1,000-mL flask and filling the flask with water to the 1,000-mL mark.

EXAMPLE 12.4 TYPICAL JAR TEST PROCEDURES FOR CHEMICAL FEED DOSAGE DETERMINATION

This section introduces an example of jar test procedure. There are numerous techniques that can be used to run jar tests and many are site specific. Normally site-specific jar test procedures should be developed to best correlate jar test results to individual water or wastewater treatment plant performance. It is important to note that a jar test procedure for a conventional water treatment plant (involving chemical feeding, rapid mixing, flocculation sedimentation, filtration, etc.) is different from that for a lime-soda ash softening plant (involving chemical feeding rapid mixing, flocculation, sedimentation, softening, recarbonation, etc.) Besides, a jar test procedure for a sedimentation-filtration plant is different from that for a flotation-filtration plant. In this example, recomend a common jar test procedure for the conventional WTP. Explain how to modify a standard jar test for (a) simulating an intended lime-soda ash softening process; (b) simulating a dissolved air flotation process; (c) simulating a test and odor removed process.

Solution:

The following is a recommended jar test procedure for simulating a conventional WTP:

1. Prepare dry chemical's stock solutions in accordance with the methods introduced in Example 12.2.
2. Prepare liquid chemical's stock solutions in accordance with the methods introduced in Examples 12.1 and 12.3.
3. Set up jar test apparatus and arrange glassware.
4. Fill all jars to the 2-L mark with plant raw water.
5. Measure the appropriate amount of chemicals into syringes and place syringes nest to jars.
6. Rapid mix
   A. Turn mixer to maximum speed.
   B. Dose alum and cationic polymer coagulant in all jars simultaneously and stir 30 seconds. Place chemicals as close to the impeller tip as possible.
   C. After 30 seconds of mixing, reduce mixer speed to 35 rpm.
7. Flocculation
   A. Maintain mixer speed at 35 rpm and mix for 20 minutes.
   B. Observe formation of floc particles in each jar.
   C. Note: Set mixer speeds and mix times to approximate actual plant conditions.
8. Setting
   A. Turn off the mixer and let jars set 20 minutes or set to actual plant condition such as actual settling time.
   B. Take samples of 30 to 40 mL from the sample tap located 10 cm from the top of jar.
   C. Sample all jars "simultaneously" and place sample into small beakers.
   D. Measure the turbidity or other target water quality parameters of each sample.
9. Recording results
   A. Record test results on the attached lab sheet.
   B. Jar with lowest turbidity or other target water quality parameters indicates best coagulant dose.

Notes:

a. For simulation of a lime-soda ash softening process, the required lime, soda ash and CO_2 gas, etc. must be fed to the jars, and there may be multiple settling periods.

b. For simulation of a dissolved air flotation process involving chemical feeding, rapid mixing, flocculation, DAF clarification, etc., the standard jar tests may also be used for determining approximate chemical dosages, order of chemical dosing, detention time, velocity gradient *G*, etc. because the chemistry of coagulation-sedimentation is very similar to that of coagulation-DAF although the coagulant dosage, mixing detention time, and flocculation detention time of DAF are slightly lower. For detailed bench-scale coagulation-DAF testing, the readers are referred to the literature (Shammas et al., 2010).

c. For taste and odor control, aeration step, powdered activated carbon addition, ozonation step, or potassium permanganate addition, etc. can be incorporated into the standard jar tests. Learning how to modify the jar test procedures for simulation of various physicochemical treatment processes is very important.

EXAMPLE 12.5 FEEDING DRY CHEMICAL TO A WATER TREATMENT PLANT

Dry chemical feeders all have their feeder calculation curves provided by the feeder manufacturers. Typically the dry chemical feed rate is calculated based on the WTP flow rate and the required chemical dosage. Once the dry chemical feed rate is calculated, from the feeder calculation curve, the feed setting is determined to feed the exact amount of dry chemical (lb/day or kg/day). Determine the lime dosage in lb/day and kg/day, required to treat 15 MDG (56.78 MLD) at Albany WTP assuming the required lime dosage is 10 mg/L.

Solution 1 (US Customary System):

1 mg/L = 8.34 lb/MG

$$\text{Lime dosage} = (15\text{ MGD})(10 \times 8.34\text{ lb/MG}) = 1{,}251\text{ lb/day}$$

The WTP operator should find and set the correct lime feed setting based on the lime dry feeder calculation curve, and feed 1,251 lb/day to water.

Solution 2 (SI System):

1 mg/L = 1,000,000 mg/ML = 1,000 g/ML = 1 kg/ML

$$\text{Lime dosage} = (56.78\text{ MLD})(10 \times 1\text{ kg/ML}) = \mathbf{567.8\ kg/day}$$

The WTP operator should find and set the correct lime feed setting based on the lime dry feeder calculation curve, and feed 567.8 kg/day to water.

EXAMPLE 12.6 ACTIVATING POLYMERS BEFORE CHEMICAL FEEDING

Polymers (including non-ionic polymers and ionic polyelectrolytes) can be fed as dry product or as a liquid. If liquid polymers are used, it is sometimes necessary to activate the polymers prior to feeding. Activating consists of mixing a diluted solution of polymer. This allows the long molecular chains that make up the polymer to “unravel” and thus activate. In general, the polymer manufacturer should be contacted to determine an appropriate dilution at which a polymer should be fed to a WTP. Recommend a procedure which can be followed to prepare a polymer dilution assuming (a) polymer density = 12.1 lb/gal (1.45 kg/L); (b) 500 gal (1892.5 L) of 0.5% by weight of polymer is to be prepared.

Solution 1 (US Customary System):

The amount of concentrated polymer that must be added to prepare any dilution may be calculated by using a mass balanced method.

1. Determine the weight of 500 gal of water.

$$\text{Water weight} = 500\text{ gal} \times 8.34\text{ lb/gal}$$
$$= \mathbf{4{,}170\ lb}$$

2. Determine the weight of polymer required to prepare a 0.5% by weight solution.

$$\text{Polymer weight} = (0.5/100) \times 4{,}170\text{ lb}$$
$$= \mathbf{20.85\ lb}$$

3. Determine the volume of polymer required.

$$\text{Polymer volume} = 20.85\text{ lb} \times (\text{gal}/12.1\text{ lb})$$
$$= \mathbf{1.72\ gal}$$

Therefore, the diluted and activated polymer solution is made up by placing 1.72 gal of concentrated liquid polymer into a 500-gallon barrel and filling the barrel with water to the 500-gallon mark. Alternatively liquid polymer can also be weighed, and 20.85 lb of polymer can be diluted with 500 gal of water.

Solution 2 (SI System):

The amount of concentrated polymer that must be added to prepare any dilution may be calculated by using a mass balance method.

1. Determine the weight of 1,892.5 L of water.

$$\text{Water weight} = 1{,}892.5\text{ L} \times 1\text{ kg/L}$$
$$= \mathbf{1{,}892.5\ kg}$$

2. Determine the weight of polymer required to prepare a 0.5% by weight solution.

$$\text{Polymer weight} = (0.5/100) \times 1{,}892.5\text{ kg}$$
$$= \mathbf{9.46\ kg}$$

3. Determine the volume of polymer required.

$$\text{Polymer volume} = 9.46\text{ kg} \times (1\text{ L}/1.45\text{ kg})$$
$$= \mathbf{6.52\ L}$$

Therefore, the diluted and activated polymer solution is made up by placing 6.52 L of concentrated liquid polymer into 1,892.5-L barrel and filling the barrel with water to the 1,892.5-L mark. Alternatively liquid polymer can also be weighed, and 9.46 kg of polymer can be diluted with 1,892.5 L of water.

EXAMPLE 12.7 FEEDING DILUTED POLYMER OR OTHER LIQUID CHEMICAL TO A WATER TREATMENT PLANT

For any liquid chemical, such as liquid polymer, with less than 2% concentration, the chemical feed rate can be calculated by a simple material balance equation,

$$(Q_{\text{lc}} + Q_{\text{wtp}})C_{\text{wtp}} = Q_{\text{lc}}C_{\text{lc}},$$

where Q_{wtp} = water treatment plant flow; C_{wtp} = liquid chemical dosage required for water treatment at WTP; Q_{lc} = liquid chemical feed rate; and C_{lc} = liquid chemical's feed concentration. The units of Q_{wtp} and Q_{lc} should be identicals while the units of C_{wtp} and C_{lc} should be identical.

The desired polymer dose should be determined by jar testing, pilot filters, zeta meter, or colloid titration method (Wang et al., 1978). Once the dose is determined, calculations may be done to set the chemical feed pumps. Typically, the chemical feed pump rate is determined by pumping into or out of a graduated cylinder that has the gradations marked in mL. As a result, the chemical feed rate in mL/min may be calculated.

Determine the polymer solution feed rate Q_{lc} (mL/min), if the polymer solution concentration Q_{lc} = 0.5% = 5,000 mg/L, the WTP flow rate Q_{wtp} = 2.5 MGD = 9.46 MLD, and the required polymer dosage Q_{wtp} = 1 mg/L.

Solution 1 (US Customary System):

Since Q_{wtp} is much greater than Q_{lc}, the above material balance equation may be reduced to a simplified material balance equation,

$$Q_{\text{wtp}}C_{\text{wtp}} = Q_{\text{lc}}C_{\text{lc}}$$

(2.5 MGD)(1 mg/L) = (Q_{lc} MGD)(5,000 mg/L)

$$Q_{lc} = (2.5/5{,}000)\text{ MGD} = 0.0005\text{ MGD}$$
$$= (2.5/5{,}000)\text{ MGD} \times (10^6\text{ gal/MG})(3.784 \times 1{,}000\text{ mL/gal})(1\text{ d}/1{,}440\text{ min})$$
$$= \mathbf{1{,}314\ mL/min}$$

Note: 1 MGD = 2,628,472 mL/min

Solution 2 (SI System):

(9.46 MLD)(1 mg/L) = (Q_{lc} MLD)(5,000 mg/L)

$$Q_{lc} = (9.46/5{,}000)\text{ MLD} = 0.001892\text{ MLD}$$
$$= (9.46/5{,}000)\text{ MLD} \times (10^6\text{ L/ML})(1{,}000\text{ mL/L})(1\text{ d}/1{,}440\text{ min})$$
$$= \mathbf{1{,}314\ mL}/\text{min}$$

Note: 1 MLD = 694,444 mL/min

EXAMPLE 12.8 FEEDING LIQUID ALUM TO A WATER TREATMENT PLANT

Typically the chemical feed pump rate is determined by pumping into or out of a graduated cylinder that has the gradations marked in milliliters. As a result, the chemical feed rate in mL/min must be calculated. Assume Albany Water Treatment Plant is treating 15 MGD (56.78 MLD) and dosing 10 mg/L of liquid alum with a liquid alum feeder. Determine the liquid alum feeding rate (mL/min) assuming the commercial liquid alum contains 5.34 lb of dry alum per each pound of liquid alum (5.34 lb/gal = 640.94 g/L) and has a sp. gr. of 1.3303.

Solution 1 (US Customary System):

1 mg/L = 8.34 lb/MG

$$\text{Alum dosage required in water} = 10\text{ mg/L as dry alum}$$
$$(15\text{ MGD})(10 \times 8.34\text{ lb/MG}) = 1{,}251\text{ lb/day}$$

Liquid alum feeding rate Q_{lc}

$$= (1{,}251\text{ lb/day})(1\text{ d}/1{,}440\text{ min})(1\text{ gal}/5.34\text{ lb})(3{,}785\text{ mL/gal})$$
$$= \mathbf{615.77\ mL/min}$$

An engineer may also use the simplified material balance equation $Q_{wtp}C_{wtp} = Q_{lc}C_{lc}$ for calculation

$$(15\text{ MGD})(10 \times 8.34\text{ lb/MG}) = Q_{lc}(5.34 \times 10^6\text{ lb/MG})$$
$$Q_{lc} = 0.0002343\text{ MGD} = \mathbf{615\ mL/min}$$

Solution 2 (SI System):

1 mg/L = 1,000,000 mg/ML = 1,000 g/ML

$$\text{Alum dosage required in water} = 10\text{ mg/L as dry alum}$$
$$(56.78\text{ MLD})(10 \times 1{,}000\text{ g/ML}) = 567{,}800\text{ g/day}$$

Liquid alum feeding rate Q_{lc}

$$= (567{,}800\text{ g/day})(1\text{ d}/1{,}440\text{ min})(1\text{ L}/640.94\text{ g})(1{,}000\text{ mL/L})$$
$$= \mathbf{615.3\ mL/min}$$

An engineer may also use the simplified material balance equation $Q_{wtp}C_{wtp} = Q_{lc}C_{lc}$, for calculation

$$(56.78\text{ MLD})(10 \times 1{,}000\text{ g/ML}) = Q_{lc}(640.9 \times 10^6\text{ g/ML})$$
$$Q_{lc} = 0.0008859\text{ MLD} = \mathbf{615\ mL/min}$$

EXAMPLE 12.9 WATER QUALITY ANALYSIS TO DECIDE CHEMICAL TYPES, DOSAGE RANGES, AND CHEMICAL FEEDERS

The water supply for Springfield Village is taken from a stream with considerable variation in water quality. The raw water analyses and the related jar tests for an average year are shown below:

Occurrence, %	10	35	26	12	10	4	3
Raw water analyses							
Turbidity, NTU	25	51	79	127	139	185	>200
Alkalinity, mg/L as $CaCo_3$	85	128	95	83	94	78	83
Temperature, °C	17	26	37	29	42	32	35
Optimum jar test results							
Alum, mg/L	20	26	38	30	40	30	36
pH, unit	6.4	6.7	6.4	6.4	6.4	6.3	6.4

Determine (a) the types of water treatment chemicals required for treating the Springfield Water Treatment Plant's raw waters and the chemical dosage ranges of coagulant and alkalinity chemical and (b) the alum feeders required, and the reason for selections, if the design flow of Springfield WTP is 1.2 MGD (4.54 MLD), and liquid alum with a sp. gr. of 1.3182 is available locally.

Solution:

(a) Chemical Selection:

The raw water turbidity ranges from 25 to 200+ NTU, which requires 20–40 mg/L of alum, as $Al_2(SO_4)_2{\cdot}14.3H_2O$. Since it is the optimum jar test, alum is concluded to be the required coagulant. Alkalinity chemical should be fed only if natural alkalinity is insufficient. According to the stoichiometry equation shown in Example 11.3, one mole of alum $Al_2(SO_4)_2{\cdot}14.3H_2O$ will react with 3 moles of alkalinity $Ca(HCO_3)_2$ as $CaCO_3$

Molecular weight of $Al_2(SO_4)_2 \cdot 14.3H_2O$

$$= 2 \times 27 + 3(36 + 4 \times 16) + 14.3 \times (2 + 16)$$

$$= 611.4$$

Molecular weight of $3CaCO_3$

$$= 3(40 + 12 + 3 \times 16)$$

$$= 300$$

The alkalinity requirement

$$= (20 \text{ to } 40\ \text{mg/L}) \times (300/611.4)$$

$$= 9.81 \text{ to } 19.62\ \text{mg/L as } CaCO_3$$

Since natural alkalinity in Springfield raw water ranges from 78 to 128 mg/L as $CaCO_3$, no extra alkalinity is needed.

After chemical coagulation, the treated water pH = 6.4. Therefore, pH adjustment chemical is required. Other required chemicals include, at least, disinfectants and corrosion control chemicals.

(b) Feeder Selection:

Alum is available in dry form and liquid form; therefore, dry alum can be fed with dry feeders, while the liquid alum can be fed with liquid feeders. Dry alum may be supplied in lumps, or in ground, rice or powdered form. Shipment of dry alum may be in 100-lb (45.4 kg) bags, in drums or in bulk truck quantities. Liquid alum is supplied as 40–50% solution delivered in minimum loads of 4,000 gal (15,140 L). The choice between dry or liquid alum use is dependent on factors such as water treatment plant size, storage space availability, feeding method, and economics. For large WTPs with design flow greater than 2.5 MGD (9.46 MLD), purchase of liquid alum is justified only when alum supplier is close enough to make differences due to negligible in transportation costs. For small WTPs with design flow 2.5 MGD (9.46 MLD) or below, liquid alum is used for the purpose of simplicity.

Since Springfield WTP's design flow = 1.2 MGD (4.54 MLD), it is a small WTP. Assume liquid alum is available locally, the plant manager selects the liquid alum feed system using ground storage (shown in Fig. 12.4b).

From Example 12.1, a liquid alum with a sp. gr. of 1.3182 has a dry alum content of 5.12 lb/gal (614.15 g/L). Since the design flow = 1.2 MGD (4.54 MLD), and the alum dosage range is 20–40 mg/L as $Al_2(SO_4)_2 \cdot 14.3H_2O$, the liquid alum feeder must cover at least the range determined in below using the simplified material balance method:

$$(Q_{\mathrm{wtp}})(C_{\mathrm{wtp}}) = (Q_{\mathrm{lc}})(C_{\mathrm{lc}})$$

$$(4.54\ \mathrm{MLD})(20\ \text{to}\ 40\ \mathrm{mg/L}) = Q_{\mathrm{lc}}(614{,}150\ \mathrm{mg/L})$$

$$Q_{\mathrm{lc}} = 0.0001478\ \text{to}\ 0.0002957\ \mathrm{MLD}$$

$$= 102.6\ \text{to}\ 205.2\ \mathrm{mL/min}$$

The actual liquid alum feeder should be wider than 102.6–205.2 mL/min. Besides, an identical spare feeder is needed.

EXAMPLE 12.10 SIZING THE RAPID MIXING BASINS AND FLOCCULATION BASINS

A water treatment plant is to treat 25 MGD (94.6250 MLD) of raw water from a large stream. Water treatment consists of chemical feeding, rapid mixing, filtration, corrosion control, and disinfection. Assume: (a) 1 square rapid mixing basin with a detention time of 30 s and a depth of 10 ft (3.0480 m); (b) four rectangular flocculation basins with a detention of 30 min, length/width ratio of 2, and a depth of 15 ft (4.5720 m). Determine the dimensions of the rapid mixing basin and four flocculation basins.

Solution 1 (US Customary System):

1. Rapid mixing basin sizing design

$$t_d = 30\ \mathrm{s}$$

$$Q = 25\ \mathrm{MGD} = 25(1.547)\ \mathrm{ft^3/s} = 38.6750\ \mathrm{ft^3/s}$$

$$V = (38.6750\ \mathrm{ft^3/s})(30\ \mathrm{s}) = 1{,}160.25\ \mathrm{ft^3}$$

$$H = 10.0\ \mathrm{ft}$$

$$V = 1160.25\ \mathrm{ft^3} = W \times L \times H = W \times W \times 10$$

$$W = 10.5\ \mathrm{ft}$$

Select the rapid mixing basin dimensions

$$\boldsymbol{W \times L \times H = 10.5\ \mathrm{ft} \times 10.5\ \mathrm{ft} \times 10\ \mathrm{ft}}$$

2. Flocculation basin sizing design

$$t_d = 30\ \mathrm{min} = 30 \times 60\ \mathrm{s} = 1{,}800\ \mathrm{s}$$

$$Q = 25\ \mathrm{MGD} = 38.6750\ \mathrm{ft^3/s}$$

$$V = (38.6750\ \mathrm{ft^3/s})(1{,}800\ \mathrm{s}) = 69{,}615\ \mathrm{ft^3}$$

$$H = 15\ \mathrm{ft}$$

There are four flocculation basins

$$V/4 = 69{,}615\ \mathrm{ft^3}/4 = L \times W \times H = 2W \times W \times 15\ \mathrm{ft^3}$$

$$W^2 = 580.12,\ \text{So}\ W = 24\ \mathrm{ft}$$

Select the flocculation basin dimensions as: four flocculation basins each has a dimension of $\boldsymbol{W \times L \times H = 24\ \mathrm{ft} \times 48\ \mathrm{ft} \times 15\ \mathrm{ft}}$

Solution 2 (SI System):

1. Rapid mixing basin sizing design

$$t_d = 30 \text{ s}$$

$$Q = 94.6250 \text{ MLD} = 94.6250 \times 1{,}000 \text{ m}^3/(1{,}440 \times 60 \text{ s}) = 1.0952 \text{ m}^3/\text{s}$$

$$V = (1.0952 \text{ m}^3/\text{s})(30 \text{ s}) = 32.8559 \text{ m}^3$$

$$H = 3.0480 \text{ m}$$

$$V = 32.8559 \text{ m}^3 = W \times L \times H = W \times W \times 3.048$$

$$W = 3.28 \text{ m}$$

Select the rapid mixing basin dimensions

$$\boldsymbol{W \times L \times H = 3.28 \text{ m} \times 3.28 \text{ m} \times 3 \text{ m}}$$

2. Flocculation basin sizing design

$$t_d = 30 \text{ min} = 1{,}800 \text{ s}$$

$$Q = 94.6250 \text{ MLD} = 1.0952 \text{ m}^3/\text{s}$$

$$V = (1.0952 \text{ m}^3/\text{s})(1{,}800 \text{ s}) = 1{,}971.36 \text{ m}^3$$

$$H = 4.5720 \text{ m}$$

There are four flocculation basins

$$V/4 = 1{,}971.36 \text{ m}^3/4 = L \times W \times H = 2W \times W \times 4.5720 \text{ m}^3$$

$$W^2 = 53.8976, \text{ So } W = 7.34 \text{ m}$$

Select the flocculation basin dimensions as: four flocculation basins, each has a dimension of **$W \times L \times H$ = 7.34 m × 14.68 m ×4.57 m**

EXAMPLE 12.11 DESIGN OF IN-LINE MECHANICAL BLENDERS

Choose a typical commercial in-line mechanical blender to treat a maximum daily demand of 1.6 MGD (6.0560 MLD) based on the technical information in Section 12.3.3. Water temperature ranges from 8°C to 18°C. Check the velocity gradient to see whether or not it is in the range of 3,000–5,000 s^{-1} for the in-line mechanical blender application.

Solution 1 (US Customary System):

Select a residence time of 0.5 s in blender (check on typical residence time of a commercial blender) from Section 12.3.3.

0.5 water hp is required for each MGD of water flow.

$$P = \text{required power input} = (0.5 \text{ hp}/1 \text{ MGD})(1.6 \text{ MGD})$$

$$= 0.8 \text{ water hp}$$

$$= 0.8 \text{ hp} \times (550 \text{ ft-lb/s})/(1 \text{ hp})$$

$$= 440 \text{ ft-lb/s}$$

$$t_d = 0.5 \text{ s}$$

$$Q = 1.6 \text{ MGD} = 1.6 \text{ MGD} \times (1.547 \text{ cfs/MGD}) = 2.4752 \text{ ft}^3/\text{s}$$

$$V = \text{water volume to which power is supplied at } 0.5 \text{ s}$$

$$= Qt_d = (2.4752 \text{ ft}^3/\text{s})(0.5 \text{ s}) = 1.2376 \text{ ft}^3$$

At 8°C

$$\mu = \text{viscosity} = 1.387 \text{ centipoises at } 8°\text{C}$$

$$= 1.387 \text{ centipoises} \times (2.088 \times 10^{-5} \text{ lb-s/ft}^2)/(1 \text{ centipoises})$$

$$= 2.90 \times 10^{-5} \text{ lb-s/ft}^2 \text{ at } 8°\text{C}$$

$$G = (P/\mu V)^{0.5}$$

$$= \{(440 \text{ ft-lb/s})/[(2.9 \times 10^{-5} \text{ lb-s/ft}^2)(1.2376 \text{ ft}^3)]\}^{0.5}$$

$$= 3{,}500 \text{ s}^{-1}$$

At 18°C

$$\mu = 2.2 \times 10^{-5} \text{ lb-s/ft}^2 \text{ at } 18°\text{C}$$

$$G = \{(440 \text{ ft-lb/s})/[(2.2 \times 10^{-5} \text{ lb-s/ft}^2)(1.2376 \text{ ft}^3)]\}^{0.5}$$

$$= 4{,}000 \text{ s}^{-1}$$

The range of *G* values lies within the recommended values of 3,000–5,000 s^{-1}

Solution 2 (SI System):

Select a residence time of 0.5 s in blender (check on typical residence time of a commercial blender) from Section 12.3.3. 0.37 kW is required for 43.8 L/s of water flow. 6.0560 MLD = 6.0560 × 10^6L/1,440 × 60 s = 70.1 L/s

$$P = \text{required power input} = (0.37 \text{ kW}/43.8 \text{ L/s})(70.1 \text{ L/s})$$

$$= 0.592 \text{ kW}$$

$$= 581.61 \text{ N-m/s or watts}$$

$$t_d = 0.5 \text{ s}$$

$$Q = 6.0560 \text{ MLD} = 6.0560 \times 10^3 \text{ m}^3/(1{,}440 \times 60 \text{ s})$$

$$= 0.07 \text{ m}^3/\text{s}$$

$$V = \text{water volume to which power is supplied at 0.5 s}$$

$$= Qt_\text{d}$$

$$= (0.07 \text{ m}^3/\text{s})(0.5 \text{ s}) = 0.035 \text{ m}^3$$

At 8°C

$$\mu = \text{viscosity} = 1.387 \text{ centipoises at } 8°\text{C}$$

$$= 1.387 \times 0.001 \text{ Pa-s or N-s/m}^2$$

$$= 1.387 \times 10^{-3} \text{ N-s/m}^2$$

$$G = (P/\mu V)^{0.5}$$

$$= \{(581.61 \text{ N-m/s})/[(1.387 \times 10^{-3} \text{ N-s/m}^2)(0.035 \text{ m}^3)]\}^{0.5}$$

$$= 3{,}500 \text{ s}^{-1}$$

At 18°C

$$\mu = 1.060 \times 10^{-3} \text{ N-s/m}^2$$

$$G = \{(581.61 \text{ N-m/s})/[(1.06 \times 10^{-3} \text{ N-s/m}^2)/(0.035 \text{ m}^3)]\}^{0.5}$$

$$= 4{,}000$$

The range of *G* values lies within the recommended values of 3,000–5,000 s^{-1}.

EXAMPLE 12.12 RAPID MIXING BASIN SIZING AND MECHANICAL MIXER DESIGN

Calculate the volume and dimension ($L \times W \times H$) of a square rapid mixing basin and design the mechanical mixer, assuming (a) maximum design flow = 100 MLD = 26.42 MGD; (b) number of mixing basins = 4; (c) number of mixing stages = 1; (d) mixing detention time = 30 seconds; (e) mixer velocity gradient (G) = 950/seconds; (f) depth-to-width ratio of the mixing basin = 1.5; (g) raw water temperature range = 5°C to 29°C; (h) efficiency of mechanical mixer's gearbox = 90%; (i) power number of the impeller given by the manufacturer $N_p = 2.75$; and (j) Reynolds number equation for rapid mixers $R = d^2 n\rho/\mu$, where d = impeller diameter, m; n = impeller rotation speed, revolution per second, or rps; ρ = mass density, kg/m^3; and μ = dynamic viscosity = 1.518×10^{-3} N-s/m^2 at 5°C.

Solution:

Four square rapid mixing basins can be properly sized using the maximum design flow and the required detention time. The Ten-States Standards specify that the mixing detention period should not be more than 30 seconds, with mixing equipment capable of imparting a minimum velocity gradient (G) of at least 750 fps/ft or m/s/m, or s^{-1}. The design engineer should determine the appropriate detention time and G value through a series of jar testing. The following are calculations for mixing basin sizing and mechanical mixer design.

$$N = 4 \text{ mixing values}$$

$$Q = 100 \text{ MLD} = 100{,}000 \text{ m}^3/\text{day} = 69.44 \text{ m}^3/\text{min} = 1.1574 \text{ m}^3/\text{s}$$

$$Q/N = (1.1574 \text{ m}^3/\text{s})/4 = 0.2894 \text{ m}^3/\text{s}$$

$$V \text{ for each mixing basin} = (Q/N)t_d$$

$$= (0.2894 \text{ m}^3/\text{s})(30 \text{ s}) = 8.68 \text{ m}^3$$

$$\text{Volume} = L \times W \times H = W \times W \times 1.5\ W = 1.5\ W^3 = 8.68 \text{ m}^3$$

$$W^3 = 8.68/1.5 = 5.7867$$

$$W = 5.7867^{0.333} = 1.8 \text{ m}; L = W = 1.8 \text{ m}; H = 1.5 \times 1.8 \text{ m} = 2.7 \text{ m}$$

$$\textbf{Dimension} = \boldsymbol{L \times W \times H} = \textbf{1.8 m} \times \textbf{1.8 m} \times \textbf{2.7 m}$$

Design each mechanical mixer for a velocity gradient (G) of 950/s at a flow rate of 0.29 m^3/s, which is one-fourth of the maximum design flow rate. The expected raw water temperature is in the range of 5°C to 29°C. The lowest temperature presents the critical condition in this mixer design.

$$\mu = 1.518 \times 10^{-3} \text{ N-s/m}^3 \text{ at } 5°\text{C}$$

$$V = \text{mixing basin volume} = 8.68 \text{ m}^3$$

Rearranging Eq. (12.3) power equation.

$$P = G^2 V\mu$$

$$P = (950/\text{s})^2(8.68 \text{ m}^3)(1.518 \times 10^{-3} \text{ N-s/m}^2)$$

$$= 11{,}892 \text{ N-m/s}$$

$$= 11{,}892 \text{ W}$$

$$= 11{,}892 \text{ kg-m}^2/\text{s}^3$$

$$= 11.89 \text{ kW}$$

Since P is the power imparted to the water, the power of the drive (P_d) is calculated by dividing P by the efficiency of the gearbox, E_g, which is typically around 90%.

$$P_d = P/E_g$$

$$= 11.89 \text{ kW}/0.9$$

$$= 13.21 \text{ kW}$$

Selection of turbine diameter $d = 0.5 \times$ mixing basin width $= 0.5 \times 1.8$ m $= 0.90$ m

Calculation of impeller rotation speed n, assuming $N_p = 2.75$ given by the manufacturer.

$$n = (P/\rho N_p d^5)^{1/3} \quad \textbf{(12.3b)}$$

$$= [(11,892 \text{ kg-m}^2/\text{s}^3)/(1{,}000 \text{ kg/m}^3)(2.75)(0.9 \text{ m})^5]^{1/3}$$

$$= [7.3788]^{1/3}$$

$$= 1.94 \text{ rps (or 117 rpm)}$$

Find dynamic viscosity of water at 5°C.

$$\mu = 1.518 \times 10^{-3} \text{ N-s/m}^2$$

$$= 1.518 \times 10^{-3}[(\text{kg-m/s}^2)\text{s}]/\text{m}^2$$

$$= 1.518 \times 10^{-3} \text{ kg/s-m}$$

Here, $1\ N = 1$ kg-m/s^2.

Check Reynolds number for turbulent flow using a Reynolds number equation for rapid mixers.

$$\mathbb{R} = d^2 n\rho/\mu = d^2 n\gamma/\mu g$$

$$= (0.9 \text{ m})^2(1.94 \text{ rps})(1{,}000 \text{ kg/m}^3)/(1.518 \times 10^{-3} \text{ kg/s-m})$$

$$= 1.0352 \times 10^6 > 10{,}000$$

Therefore, it is in the turbulent range. Okay.

EXAMPLE 12.13 DETERMINATION OF HEADLOSS, POWER, AND LOADING OF A BAFFLED CHANNEL

Water zigzags through a baffled channel basin at a velocity of 0.5 ft/s (0.15 m/s) and speeds up to 1.5 ft/s (0.46 m/s) in the slots. There are 19 around-the-end baffles. The flocculation basin is 10 ft × 30 ft × 60 ft (3.05 m × 9.14 m × 18.29 m). Estimate

1. The loss of head, neglecting normal channel friction
2. The power dissipated
3. The G and Gt_d values for a flow of 6.46 MGD (10.0 ft^3/s = 0.28 m^3/s), with a displacement time of 30 min
4. The channel loading

Assume a water temperature of 50°F (10°C), that is, $\mu = 2.74 \times 10^{-5}$ lb force-s/ft^2 (1.31×10^{-3} N-s/m^2) .

Solution 1 (US Customary System):

1. The loss of head

$$h = nv_1^2/2g + (n-1)v_2^2/2g$$

$$h = 20 \times 0.5^2/2g + 19 \times 1.5^2/2g = \mathbf{0.74} \text{ ft.}$$

2. The power dissipated

$$P = Q\rho g h$$

$$P = 10 \text{ ft}^3/\text{s} \times 62.4 \text{ lb/ft}^3 \times 0.74 \text{ ft}$$

$$= \mathbf{462 \text{ ft-lb/s} = 0.84 \text{ hp} = 626 \text{ W}}.$$

3. The G and Gt_d values

$$V = 10 \text{ ft} \times 30 \text{ ft} \times 60 \text{ ft} = 18{,}000 \text{ ft}^3.$$

$$G = \sqrt[2]{P/\mu V}$$

$$G = [462/(2.74 \times 10^{-5} \times 18{,}000)]^{1/2}$$

$$= \mathbf{30.6/s}.$$

$$Gt_d = 30.6/\text{s} \times 30 \text{ min} \times 60 \text{ s/min} = \mathbf{5.5 \times 10^4}.$$

4. The channel loading, flow per unit volume

$$Q/V = 6.46 \times 10^6\ \text{gpd}/(18{,}000)\ \text{ft}^3$$
$$= \mathbf{360\ gpd/ft^3}.$$

Solution 2 (SI System):

1. The loss of head

$$h = nv_1^2/2g + (n-1)v_2^2/2g$$
$$h = 20 \times (0.15)^2/2 \times 9.81 + 19 \times (0.46)^2/2 \times 9.81$$
$$= \mathbf{0.23\ m}.$$

2. The power dissipated

$$P = Q\rho gh$$
$$P = 0.28\ \text{m}^3/\text{s} \times 9{,}804\ \text{N/m}^3 \times 0.23\ \text{m}$$
$$= \mathbf{627\ N\text{-}m/s{=}0.627\ kW = 627\ W}.$$

3. The G and Gt_d values

$$V = 3.05\ \text{m} \times 9.14\ \text{m} \times 18.29\ \text{m} = 510\ \text{m}^3.$$
$$G = \sqrt[2]{P/\mu V}$$
$$G = [(627\ \text{N-m/s})/(1.31 \times 10^{-3}\ \text{N-s/m}^2)(510\ \text{m}^3)]^{1/2}$$
$$= \mathbf{30.6/s}.$$
$$Gt_d = 30.6/\text{s} \times 30\ \text{min} \times 60\ \text{s/min} = \mathbf{5.5 \times 10^4}.$$

4. The channel loading, flow per unit volume

$$Q/V = [(0.28\ \text{m}^3/\text{s})/(510\ \text{m}^3)](60\ \text{s/min})(60\ \text{min/h})(24\ \text{h/day})$$
$$= \mathbf{47.4\ m^3/day/m^3}.$$

EXAMPLE 12.14 DETERMINATION OF THE VELOCITY DIFFERENTIAL, POWER, DETENTION TIME, VELOCITY GRADIENT, AND LOADING OF A FLOCCULATOR

A flocculator designed to treat 20 MGD (75.7 MLD = 0.88 m^3/s) is 100 ft long, 40 ft wide, and 15 ft deep (30.48 m × 12.19 m × 4.57 m). It is equipped with 12 in. (305 mm) paddles supported parallel to and moved by four 40 ft (12.19 m) long horizontal shafts that rotate at a speed of 2.5 rpm. The center line of the paddles is 6.0 ft (1.83 m) from the shaft, which is at mid-depth of the tank. Two paddles are mounted on each shaft, one opposite the other. If the mean velocity of the water is approximately one-fourth the velocity of the paddles and their drag coefficient is 1.8, find

1. The velocity differential between the paddles and the water
2. The useful power input and the energy consumption
3. The detention time
4. The value of G and the product Gt_d
5. The flocculator loading

Assume a water temperature of 50°F (10°C), $\mu = 2.74 \times 10^{-5}$ (lb force)(s)/ft^2 (1.31×10^{-3} N-s/m^2).

Solution 1 (US Customary System):

1. The paddle velocity is

$$v_i = 2\pi rn = 2\pi \times 6(2.5/60) = 1.57\ \text{ft/s, and}$$

the velocity differential is

$$\begin{aligned} v &= (1-k)v_i \\ &= (1-0.25)1.57 \\ &= \mathbf{1.18\ ft/s}. \end{aligned}$$

2. Because the area of the paddles is $A = 40 \times 2 \times 4 \times 1 = 320\ \text{ft}^2$ and the coefficient of drag $C_\text{D} = 1.8$, the useful power input, by Eq. (12.6), is

$$\begin{aligned} P &= {}^1\!/_2 C_D \rho A v^3 \\ P &= 0.5 \times 1.8(62.4/32.2)(320)(1.18)^3 \\ &= 918\ \text{ft-lb/s} \\ &= 918/550\ \text{hp} \\ &= \mathbf{1.67\ hp\ or\ 1.24\ kW}. \end{aligned}$$

The energy consumption per million gallons, therefore, is 1.67 × 24/20 = 2.0 hph/MG, or 1.5 kWh/MG (0.39 kWh/ML) treated. For electrical drive, there must be added the energy required to overcome mechanical friction and to provide for electrical losses in the lines and motor. (In practice, flocculators consume 2–6 kWh/MG treated or 0.53–1.6 kWh/ML treated.)

3. Because the volume of the tank is $40 \times 100 \times 15 = 6 \times 10^4\ \text{ft}^3$, the detention period is

$$\begin{aligned} t_d &= V/Q \\ &= 6 \times 10^4\ \text{ft}^3 \times 7.48\ \text{gal/ft}^3 \times 24\ \text{h/day} \\ &\quad \times 60\ \text{min/h}/(20 \times 10^6)\ \text{MGD} \\ &= \mathbf{32.5\ min}. \end{aligned}$$

4. The value of G is

$$\begin{aligned} G &= \sqrt[2]{P/\mu V} \\ G &= [918/(2.74 \times 10^{-5} \times 6 \times 10^4)]^{1/2} \\ &= \mathbf{23.7\ ft/s/ft}. \\ Gt_d &= 23.7/\text{s} \times 32.5\ \text{min} \times 60\ \text{s/min} \\ &= \mathbf{4.64 \times 10^4}. \end{aligned}$$

5. The flocculator loading is

$$\begin{aligned} Q/V &= 20 \times 10^6/(6 \times 10^4) \\ &= \mathbf{333\ gpd/ft^3} \\ &= 44.5\ \text{ft}^3/\text{d}/\text{ft}^3. \end{aligned}$$

Solution 2 (SI System):

1. The paddle velocity is

$$v_i = 2\pi rn = 2\pi \times 1.83(2.5/60) = 0.48\ \text{m/s, and}$$

the velocity differential is

$$\begin{aligned} v &= (1-k)v_i \\ &= (1-0.25)(0.48) \\ &= \mathbf{0.36\ m/s}. \end{aligned}$$

2. Area of the paddles is A = (12.19 × 0.30 m^2 /paddle) (4 shafts)(2 paddles/shaft) = 30 m^2, the useful power input, by Eq. (12.6), is

$$P = {}^1\!/\!_2 C_D \rho A v^3$$
$$P = 0.5 \times 1.8[(9{,}804\,\mathrm{N/m^3})/(9.81\,\mathrm{m/s^2})](30\,\mathrm{m^2})(0.36\,\mathrm{m/s})^3$$
$$= 1{,}260\,\mathrm{N\text{-}m/s}$$
$$= \mathbf{1.25\,kW}.$$

Energy consumption per ML = 1.25 × 24/75.7 ML = 0.40 kWh/ML treated. For electrical drive, there must be added the energy required to overcome mechanical friction and to provide for electrical losses in the lines and motor.

3. Volume of tank = 30.48 × 12.19 × 4.57 = 1,700 m^3

$$\text{The detention period is } t_\mathrm{d} = V/Q$$
$$= [(1{,}700\,\mathrm{m^3})/(0.88\,\mathrm{m^3/s})]/(60\,\mathrm{s/min})$$
$$= \mathbf{32.2\,min}.$$

4. The value of G is

$$G = \sqrt[2]{P/\mu V}$$
$$G = [1{,}260/(1.31 \times 10^{-3} \times 1{,}700)]^{1/2}$$
$$= \mathbf{23.8/s}.$$
$$Gt_\mathrm{d} = (23.8/\mathrm{s})(32.2\,\mathrm{min} \times 60\,\mathrm{s/min})$$
$$= \mathbf{4.60 \times 10^4}.$$

5. The flocculator loading is

$$Q/V = 75.7 \times 10^6\,\mathrm{L/day}/(1{,}700\,\mathrm{m^3})$$
$$= \mathbf{44{,}500\,L/day/m^3\ or\ 44.5\,m^3\,/day/m^3}.$$

12.7 FLOCCULATOR PERFORMANCE

The performance of mixing and stirring devices for flocculation is a function of (a) the substances to be removed, (b) the mixer design, and (c) the additives, if any, introduced to promote coagulation. Examples of common operational practices in successful chemical coagulation are, in the simplest cases, routine additions of standard amounts of chemicals; in more closely supervised cases, daily, hourly, or even more frequent adjustments of coagulant dosages to the results obtained in *jar tests.*

Mixing must be thorough enough to encourage interparticle contact, but gentle enough to prevent disintegration of existing flocculated particles. The three basic requirements for flocculation process performance are

1. Installing mixing equipment that improve agitation
2. Design of inlet and outlet to minimize destabilizing turbulence
3. Installing baffling to reduce basin short-circuiting

12.7.1 Mixing

Different mixing equipment can improve turbulence patterns in the basin to maximize the formation of flocculated particles and minimize the destruction of previously formed floc. There are several types of mixers used for flocculation, with the mechanical mixer being the most common. Mechanical mixers for flocculation basins are differentiated by overall design and type of agitator.

The most important goal in flocculator design is the efficient removal of floc during sedimentation and filtration. Many flocculator units, especially low-energy mixers, tend to maximize floc size rather than floc density, which affects the speed with which floc settles. High-energy mixers create smaller and denser floc which settles faster and occupies less volume in the filter bed than the larger floc created by low-energy mixers.

Paddle, walking-beam, and flat blade turbine mixers are known as low-energy mechanical mixers. The paddle or reel mixer operates with low tip speeds of 2–15 rpm to prevent floc destruction. The flat blade turbine typically operates at 10–15 rpm. Flat blade turbines can produce excessively high velocity gradients of over 45 rpm.

Axial flow propellers or turbines, known as high-energy mechanical flocculators, typically operate at 150–1,500 rpm with no limit on blade tip speed. Axial flow propellers are favored in some situations because they produce uniform turbulence and are simple to install and maintain.

12.7.2 Flocculator Inlet and Outlet Structures

The desired detention time (i.e., residence time) determines the size and occasionally the design configuration of the flocculation basin. The basin's internal design then determines detention time effectiveness. The design of the basin inlet, outlet, and internal circulation patterns all affect floc formation and destruction.

Typical detention times range from 15 to 45 min. These are influenced by the influent water conditions, type of coagulant used, and requirements of downstream processes. Cold, low turbidity water could require 30 min of detention, while the same water undergoing direct filtration at higher temperatures could require only 15 min of detention. In small plants with high efficiency volumetric mixing, 10–15 min of detention might be acceptable.

If water passes through the flocculation basin in much less than the volumetric residence time, the influent stream is said to have "short-circuited." To achieve effective flocculation and minimize short-circuiting, designers must pay particular attention to inlets, outlets, and internal baffling. Inlet and outlet turbulence is the major source of destructive energy in the flocculation basin contributing to short-circuiting.

Effective baffle, inlet, and outlet design reduces short-circuit problems. Improvements that are often overlooked can include

1. Adding inlet diffusers to improve the uniformity of the distribution of incoming water
2. Enlarging connecting conduits to reduce floc-disrupting turbulence
3. Adding a secondary entry baffle across the inlet to the flocculation basin to impart increased head loss for uniform water entry

Improving inlet and outlet conditions will reduce basin turbulence and thus lessen floc breakup, increase detention times, and allow more efficient coagulant usage.

12.7.3 Improving Basin Circulation with Baffles

Flocculation basins can be baffled to direct the movement of the water through the basin. Baffling, usually near the

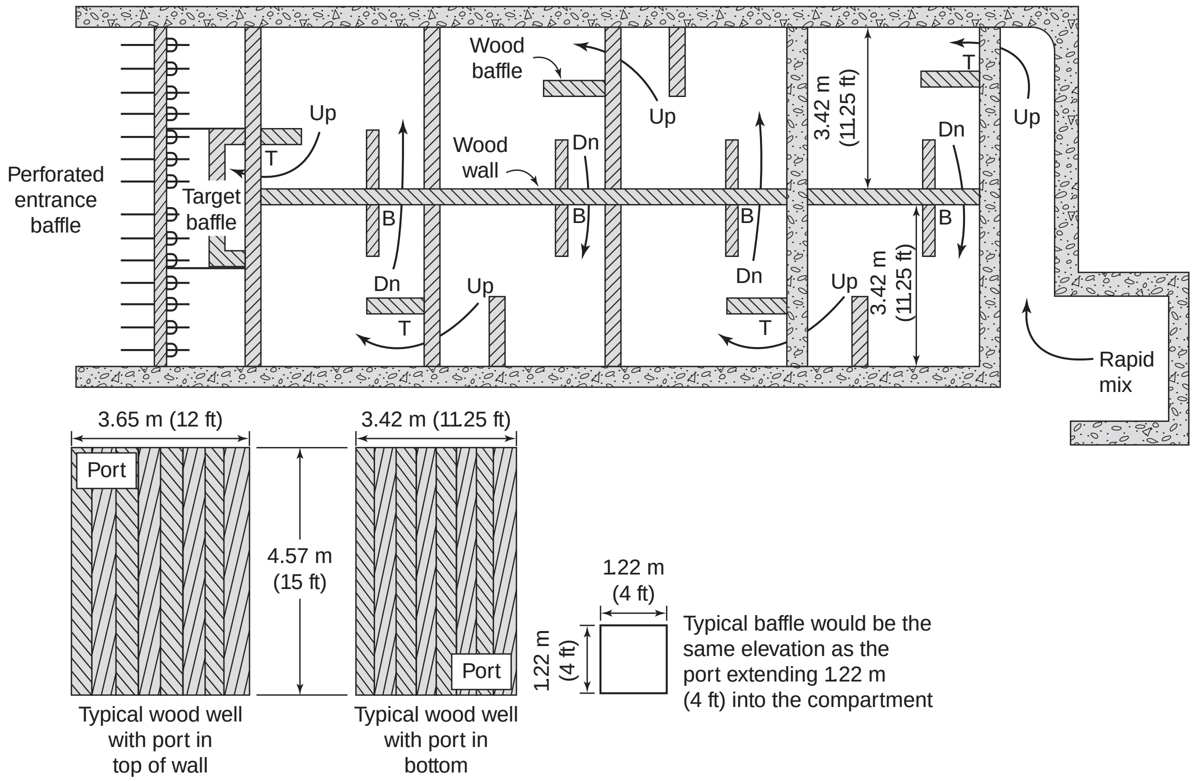


Figure 12.13 Divided flocculation basin (After U.S. EPA, 1990).

Table 12.6 Estimated costs of rapid mix in 2012 USD

Category	Plant capacity (MGD)	Average flow (MGD)	Capital cost (USD 1,000)	Operation and maintenance costs		Total cost (¢/1,000 gal)	Total cost (¢/1,000 L)
				USD 1,000/year	¢/1,000 gal		
1	0.026	0.013	35.1	7.46	156	243	64.3
2	0.068	0.045	46.5	7.71	46.9	80.1	21.2
3	0.166	0.133	60.0	18.6	38.8	54.0	14.3
4	0.500	0.400	82.3	21.0	34.2	21.0	5.55
5	2.500	1.300	127	35.4	7.46	10.65	2.82
6	5.850	3.250	169	59.6	5.06	6.66	1.76
7	11.59	6.750	234	102	4.26	5.32	1.41
8	22.86	11.50	370	184	4.26	5.32	1.41
9	39.68	20.00	581	309	4.26	4.26	1.13
10	109.9	55.50	1560	833	3.99	4.26	1.13
11	404.0	205.0	5,590	3,000	3.99	3.70	0.98
12	1,275	650.0	17,750	9,424	3.63	3.70	0.98

Conversion factors: 1 MGD = 3.785 MLD; 1¢/1,000 gal = 0.264 ¢/1,000 L.

basin entrances and exits, can improve basin circulation and achieve more uniform flocculation. Baffles, commonly made of wood, plastic, concrete, or steel are either "over–under" or "around-the-end" in design. "Over–under" designs direct water flow either over or under the baffle, while "around-the-end" designs direct the water around either end of the baffle. Baffles should be designed to direct the water flow such that the velocity gradients are less than those produced in the flocculation process but greater than those in the flow moving laterally across the inlets. An example of a basin divided with baffling is illustrated in Fig. 12.13.

Flocculation basins are designed with multiple compartments, in addition to minimizing short-circuiting is to facilitate the incorporation of zones of reduced energy input and tapered velocity gradients. The tapered feature may be provided by varying the rotational speed (variable-speed drives), the paddle size, the number of paddles, and the diameter of the paddle wheels. A typical example of such design is the provision of three flocculation compartments having *G* values of 60, 40, and 20 s^{-1}, respectively.

12.8 COSTS

Estimated capital and operation and maintenance costs for rapid mix and flocculation are presented in Tables 12.6 and 12.7. All costs have been converted to current 2012 USD using US Army Corps of Engineers Yearly Average Cost Index for Utilities (see Appendix 16).

Table 12.7 Estimated costs of flocculation in 2012 USD

Category	Plant capacity (MGD)	Average flow (MGD)	Capital cost (USD 1,000)	Operation and maintenance costs		Total cost (¢/1,000 gal)	Total cost (¢/1,000 L)
				USD 1,000/year	¢/1,000 gal		
1	0.026	0.013	26.6	2.66	57.8	120	31.7
2	0.068	0.045	48.0	2.93	18.4	53.5	14.1
3	0.166	0.133	90.5	6.13	13.1	35.4	9.37
4	0.500	0.400	195	7.18	4.80	20.5	5.41
5	2.500	1.300	578	10.1	2.13	16.5	4.36
6	5.850	3.250	866	17.1	1.33	9.84	2.60
7	11.59	6.750	1,110	23.1	1.07	6.13	1.52
8	22.86	11.50	1,560	38.6	1.07	5.32	1.41
9	39.68	20.00	2,230	60.9	0.80	4.52	1.20
10	109.9	55.50	4,870	143	0.80	3.46	0.92
11	404.0	205.0	15,690	484	0.53	3.19	0.84
12	1,275	650.0	51,110	1,520	0.53	3.19	0.84

Conversion factors: 1 MGD = 3.785 MLD; 1¢/1,000 gal = 0.264 ¢/1,000 L.

PROBLEMS/QUESTIONS

12.1 Summarize the causes of potential problems for each of the first three treatment elements: chemical feed, rapid mix, and flocculation.

12.2 What are the main parameters that determine the performance of rapid mixer?

12.3 Name the types of devices that are used for rapid mixing.

12.4 Compare and discuss the similarity and differences between the rapid mixing and flocculator systems.

12.5 What are the main variables that determine the performance of flocculation?

12.6 Mixing must be thorough enough to encourage interparticle contact, but gentle enough to prevent disintegration of existing flocculated particles. What are the three basic requirements for flocculation process performance?

12.7 A rapid-mix tank is designed to treat 100,000 m^3/day of turbid water. If the detention time is 30 s and the water operating temperature is 15°C, find

1. The required volume of tank
2. The required mixing power

12.8 A rapid-mix tank is designed to treat 26 MGD of turbid water. If the detention time is 30 s and the water operating temperature is 60°F, find

1. The required volume of tank
2. The required mixing power

12.9 A flocculation basin is 24 m long, 4.5 m wide, and has a working water depth of 3.6 m. The net input power for slow mixing is 0.50 kW. Is this basin adequate for treating 25,000 m^3/day of water for turbidity removal? Assume a water temperature of 10°C.

12.10 A flocculation basin is 80 ft long, 15 ft wide, and has a working water depth of 12 ft. The net input power for slow mixing is 0.65 hp. Is this basin adequate for treating 6.5 MGD of water for turbidity removal? Assume a water temperature of 50°F.

12.11 A water treatment plant has a flow of 30,000 m^3/day. The detention times in rapid-mix and flocculation tanks are 40 s and 30 min, respectively. The water temperature is 20°C. Assume that the optimal velocity gradient for rapid mixing is 800 s^{-1} and for slow mixing is 40 s^{-1}. Determine

1. Size of rapid-mix tank
2. Power requirement for rapid mixing
3. Size of flocculation tank
4. Power requirement for slow mixing

12.12 A water treatment plant has a flow of 8.0 MGD. The detention times in rapid-mix and flocculation tanks are 40 s and 30 min, respectively. The water temperature is 70°F. Assume that the optimal velocity gradient for rapid mixing is 800 s^{-1} and for slow mixing is 40 s^{-1}. Determine

1. Size of rapid-mix tank
2. Power requirement for rapid mixing
3. Size of flocculation tank
4. Power requirement for slow mixing

12.13 A flocculation basin designed to treat 50,000 m^3/day of water is 21 m long, 15 m wide, and 3.60 m deep. The paddle-wheel units consist of four horizontal shafts that rotate at 4 rpm. The shafts are located perpendicular to the direction of flow at mid-depth of the basin. Each shaft is equipped with four paddle wheels 3 m in diameter and each wheel has four blades 3.30 m long and 150 mm wide with two blades located on each side of the wheel. The blades are 300 mm apart. Assume the water velocity to be 30% of the velocity of the paddles, the drag coefficient $C_D = 1.5$, and that the water temperature is 10°C. Determine

1. The power input to the water
2. The velocity gradient
3. The retention time
4. The *Gt* value

12.14 A flocculation basin designed to treat 13 MGD of water is 70 ft long, 50 ft wide, and 12 ft deep. The paddle-wheel units consist of four horizontal shafts that rotate at 4 rpm. The shafts are located perpendicular to the direction of flow at mid-depth of the basin. Each shaft is equipped with four paddle wheels 10 ft in diameter and each wheel has four blades 11 ft long and 6 inches wide with two blades located on each side of the wheel. The blades are 1 ft apart. Assume the water velocity to be 30% of the velocity of the paddles, the drag coefficient $C_D = 1.5$, and that the water temperature is 10°C. Determine

1. The power input to the water
2. The velocity gradient
3. The retention time
4. The *Gt* value

12.15 A hydraulically mixed flocculation basin is to be designed for a water treatment plant that has a capacity of 100,000 m^3/day. The flocculator is to be of an around-the-bend baffled basin. Assume a water temperature of 10°C, a detention time of 30 min, and a velocity gradient of 40 s^{-1}. Determine

1. The required head loss in the channeled basin
2. The required number of channels
3. The basin dimensions

12.16 A hydraulically mixed flocculation basin is to be designed for a water treatment plant that has a capacity of 26 MGD. The flocculator is to be of an around-the-bend baffled basin. Assume a water temperature of 50°F, a detention time of 30 min, and a velocity gradient of 40 s^{-1}. Determine

1. The required head loss in the channeled basin
2. The required number of channels
3. The basin dimensions

12.17 A water treatment plant is designed for a maximum flow of 3.06 MGD. Determine the volume of the flocculation tank and calculate the total horsepower requirement. Assume a velocity gradient, G of 30 s^{-1} and a Gt number of 81,000. $\mu = 2.52 \times 10^{-5}$ lb-s/ft^2.

12.18 A water treatment plant is designed for a maximum flow of 11,500 m^3/day. Determine the volume of the flocculation tank and calculate the total horsepower requirement. Assume a velocity gradient, G of 30 s^{-1} and a Gt number of 81,000. $\mu = 1.2 \times 10^{-3}$ kg/m-s.

12.19 A water flocculation tank using over-and-under baffles is to treat 15 MGD (657 L/s) and is to be 29 ft (8.84 m) long and 13 ft (3.96 m) wide. The water depth is to be 12 ft (3.66 m). The tank has

15 baffles; each is to be 4 in. (100 mm) thick. Assume you as an engineer were asked to check the above flocculation basin design. In your opinion how will this unit perform? Give your reasons and explain why. ($\mu = 2 \times 10^{-5}$ lb-s/ft^2 or 0.456×10^{-3} N-s/m^2 and $k_{\text{bend}} = 2$).

12.20 A flocculation tank using over-and-under baffles is to treat 10 MGD (438 L/s) of water. The tank is 38 ft (11.58 m) long, 10 ft (3.05 m) wide, and has a water depth of 18 ft (5.49 m). The tank has 21 baffles, each being 6 in. (152 mm) thick. Assume you as an engineer were asked to check the above flocculation basin design. In your opinion how will this unit perform? Give your reasons and explain why. ($\mu = 2.3 \times 10^{-5}$ lb-s/ft^2 or 1.099×10^{-3} N-s/m^2 and $k_{\text{bend}} = 3$).

12.21 Determine the grams of dry alum required to make up 10 L (10,000 mL) of 1% (10,000 mg/L) stock solution for feeding to a small laboratory scale experimental water treatment system.

12.22 Determine the volume of a liquid polymer that must be added (a) to prepare 1 L of 0.1% stock solution for a jar test and (b) to prepare 200 gal (757 L) of 10% chemical feed solution for a pilot plant operation. Assume the dry polymer content of the liquid polymer = 0.92 lb/gal = 107.95 g/L.

12.23 Calculate the amount of sodium hypochlorite solution required to feed to a well for disinfection. The well has a casing 6 in. (0.1524 m) in diameter and is 150 ft. (45.72 m) deep. The sodium hypochlorite solution is available as 5.25% chlorine. Considering that disinfection is the final step in completion if well construction, describe briefly that disinfection procedure using sodium hypochlorite solution. Assume the required chlorine dosage for disinfection = 100 mg/L as chlorine.

12.24 The following formula (Percent Solution Method) is also commonly used for liquid chemical's stock solution preparation:

$$V_{\text{lc}} = P_{\text{ss}}(V_{\text{ss}})(D_{\text{w}})/[100\, D_{\text{lc}}],$$

where V_{lc} = volume of liquid chemical that must be added to prepare a desired stock solution or chemical feed solution, L; P_{ss} = desired weight percent stock solution or feed solution, %; V_{ss} = desired volume of stock solution or feed solution, L; D_{w} = weight of water = 8.34 lb/gal =1,000 g/L; and D_{lc} = density of liquid chemical being used, lb/gal or g/L. The liquid chemical can be inorganic (such as liquid alum) or organic (such as liquid polymer). The density of liquid chemical can be obtained from the chemical supplier or manufacture.

Determine the volume of a liquid polymer that must be added (a) to prepare 1 L of 0.1% stock solution for a jar test and (b) to prepare 200 gal (757 L) of 10% chemical feed solution for a pilot plant operation. Assume the density of the liquid polymer = 8.84 lb/gal = 1,060 g/L.

12.25 Assume a 0.1% (1,000 mg/L) alum stock solution is desired, so every mL of stock solution added to a 1-L jar test unit equals 1 mg/L chemical feed dosage. Also assume that the commercial liquid alum purchased has a sp. gr. of 1.3303 g/mL, and a concentration of 5.34 lb/gal (640.94 g/L). Determine the volume of liquid alum required to produce one liter of 0.1% alum stock solution, using the percent solution method.

12.26 Solve Problem 12.5 again using the material balance method

$$(V_{\text{ss}})(C_{\text{ss}}) = (V_{\text{lc}})(C_{\text{lc}}).$$

12.27 Calculate the liquid chemical feeder rate assuming the water treatment plant (WTP) flow = 15 MGD (56.78 MLD), the required potassium permanganate concentration for the taste and odor control = 3 mg/L as $KMnO_4$, and the feed solution in the day tank = 30% as $KMnO_4$.

12.28 Design a mechanical rapid mixing basin assuming

(a) design flow Q = 1.5 MGD = 2.32 ft^3/s = 65.7 L/s

(b) lake water quality: turbidity = 11–61 NTU; natural alkalinity = 28–37 mg/L as $CaCO_3$; temperature = 14–23°C

(c) jar test results: alum dosage = 26–30 mg/L as $Al_2(SO_4)$, 14.3 H_2O; lime dosage = 11–18 mg/L as $Ca(OH)_2$; pH = 6.4–6.8 units

(d) rapid mixing basin requirement: $N = 2$ square basin; G = 700–1,000 s^{-1}; detention time t_{d} = 30 s; mixer motor efficiency = 0.90%

(e) viscosity μ at coldest temperature of 14°C = 1.1748 centipoises = 2.45×10^{-5} lb-s/ft^2 = 1.1748×10^{-3} N-s/m^2.

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Chapter 13

Aeration, Gas Transfer, and Oxidation

13.1 SOURCES OF GASES IN WATER

Within the hydrological cycle, freshwater is exposed to the earth's atmosphere in falling rain and snow, and in runoff from rainfall and snowmelt gathered into brooks and rivers, ponds, lakes, and reservoirs. In reduced volume, freshwaters are exposed also to ground air within the voids of soils through which seepage waters flow. From the free atmosphere, surface waters absorb mainly oxygen and nitrogen; in smaller amounts, carbon dioxide, hydrogen sulfide, and other gases released to the atmosphere (a) by household and industrial operations (mainly the combustion of fuels) and (b) by the respiration of living things ranging from man and the higher animals to the saprophytes responsible for the degradation of organic matter. From the ground air, groundwater may absorb methane, hydrogen sulfide, and large amounts of carbon dioxide, all of them gases of decomposition that accumulate in the ground when plants die, the stubble of crops is left to rot, leaves fall, and organic waste substances are destroyed by bacteria, molds, and other microorganisms of the teeming soil. Concurrently, groundwater may surrender their dissolved oxygen (DO) to the saprophytes. If all of the available oxygen disappears, decomposition becomes anaerobic. Similar changes take place, also, in the stagnant depths of ponds, lakes, and reservoirs and in tidal estuaries in which organic detritus is laid down in benthal deposits.

Some apparently clean ponds and lakes and the backwaters of streams may, at times, support luxuriant growths of algae and related organisms; some of them contain volatile oils responsible for bad odors and tastes. During daylight the algae and other plants or plantlike organisms absorb carbon dioxide from the environment and release oxygen to it in the course of *photosynthesis*. At night, *respiration* reverses the absorption and desorption of these gases.

From what has been said, it is clear that the discharge of putrescible or decomposable organic matter into natural waters by households and industry and its entrance into these waters as decaying vegetation or as fertilizing elements through runoff from agricultural lands increase the aquatic food supply and with it the generation of gases of decomposition, while drawing heavily on available oxygen resources.

13.2 OBJECTIVES OF GAS TRANSFER

Gas-transfer operations serve a multitude of purposes in water treatment, occupy a unique place in water-quality management, and are important factors in the pollution and self-purification of natural waters. Although gas transfer is a physical phenomenon in which gas molecules are exchanged between a liquid and a gas at a gas–liquid interface, the physical operation is accompanied, more often than not, by chemical, biochemical, and biological as well as biophysical changes. These consequences may, indeed, be the primary purpose of the operation. The objectives of gas transfer are correspondingly varied in concept and manifold in the ways of their attainment. In most instances the shared engineering objective of aeration is either the removal of gases and other volatile substances from water or their addition to water, or both at the same time. In some instances, however, air may also be injected into water solely for purposes of agitation. Gas exchange then becomes incidental. Examples are aerated grit chambers and flocculating chambers.

Aeration for gas exchange in its simplest and most direct form has the following aims:

1. Addition of oxygen to oxidize dissolved iron and manganese in waters drawn from the ground
2. Removal of carbon dioxide to reduce corrosion and interference with lime-soda softening
3. Removal of hydrogen sulfide to eliminate odors and taste, decrease the corrosion of metals and disintegration of cement and concrete, and lessen interference with chlorination
4. Removal of methane to prevent fires and explosions
5. Removal of volatile organic compounds (VOCs) and similar odor- and taste-producing substances (a) released by algae and other microorganisms and (b) VOCs from industrial pollution of groundwater.

Water Engineering: Hydraulics, Distribution and Treatment, First Edition. Nazih K. Shammas and Lawrence K. Wang.

Aeration is but one form of gas exchange. There are others in which water is exposed not to natural air but to (a) a specific pure gas, (b) air surcharged with a specific gas, or (c) air or gas, including air–gas mixtures, at pressures above or below atmospheric and possibly at high or low temperatures. Examples are (a) addition of carbon dioxide from flue gas or carbon dioxide generators to *recarbonate* lime-softened water that has purposely been overtreated or to promote or complete the removal of hydrogen sulfide; (b) addition of ozone from ozone generators or chlorine gas from chlorine dispensers (dissolved under vacuum in an ejector and fed as a solution of gas in water), for either the disinfection of waters or the destruction of odors and tastes in waters; and (c) removal of corrosion-promoting oxygen as well as other gases (degasification) by spraying water into a vacuum chamber at ordinary temperatures or at elevated temperatures.

13.3 ABSORPTION AND DESORPTION OF GASES

The absorption and desorption, precipitation, or release of gases by water finds expression in the general gas law:

$$pV = nRT \tag{13.1}$$

where p is the absolute pressure of the gas, V is its volume, n is the number of moles, R is the universal gas constant (8.3136×10^7 dyne-cm per g-mole and °C absolute, or 1,546 lb-ft per lb-mole and °F absolute), and T is the absolute temperature (273.1 + °C) [Kelvin], or (459.7 + °F) [Rankin]. Component and amplifying concepts and formulations comprise Avogadro's hypothesis, laws named after Dalton, Henry, Boyle, Charles, Gay-Lussac, and Graham, and also Boltzmann's molecular constant.

1. Avogadro's law states that equal volumes of different gases at the same pressure and temperature contain the same number of molecules, namely 6.02×10^{23} molecules per mole, a mole being a mass numerically equal to the molecular weight.
2. Dalton's law of partial pressures: $pV = V(p_1 + p_2 + \cdots)$.
3. Henry's Law: See Eq. (13.2)
4. Boyle law: $p_1V_1 = p_2V_2$ at constant T.
5. Charles Law: $V_1/V_2 = T_1/T_2$ at constant p.
6. Graham $k_{d1}/k_{d2} = \sqrt[2]{p_2/p_1}$
7. Boltzmann n in Eq. (13.1)

If attention is directed specifically to water in contact with an atmosphere of air and other gases or gas mixtures, the most important relationships can be summarized as follows:

The *solubility* of a gas depends on (a) its partial pressure in the atmosphere in contact with the water, (b) the water temperature, and (c) the concentration of impurities in the water.

The rate of solution and precipitation of a gas is controlled by (a) the degree of undersaturation or supersaturation of the water, (b) the water temperature, and (c) the interfacial area of gas contact and water exposure, including the prevention, by movement of the atmosphere and the water, of the buildup of stationary gas and water films at the gas–water interface.

The rate of gas dispersion in water depends upon the rate of (a) molecular diffusion, (b) eddy diffusion by convection, and (c) eddy diffusion by agitation.

Implicit in these statements is that the gas does not react chemically with the water. Of the gases in the earth's atmosphere, oxygen, nitrogen, carbon dioxide, and the rare gases are of this kind, although carbon dioxide does react to the extent of about 1% to form carbonic acid (H_2CO_3). Among other gases of significance in natural and treated water and in wastewaters and products of wastewaters such as digesting sludge, methane and hydrogen are inert; hydrogen sulfide is less so; and chlorine is strongly reactive.

In accordance with Dalton's law of partial pressures, the molecules of each gas in a gas mixture exert the pressure they would if they were present alone and the sum of these partial pressures equals the total pressure, that is, $pV = V\Sigma p$. In accordance with Henry's law, moreover, the saturation concentration of a gas in a liquid, such as water, is directly proportional to the concentration, or partial pressure, of the gas in the atmosphere in contact with the liquid, that is,

$$c_s = k_s p \tag{13.2}$$

where c_s is the saturation concentration of the gas in the water, p the partial pressure of the gas in the gas phase, and k_s the proportionality constant or *coefficient of absorption*. The unit of c_s is conveniently mL/L, for p as a proportionality pressure or volume and k_s in mL/L. Atmospheric pressure, namely, 1 atm or 760 mm (29.2 in.) Hg, is commonly specified for k_s, and p is then the pressure in atmospheres. The solubilities of a number of gases important in water and wastewater engineering are shown in Appendix 18. Volumes are converted to weights on the basis of Avogadro's hypothesis that equal volumes of ideal gases contain, at the same temperature and pressure, equal numbers of molecules. At standard temperature and pressure (0°C and 760 mm Hg, or 32°F and 29.92 in. Hg), the molal volume of any gas is 22,412 mL per g-mole or 359 ft^3 per lb-mole. The gas volume at a given temperature and pressure is reduced to standard conditions by means of Eq. (13.3) as

$$V_0 = [(p - p_w)/p_0](T_0/T)V \tag{13.3}$$

Here p, V, and T are as in Eq. (13.1); p_w is the vapor pressure of water; and the subscript zero denotes standard conditions.

Water is saturated with a gas when the proportionality implied in Henry's law is fully effective. Rising temperatures decrease the saturation value, as do the salts of hard and brackish waters. A higher altitude or a falling barometer

EXAMPLE 13.1 CALCULATION OF OXYGEN EQUILIBRIUM CONCENTRATION

What is the equilibrium concentration of oxygen in pure water at 0°C exposed to air under a barometric pressure of 760 mm?

Solution:

From Appendix 18, $k_s = 49.3$ mL/L. Because dry air normally includes 20.95% of oxygen by volume and air in contact with water is generally saturated with water vapor, the partial pressure of oxygen is

$$0.2095 \times (760 - 4.58) = 158 \text{ mm Hg}$$

4.58 being the vapor pressure p_w of water at 0°C from Appendix 4.

Note: 0.06 m of water head at 0°C = 0.2 ft of water head at 32°F = 0.2 × 0.8826 in. Hg. = 4.48 mm Hg. Here 1 ft of water head = 0.8826 in. Hg pressure. The water vapor in mm Hg can also be easily calculated using Equ. 13.7 or 13.8. $P_w = 4.58$ mm Hg at T = 0°C if Equ. 13.8 is used.

The volume concentration of oxygen, therefore, is

$$49.3 \times 158/760 = 10.25 \text{ mL/L}$$

and because 1 mL of oxygen O_2 weighs 2 × 16 × 1,000 mL/22,412 mL = 1.43 mg, the weight concentration of oxygen is

$$1.43 \times 10.25 = \mathbf{14.7\ mg/L}.$$

Note: Compare with the saturation value of oxygen at 0°C and 0 mg/L chloride given in Appendix 19.

reduces solubility in the ratio of observed to standard pressure, the approximate change in pressure with altitude being 1% for every 270 ft (82.3 m) of elevation.

Saturation values for oxygen in fresh and brackish waters at different temperatures are given in Appendix 19. North Atlantic Ocean water of substantially full strength has a chloride content of nearly 18,000 m/L, and its DO saturation is about 82% that of fresh water. The DO saturation of domestic wastewaters is about 95% that of clean water.

Wang and Elmore's (1981) equations are introduced below for calculation of the saturation values of DO in fresh and brackish water under various water temperatures, chloride concentrations, and pressures:

$$c_{sf} = 14.53475 - 0.4024407T + 0.834117 \times 10^{-2}T^2 - 0.1096844 \times 10^{-3}T^3 + 0.6373492 \times 10^{-6}T^4 \qquad \textbf{(13.4)}$$

$$c_{sb} = c_{sf} + \mathrm{CL} \times 10^{-3}(-0.1591768 + 0.5374137 \times 10^{-2}T - 0.1152163 \times 10^{-3}T^2 + 0.1516847 \times 10^{-5}T^3 - 0.8862202 \times 10^{-8}T^4) \qquad \textbf{(13.5)}$$

$$c_{sp} = c_{sb}(P - p)/(760 - p) \qquad \textbf{(13.6)}$$

$$p = 4.571512 + 0.352142T + 0.007386T^2 + 0.000371T^3 \qquad \textbf{(13.7)}$$

where c_{sf} = saturation concentration of DO in fresh water at normal barometric pressure (760 mm Hg) and any temperature, mg/L; c_{sb} = saturation concentration of DO in brackish or fresh water at normal barometric pressure (760 mm Hg), any water temperature, and any chloride concentration, mg/L; c_{sp} = saturation concentration of DO in any water at any barometric pressure, any water temperature, and any chloride concentration, mg/L; T = water temperature, °C; CL = chloride concentration, mg/L; P = barometric pressure, mm Hg; and p = pressure of saturated water vapor, mm Hg.

The following equation is a more accurate water vapor pressure equation also developed by Wang and Elmore (1981):

$$p = 4.581148 + 0.3058575T + 0.1954036 \times 10^{-1}T^2 - 0.7095922 \times 10^{-3}T^3 + 0.3928136 \times 10^{-4}T^4 - 0.5021040 \times 10^{-6}T^5 \qquad \textbf{(13.8)}$$

EXAMPLE 13.2 DETERMINATION OF DISSOLVED OXYGEN (DO) SATURATION VALUE AND % SATURATION

Find the approximate DO saturation value at 10°C and 760 mm of (1) distilled water and (2) salt water containing 18,000 mg/L of chloride; and also (3) the percentage saturation of these waters when their DO content is 5 mg/L.

Solution 1 (Using the Tabulated Data):

From Appendix 4, indirectly, or from Equ. 13.8 directly (T = 10°C) the vapor pressure of water at 10°C is 9.21 mm Hg

1. From Appendix 19 at 10°C and for distilled water, c_s = **11.3 mg/L**.

2. From Appendix 19 at 10°C and by interpolation between 15,000 and 20,000 mg/L chloride concentrations

$$\begin{aligned} c_s &= 8.9 + (9.5 - 8.9) \times 2{,}000/5{,}000 \\ &= 8.9 + 0.3 \\ &= \mathbf{9.2\ mg/L}. \end{aligned}$$

3. For $c_s = 11.3$ mg/L, the percentage saturation is 100 × 5/11.3 = **44.2%**; for $c_s = 9.2$ mg/L, it is 100 × 5/9.2 = **54.3%.**

Solution 2 (Using Wang and Elmore Equations):

The saturated water vapor pressure at 10°C is

$$\begin{aligned} p &= 4.571512 + 0.352142T + 0.007386T^2 + 0.000371T^3 \\ &= 4.571512 + 0.352142(10) + 0.007386(10)^2 + 0.000371(10)^3 \\ &= 9.202532 \text{ mm Hg}. \end{aligned}$$

1. The saturation concentration of DO in distilled water at 10°C is

$$\begin{aligned} c_{sf} &= 14.53475 - 0.4024407T + 0.834117 \times 10^{-2}T^2 - 0.1096844 \times 10^{-3}T^3 + 0.6373492 \times 10^{-6}T^4 \\ &= 14.53475 - 0.4024407(10) + 0.834117 \times 10^{-2}(10)^2 - 0.1096844 \times 10^{-3}(10)^3 + 0.6373492 \times 10^{-6}(10)^4 \\ &= 11.22 \text{ mg/L}. \end{aligned}$$

2. The saturation concentration for DO in brackish water containing 18,000 mg/L of chloride at 10°C is

$$\begin{aligned} c_{sb} &= c_{sf} + \mathrm{CL} \times 10^{-3}(-0.1591768 + 0.5374137 \times 10^{-2}T - 0.1152163 \times 10^{-3}T^2 + 0.1516847 \times 10^{-5}T^3 - 0.8862202 \times 10^{-8}T^4) \\ &= 11.22 + 18{,}000 \times 10^{-3}[-0.1591768 + 0.5374137 \times 10^{-2}(10) - 0.1152163 \times 10^{-3}(10)^2 + 0.1516847 \times 10^{-5}(10)^3 \\ &\quad - 0.8862202 \times 10^{-8}(10)^4] \\ &= 11.22 - 2.07952 \\ &= \mathbf{9.14\ mg/L}. \end{aligned}$$

3. For the distilled water at 10°C, if actual DO = 5 mg/L, the percentage of saturation is

$$100 \times 5/11.22 = \mathbf{44.5}\%.$$

For the brackish water containing 18,000 mg/L chloride at 10°C, if actual DO = 5 mg/L, then the percentage DO saturation is

$$100 \times 5/9.14 = \mathbf{54.7}\%.$$

13.4 RATES OF GAS ABSORPTION AND DESORPTION

If it is postulated that the *rate of gas absorption* is proportional to its degree of undersaturation (or saturation deficit) in the absorbing liquid,

$$\mathrm{d}c/\mathrm{d}t = K_g(c_s - c_t) \qquad \textbf{(13.9)}$$

where dc/dt is the change in concentration, or rate of absorption transport, or transfer, at time t; c_s the saturation concentration at a given temperature; c_t the concentration at time t; and K_g a proportionality factor for existing conditions of exposure. Integration between the limits c_o at $t = 0$ and c_t at $t = t$ then yields the basic equation

$$c_t - c_o = (c_s - c_o)[1 - \exp(-K_g t)] \qquad \textbf{(13.10a)}$$

$$c_t = c_o + (c_s - c_o)[1 - \exp(-K_g t)] \qquad \textbf{(13.10b)}$$

where K_g increases with temperature and the degree of mixing of the gas and liquid, that is, the rate of renewal of the gas–liquid interface and the degree of eddy diffusion. The temperature effect follows the van't Hoff–Arrhenius relationship, but mixing effects are difficult to define unless useful power expenditure can be identified. Because the molecules of gas must pass through the gas–liquid interface, K_g is also a function of A/V, the area of interface per unit volume of liquid. Accordingly $K_g = k_g A/V$, where k_g is the *gas-transfer coefficient*. For absorption, $c_o < c_t < c_s$, and both $c_t - c_o$ and $c_s - c_o$ are positive. For desorption, $c_s < c_t < c_o$, and both $c_t - c_o$ and $c_s - c_o$ are negative. An implicit assumption is that the rate of gas transfer across the gas–liquid interface, rather than the rate of diffusion of the dissolved gas within the liquid, is the controlling factor. Becker reports the following values of $k_g\ \theta^{T}c^{-20}$ in cm/h for the absorption of oxygen, nitrogen, and air from bubbles in the temperature range 3.5–35°C: O_2, $32.3 \times 1.018^{T}c^{-20}$; N_2, $34.0 \times 1.019^{T}c^{-20}$; air,

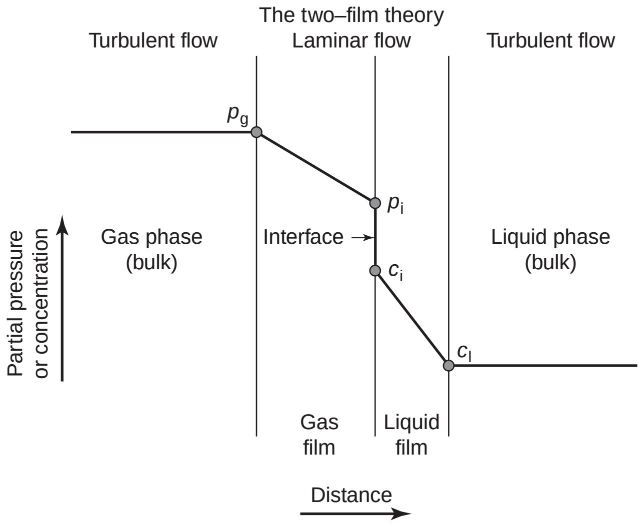


Figure 13.1 Pressure and concentration gradients in gas and liquid films at a gas–liquid interface (After Fair et al., 1971).

$32.1 \times 1.019^{T}c^{-20}$. Understandably, these coefficients apply only for conditions of exposure obtaining in Becker's experiments. Values can be both higher and lower in different circumstances.

According to the two-film theory of Lewis and Whitman (other theories are the penetration theory and the boundary layer theory), boundary films form at the interface within both the liquid and the gas, and rate of passage through them is governed by the thickness of the films (Fig. 13.1). Film thickness itself is fundamentally a function of kinematic viscosity but it can be decreased by stirring or agitating the main body of gas or liquid. The rate of diffusion through the films depends on the area of the interface and the concentration gradient within the component films, or

$$dW/(A\,dt) = -k_{d(g)}(p_g - p_i) = -k_{d(l)}(c_i - c_l) \qquad \textbf{(13.11)}$$

where $dW/(A\ dt)$ is the weight of gas passing through a unit area in a unit time; p_g and p_i are, respectively, the partial pressures of the gas in the main body of the gas and at the interface; c_i and c_l are, respectively, the concentration of the gas at the interface and in the main body of the liquid; and $k_{d(g)}$ and $k_{d(l)}$ are, respectively, the film-diffusion or transfer coefficients in the gaseous and liquid phases. These coefficients possess the dimension of velocity (L/t). Equilibrium is obtained at the interface. Hence p_i is a function of c_i and equals c_i/k_s when Henry's law applies. Three general situations are encountered:

1. The gas is highly soluble in the liquid (e.g., NH_3 in water). In these circumstances c_i is large even when p_i is small. Passage of the gas molecules across the gas film then becomes the controlling factor, and Eq. (13.11) approaches a value of $dW/(A\ dt) = k_{d(g)}p_g$ because p_i is negligible. Hence $dc/dt = k_{d(g)}p_gA/V$, and it follows that gas transfer can be promoted by reducing the thickness of the gas film by moving or stirring the gas.
2. The solubility of the gas in the liquid is low (e.g., O_2, N_2, and CO_2 in water). For these conditions c_i practically equals c_s, the concentration at which the dissolved gas is in equilibrium with the gas in the atmosphere (saturation concentration). Passage of gas molecules through the liquid film is then the controlling factor, and Eq. (13.11) approaches a value of $dW/(A\ dt) = k_{d(l)}\ (c_s - c_t)$, where c_t is the concentration in the liquid. Hence $dc/dt = k_{d(l)}\ (c_s - c_t)A/V$, which is identical with Eq. (13.9). Gas transfer is then promoted by reducing the thickness of the liquid film by stirring or agitating the liquid.
3. For gases of intermediate solubility (e.g., H_2S in water), the effect of both films remains important. Both gas and liquid must, therefore, be stirred or agitated to hold down film thicknesses if gas transfer is to be promoted. So long as Henry's law can be applied with reasonable satisfaction, an overall coefficient k, depending on both film coefficients, can be used, and we may write $dc/dt = -k(c_s - c_t)A/V$.

Observed magnitudes of $k_{d(l)}$ for gases of low solubility are summarized in Appendix 20, together with their molecular diffusion coefficients and calculated film thicknesses.

If the concentration gradient across the interface of a gas bubble is constant, the diameter of the bubble will decrease linearly with time. The rate of decrease is proportional to the gas-transfer coefficient and concentration gradient and inversely proportional to the molecular weight of the gas. Turbulence, temperature, and concentration of dissolved substances all play a part. The higher the concentration of dissolved substances, the lower is the transfer coefficient.

The rate at which gas is absorbed by a falling drop of water or a rising bubble of gas is greatest at the moment of formation. It decreases rapidly thereafter, because the film of water at the interface increases in thickness in the absence of internal liquid motion.

In contrast to absorption, the *rate of desorption*, *precipitation*, *release*, *or dissolution of a gas* from a liquid becomes proportional to its degree of oversaturation in the liquid or the saturation surplus. It follows that the equations for rates of absorption also apply to rates of dissolution. As stated before, the fact that the saturation concentration, c_s, will be less than the observed concentrations, c_o and c_t, makes for negative differences.

EXAMPLE 13.3 COMPUTATION OF TRANSFER COEFFICIENT

In an experiment on the removal of carbon dioxide from water sprayed into the air in spherical droplets 0.55 cm in diameter, the initial supersaturation of the water with carbon dioxide was 27.5 mg/L. After 1 s of exposure this was reduced to 11.5 mg/L. Find the coefficient of gas transfer.

Solution:

Because $c_s - c_o = -27.5\ \text{mg/L}$ and $c_t - c_s = 11.5\ \text{mg/L}$,

$$c_t - c_o = -16\ \text{mg/L}.$$

In accordance with Eq. (13.10a):

$$\begin{gathered} c_t - c_o = (c_s - c_o)[1 - \exp(-K_g t)] \\ (-16) = -27.5[1 - \exp(-K_g \times 1)] \\ \text{Exp}(-K_g) = 1 - 0.582 = 0.418 \\ K_g = 0.872\ \text{s}^{-1}. \end{gathered}$$

The droplet volume per unit surface area being $V/A = \text{D}/6 = 0.55/6 = 9.17 \times 10^{-2}$ cm,

$$k_{d(g)} = K_g \times V/A = 0.872 \times 9.17 \times 10^{-2} = 8.00 \times 10^{-2}\ \text{cm/s}.$$

The transfer coefficient then equals

$$8.00 \times 10^{-2} \times 3{,}600 = \mathbf{288\ cm/h}.$$

13.5 TYPES OF AERATORS

Four types of aerators are in common use:

1. Gravity aerators
2. Spray aerators
3. Diffusers
4. Mechanical aerators

Their aim is to create extensive, new, and self-renewing interfaces between air and water, to keep interfacial films from building up in thickness, to optimize the time of gas transfer, and to accomplish these aims with a minimal expenditure of energy.

13.5.1 Gravity Aerators

Characteristic designs include (see Fig. 13.2) *cascades*, in which the available fall is subdivided into a series of steps; *inclined planes*, usually studded with *riffle* plates set into the planes in herringbone fashion and breaking up the sheet of water that would otherwise form; *vertical stacks*, through which droplets fall and updrafts of air ascend in countercurrent flow; and *stacks of perforated pans or troughs*, often filled with contact media such as coke or stone, the water dropping freely from pan to pan or trough to trough and trickling over the surfaces of the contact media present.

13.5.2 Spray Aerators

Spray or pressure aerators (Figs. 13.3 and 13.4) spray droplets of water into the air from stationary or moving orifices or nozzles. A whirling motion imparted to the droplets makes for turbulence at the air–water interface.

From *orifices* or *nozzles in stationary pipes*, the water rises either vertically or at an angle and falls onto a collecting apron, a contact bed, or a collecting basin serving some other useful purpose. Longer exposure in vertical jets is offset in some measure by freer access of air to the trajectory of inclined jets.

From *orifices* or *nozzles in movable pipes*, the jets issue horizontally from the openings and assist in rotating horizontal, radial pipes suspended from a central inlet column or in propelling horizontal, traveling pipes supported at both ends by carriages running on a rail. Both rotary and rectilinear distributor pipes may be driven hydraulically or electrically.

Stationary orifices and nozzles are common in water purification plants; and movable distributors in wastewater works (trickling filters).

13.5.3 Air Diffusers

Most air diffusers (Fig. 13.5) or injection aerators bubble compressed air into water through orifices or nozzles in air piping, diffuser plates or tubes, or *spargers*. Ascending bubbles acquire smaller terminal velocities than would drop falling freely in air through the same distance. This increases the exposure time of air bubbles but reduces turbulence at the bubble interface. Spiral or cross-current flow can lengthen the path of travel of both air and water. It is conveniently induced by injecting air along one side of the tank.

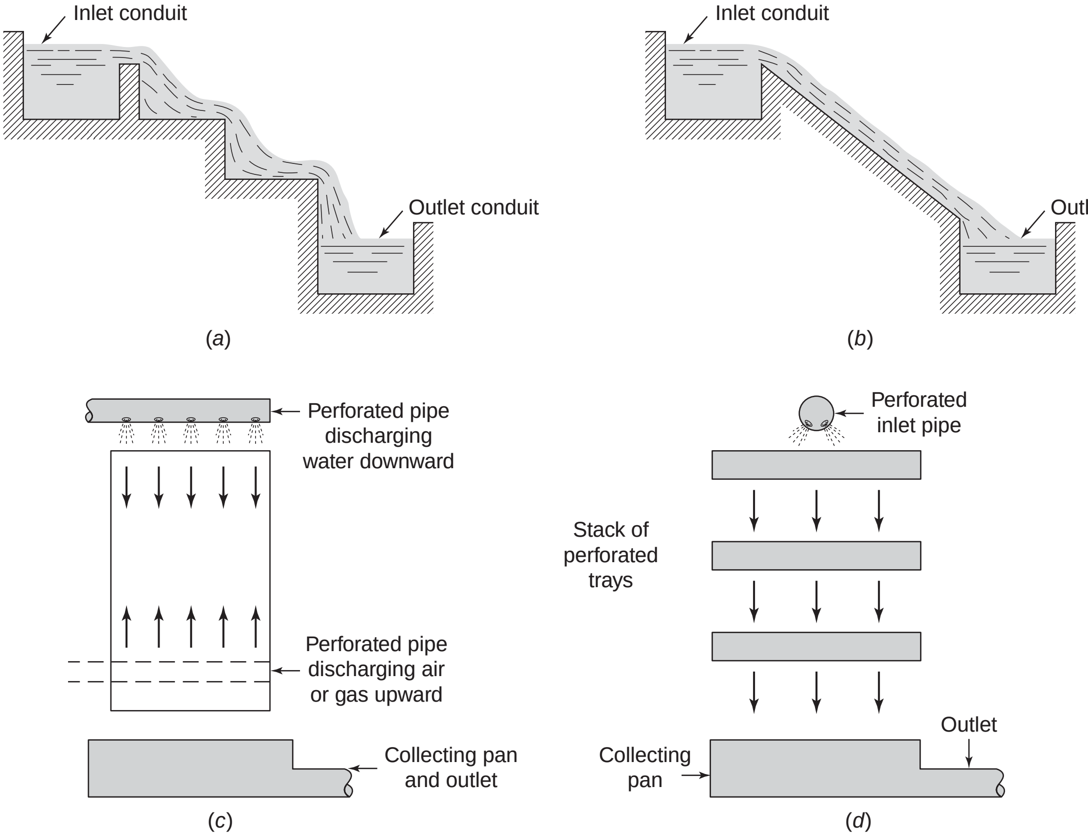


Figure 13.2 Gravity aerators: (a) cascade; (b) inclined apron possibly studded with riffle plates; (c) tower, with countercurrent flow of air (gas) and water; and (d) stack of perforated pans possibly containing contact media (After Fair et al., 1971).

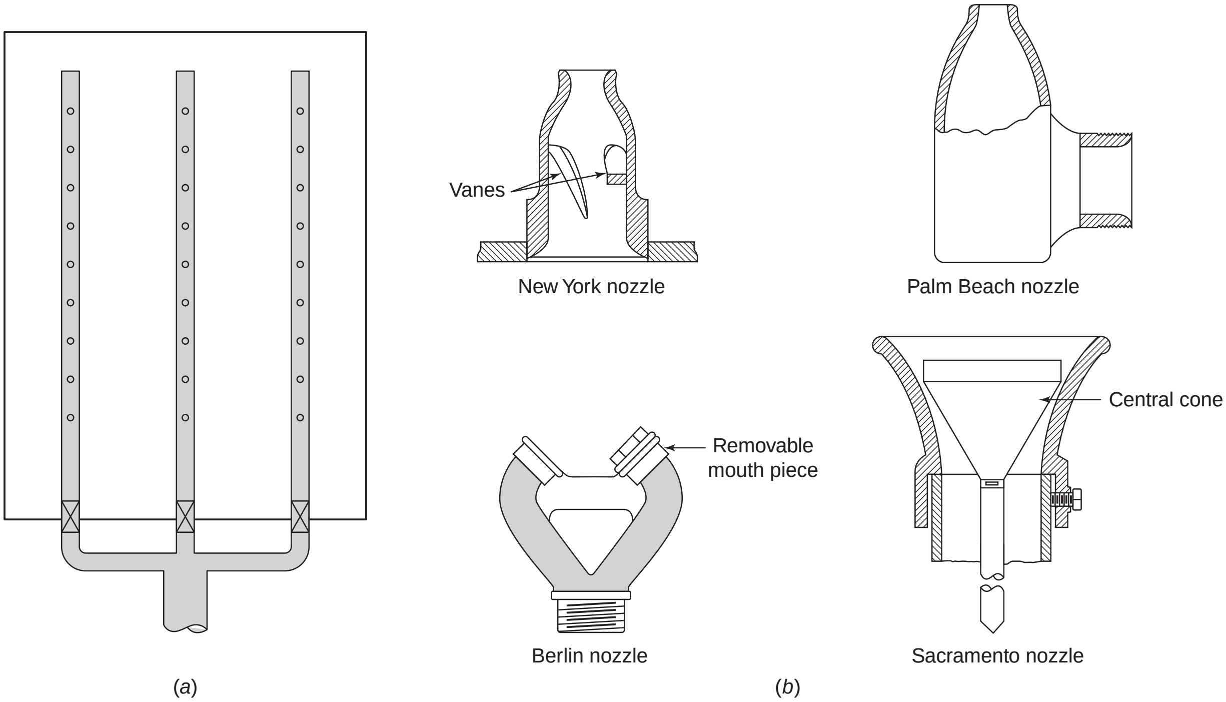


Figure 13.3 Spray aerator and nozzles: (a) nozzled aerator; (b) aerator nozzles. The coefficients of discharge of these nozzles vary from 0.85 to 0.92 (After Fair et al., 1971).

Figure 13.4 Aerator nozzles in lake (Courtesy of Wikipedia; http://en.wikipedia.org/wiki/File:Jet-d%27eau-Gen%C3%A8ve.jpg).

Figure 13.5 Fine bubble membrane diffusers in an aeration tank (Courtesy of Wikipedia; http://en.wikipedia.org/wiki/File:DualAir_Grid_System.jpg).

Air diffusion is employed in water as well as wastewater treatment. Its best known application is in the activated sludge process of wastewater treatment. Less well known is the operation, especially during hot summer months, of floating compressors capable of (a) raising the oxygen content of receiving waters that are overloaded with waste matters and might become septic and (b) destroying their stratification. During the winter season the equipment may be kept at work in inland harbors, where air blown into the waters will prevent the formation of sheet ice and keep the harbor open for shipping.

13.5.4 Mechanical Aerators

Of the many different kinds of mechanical aerators, the following are of special interest to the designer of simple systems: *submerged paddles* that circulate the water in aeration chambers and renew its air–water interface; *surface paddles* (Fig. 13.6) *or brushes* that dip lightly into aeration chambers but far enough to circulate their waters, release air bubbles, and throw a spray of droplets onto their water surface; *propeller blades* that whirl at the bottom of a central downdraft tube in an aeration chamber and aspirate air into the water; and *turbine blades* that cap a central updraft tube in an aeration chamber and spray droplets over its water surface.

Mechanical aerators of these kinds are employed principally in the treatment of wastewaters by the activated sludge process. Here a function of mechanical as well as injection aeration as fully as important as that of gas transfer is to keep the activated floc in mobile and useful suspension.

Unless they are housed, gravity and pressure aerators for water works may have to be bypassed in winter to keep the water from freezing. Wastewater, however, is normally warm enough not to congeal during the brief interval of its exposure in advance of trickling filtration. During high winds, rising jets of water may have to be throttled down or shut off entirely to keep their spray within the boundaries of the aerator. Aerator spaces, especially enclosed spaces, should be well ventilated not only to create effective differentials in gas concentrations between the two phases, but also to prevent (1) asphyxiation of operating or repair crews and visitors by

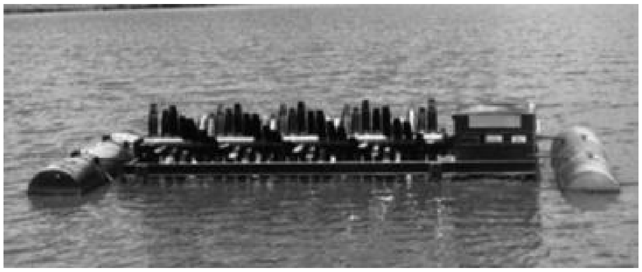


Figure 13.6 Paddlewheel aerator (Courtesy of Southern Regional Aquaculture Center).

carbon dioxide, (2) their poisoning by hydrogen sulfide, and (3) formation of explosive mixtures of methane with air.

13.6 FACTORS GOVERNING GAS TRANSFER

What has just been said about aerators and their aims is supported most immediately by the common gas-transfer equations. In accordance with Eq. (13.10a), for example,

$$c_t = c_o + (c_s - c_o)\{1 - \exp[-k_g(A/V)t]\} \qquad \textbf{(13.10b)}$$

transfer can be optimized, no matter what its direction by

1. Generating the largest practicable area, A, of interface between a given water volume, V, and air, a pure gas, an air–gas mixture, or a mixture of gases
2. Preventing the buildup of thick interfacial films or by breaking them down to keep the transfer coefficient, k_g, high
3. Inducing as long a time of exposure, t, as possible
4. Ventilating the aerator and its components well enough to maintain the highest possible driving force or concentration difference, $(c_s - c_t)$ for absorption and $(c_t - c_s)$ for desorption

Values of the proportionality constant $K_g = (A/V)k_g$ must normally be determined experimentally and verified in plant-scale tests. A plot of $y = \log(c_t - c_s)$ against $x = t$ should yield a straight line with an intercept $y_o = \log(c_o - c_s)$ at $t = 0$ and a slope $(\log y_2 - \log y_1)/(t_2 - t_1) = K_g$.

13.7 DESIGN OF GRAVITY AERATORS

The controlling element in gravity aerators (Fig. 13.2) is the available head. It can be put to use in a single or in multiple descent. At a given instant the rate of free fall $dh/dt = v = gt$ and $\int_o^h dh = g\int_o^h t\,dt$, or $h = {}^1\!/_2\, gt^2$, where h is the height of fall in ft, t the time in seconds, v the velocity in ft/s, and g the acceleration of gravity = 32.2 ft/s^2. It follows that, in a single descent through a height h, the elapsed time is $t = \sqrt[2]{2h/g}$ and that in n descents through the same vertical distance

$$t = n\sqrt[2]{2h/ng} = \sqrt[2]{2nh/g} \qquad \textbf{(13.12)}$$

In other words, t is proportional to $\sqrt[2]{n}$.

When Eq. (13.12) is used in SI units, t = time, s; h = height of fall, m; g = acceleration of gravity, 9.81 m/s^2; and n = number of descents.

However, it should be said that the quality of exposure is usually poorer in multiple descents, because droplets do not necessarily break away from jets of falling water as soon as they strike the air.

EXAMPLE 13.4 DESIGN OF MULTIPLE DESCENTS GRAVITY AERATOR

Find the time of exposure of water falling through a distance of 9 ft (2.74 m) (1) in single descent and (2) in four descents.

Solution 1 (US Customary System):

For $n = 1$ and $h = 9$ ft, Eq. (13.12) states that

$$t = \sqrt[2]{2nh/g}$$

$$t = \sqrt[2]{(2)(1)(9)/32.2}$$

$$= \mathbf{0.75\ s}.$$

For $n = 4$ and $h = 9$ ft

$$t = \sqrt[2]{(2)(4)(9)/32.2}$$

$$t = \mathbf{1.5\ s}.$$

Solution 2 (SI System):

For $n = 1$ and $h = 2.74$ m, Eq. (13.12) states that

$$t = \sqrt[2]{2nh/g}$$

$$t = \sqrt[2]{(2)(1)(2.74)/9.81}$$

$$= \mathbf{0.75\ s}.$$

For $n = 4$ and $h = 2.74$ m

$$t = \sqrt[2]{(2)(4)(2.74)/9.81}$$

$$t = \mathbf{1.5\ s}.$$

13.8 DESIGN OF FIXED-SPRAY AERATORS

The hydraulic performance of fixed-spray pressure aerators involves three principal parts: orifice or nozzle behavior, including applicable ballistic principles; windage or wind effects, shared with spray cooling; and pipe friction associated with multiple takeoffs along the line of flow, shared with multiple tank inlets and filter underdrains.

Wind effects are variable. However, even relatively gentle winds are more influential than air resistance, which does not reduce the height or time of rise by as much as 10% for heights under 15 ft (14.57 m) and exposures under 2 s. That is why the resistance offered by calm air can usually be neglected in aerator calculations. The time of exposure also governs the distance droplets are carried by the wind because

$$l = 2C_D v_W t_r \tag{13.13}$$

where l = the distance, ft or m; C_D = the coefficient of drag (about 0.6), dimensionless; and v_w = the wind velocity, ft/s or m/s; and t_r = the time of rise, s.

As shown in Fig. 13.7, flow through a perforated or nozzled pipe decreases stepwise at each opening or takeoff. If the diameter remains unchanged, the resistance to flow within the perforated section equals approximately the resistance that would be offered to the full entrant flow by one-third the length of perforated pipe. Thus for the idealized case of a slotted pipe and $s = kQ^n$, the flow at a distance $(L - l)$ from the end of the pipe is $Q = Q_e\ (L - l)/L$ and

$$h_f = \int_o^l s\,dl$$

$$h_f = k(Q_e/L)^n \int_o^l dl(L - l)^n$$

$$h_f = (S_e/L^n) \int_o^l dl(L - l)^n$$

$$h_f = [S_e/(n + 1)][L - (L - l)^{n+1}/L^n] \tag{13.14}$$

Here the subscript e denotes entrant flow and resistance to it. For the Chezy formula ($n = 2$) and $l = L$, $h_f = 1/3\ s_e L$, as stated at the outset and implied also geometrically by the parabolic nature of the curve for s versus l. (The reader will remember that the area inside a parabola equals two-thirds the area of the circumscribed rectangle. This makes the area outside the parabola one-third the area of the pertinent rectangle.) Actual losses may be reduced by the recovery of velocity head.

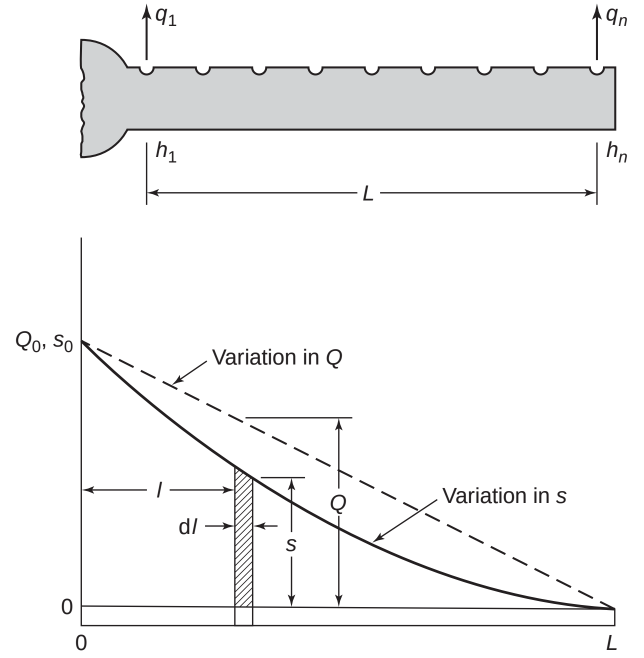


Figure 13.7 Frictional resistance to uniformly decreasing flow (idealized) (After Fair et al., 1971).

For good ventilation, sprays should not overlap more than necessary to ensure spatial economy. In water purification works, nozzles are commonly placed 2–12 ft (0.61–3.66 m) on centers, and nozzled aerators occupy 50–150 ft^2 of area per MGD capacity (1.23–3.68 m^2/MLD).

13.9 DESIGN OF MOVABLE-SPRAY AERATORS

The hydraulic performance of *movable-spray aerators* is governed by the power required to revolve rotary distributors or keep straight-line (traveling) distributors in motion, and the sizing of nozzles and fanning out of sprays for uniform distribution of the applied waters, both of them under varying hydraulic loadings. Neither ballistic nor wind effects are important, because the sprays issuing horizontally lie only 9–15 in. (230–380 mm) above the surface of the trickling filters they commonly serve.

Distributors become self-propelling under hydrostatic heads of 18–30 in. (457–762 mm). When fully effective, the impulse transmitted by the jets to the distributor is $(\gamma Q/g)v = \gamma Q c_v \sqrt[2]{2h/g}$ = 28–36 ft-lb/s at a flow rate of 1 MGD, that is, about 0.05–0.07 hp or (10–13 N-m/s at a flow of 1 MLD). Losses in supply piping within the sprinkling unit itself together with mechanical friction in moving parts are estimated at 25%. When, in the absence of recirculation, flows (night flows, for instance) drop and become so weak that they cannot keep the distributor in motion, a dosing tank must be interpolated to accumulate and discharge minimal operative flows. Otherwise the distributor must be driven by a motor.

13.10 DESIGN OF INJECTION AERATORS

The air supply of injection aerators is made available from the ambient atmosphere through (a) air filters that clean incoming air to protect compressors against abrasion and air-distribution systems against fouling and clogging; (b) air compressors that place the air under wanted pressure; (c) measuring devices (orifice or venturi meters) that measure and often record rates of air flow; (d) air piping that conveys the compressed air to points of use; and (e) diffusers, spargers, nozzles, or orifices through which the air is injected into the waters under treatment. Economical designs normally hold friction losses in air supply systems to about 25% of the depth of submergence of the terminal air-distribution units. Accordingly, the air pressure at the blowers (psig) approximates 0.54 × distribution depth (ft). Because the terminal velocity of rising bubbles is close to 1 ft/s (0.30 m/s), the time of exposure in seconds is numerically much the same as the depth in ft.

Aeration tanks are commonly 10–15 ft (3–4.5 m) deep and can maintain good transverse (often spiral) circulation at ratios of tank width to tank depth up to 2:1, if the cross-section is shaped to avoid stagnant corners. The nominal detention period of aeration units is least in water purification operations (usually 10–30 min), most in biological wastewater treatment (seldom as low as 1 h, usually up to 6 h, and sometimes up to 24 h on an average). The area (ft^2) occupied by tanks aerating 1 MGD of water is normally 9,300/(*dt*), where *d* is the depth in ft and *t* the time in min. Applied air ranges from 0.01 to 0.15 ft^3 of free air per gal of water (0.075–1.12 m^3/L) being purified, but may exceed 1.0 ft^3/gal (7.5 m^3/L) in the treatment of wastewaters.

The set of design equations for the injection aerators are summarized below:

$$P_g = 0.54D_d \quad \text{(US customary units)} \qquad \textbf{(13.15)}$$

where P_g = the gauge air pressure at the blowers, psig; D_d = the distribution depth, ft.

$$P_g = 12.3D_d \quad \text{(SI units)} \qquad \textbf{(13.16)}$$

where P_g = the gauge air pressure at the blowers, kPa; D_d = the distribution depth, m.

$$A/Q = 9{,}300/dt \quad \text{(US customary units)} \qquad \textbf{(13.17)}$$

where A = the area, ft^2; Q = the water flow, MGD; d = the aeration depth, ft; t = time, min; and A/Q = the square feet of aeration area required for aerating 1 MGD of water.

$$A/Q = 69.6/dt \quad \text{(SI units)} \qquad \textbf{(13.18)}$$

where A = the area, m^2; Q = the water flow, MLD; d = the aeration depth, m; t = time, min; and A/Q = the square meters of aeration area required for aerating 1 MLD of water.

EXAMPLE 13.5 DETERMINATION OF GAUGE PRESSURE OF AIR BLOWERS

Determine the gauge air pressure at the blowers for an injection aerator system assuming the distribution depth is 7.5 ft (2.29 m).

Solution I (US Customary System):

$$\begin{aligned} P_g &= 0.54D_d \\ &= 0.54(7.5) \\ &= \mathbf{4.05\ psig}. \end{aligned}$$

Solution II (SI System):

$$\begin{aligned} P_g &= 12.3D_d \\ &= 12.3(2.286) \\ &= \mathbf{28.12\ kPa}. \end{aligned}$$

EXAMPLE 13.6 DESIGN OF INJECTION AERATION SYSTEM

Determine the area requirement of an injection aeration system. Assume the aeration depth is 8 ft (2.44 m) and the aeration time is 2 hours.

Solution I (US Customary System):

$$\begin{aligned} A/Q &= 9{,}300/dt \\ &= 9{,}300/(8 \times 2 \times 60) \\ &= \mathbf{9.69\ ft^2/MGD}. \end{aligned}$$

Solution II (SI System):

$$
\begin{aligned}
A/Q &= 69.6/dt \\
&= 69.6/2.44 \times 2 \times 60 \\
&= \mathbf{0.24\ m^2/MLD}.
\end{aligned}
$$

13.11 MECHANICAL AERATORS

Mechanical surface aerators are widely used for many applications in modern wastewater treatment. They transfer atmospheric oxygen to the liquid by surface renewal and exchange and provide the mixing necessary for distribution of the aerated liquid throughout the tank and for maintaining suspended solids in suspension where this is necessary. Mechanical aerators have found favor because they dispense with the compressors, air-cleaning devices, piping, and diffusers, and their maintenance, which are necessary with injected-air systems, and because they fit into a wide variety of tank shapes, not being bound to the depth and width strictures common with diffused air systems.

Various types of vertical-shaft mechanical aerators have been developed in addition to the horizontal shaft brush aerator and cage rotors developed in Europe. Many configurations operate at the surface of the tank, spraying the liquid into the air by means of blades, or entraining air by creating vortices. Such units are limited to applications in relatively shallow tanks. Incorporating a draft tube on a spray aerator allows the unit great latitude in the depth of tank into which it is fitted. Downdraft tube aerators operate by entraining air at the surface and discharging the air–liquid mixture at the bottom of the tank. Turbine mixers immersed in the liquid have been combined with air spargers set underneath the turbine, the turbine serving to shear the large air bubbles emitted from the sparger.

The oxygen-transfer efficiency of mechanical aerators, about 3 lb of oxygen/hp/h (0.61 kg/kW/h), is of the same order of magnitude as for diffused air systems, except that the efficiency of vertical-shaft aerators tends to drop with larger installations. Full-scale testing of mechanical aerators is necessary for untried units, with performance specifications carefully spelled out. Such tests, however, because they use water rather than wastewaters, may not always provide data directly applicable in a treatment plant. Model tests may be helpful in assessing the significance of liquid characteristics on aeration efficiency.

13.12 OXIDATION FOR REMOVAL OF DISSOLVED IRON AND MANGANESE

Iron and manganese are major components of the earth's crust. They occur naturally in groundwater and are common constituents of acid-mine drainage and industrial wastewaters. In water distribution systems iron makes its appearance as a product of corrosion. In limnology and oceanography iron and manganese are essential elements of plant nutrition. Neither iron nor manganese is very soluble in water. Iron(III) oxide is the cause of *red water* in distribution systems; manganese oxides are the cause of *brown* or *black water*. Both make water unsuitable for laundering, and for dyeing, papermaking, and other manufacturing processes.

13.12.1 Solubility of Fe and Mn

Within the pH range of natural waters, soluble bivalent iron and manganese consist predominantly of Fe^{2+}, Mn^{2+}, FeOH^{+}, and MnOH^{+}. Within the common pH range (6–9) of carbonate-bearing waters, their solubility in the bivalent oxidation state is governed generally by the solubility products of their carbonates, not their hydroxides. Thus maximum soluble [Fe(II)] or [Mn(II)] depends on pH and bicarbonate content. Using the appropriate equilibrium constants of Appendixes 21 and 22, a solubility diagram for iron(II) and manganese(II) has been constructed in Fig. 13.8. Small

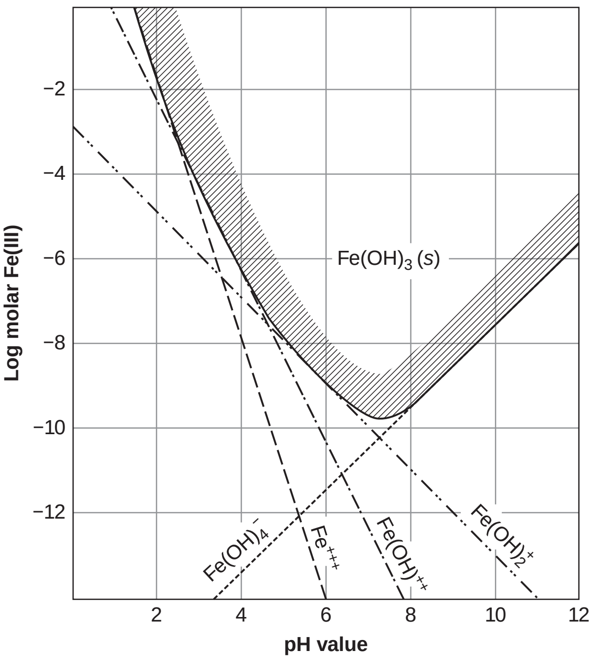


Figure 13.8 Solubility of Fe(III) at 25°C. Various Fe(III) components are in equilibrium with solid $\mathrm{Fe(OH)_3}$. The line surrounding the shaded area identifies total soluble iron(III) as $[\mathrm{Fe}^{3+}] + [\mathrm{Fe(OH)}^{2+}] + [\mathrm{Fe(OH)}^{2+}] + [\mathrm{Fe(OH)}_4^{-}]$. The solubility contribution by $\mathrm{Fe_2(OH)_2}^{3+}$ is negligible above pH 4 and is not included here (After Fair et al., 1971).

quantities of sulfide (H_2S, HS^-, S^{2-}) can limit Mn(II) and Fe(II) solubility even more than carbonate. The effect of hydrolysis, that is, the formation of $FeOH^+$ and $MnOH^+$, has been neglected because it becomes significant only above pH 10.

The solubility of Fe(III) in natural waters is generally governed by that of ferric hydroxide, $Fe(OH)_3$, or ferric oxide hydroxide, FeOOH. To evaluate it, the formation constants for soluble hydroxo-ferric complexes, $FeOH^{2+}$, $FeOH_2{}^+$, $Fe_2(OH)_2{}^{4+}$, and $Fe(OH)_4{}^-$, must be considered together with the solubility product of hydrous ferric oxide. The resulting solubility relations are shown in Fig. 13.8. Within the common pH range, total soluble Fe(III) is seen not to exceed concentrations of approximately 10^{-3} mg/L. The solubility of MnO_2 is even lower than that of hydrous ferric oxide, and no soluble Mn(IV) can be detected within the pH range 3–10. In the absence of strong complex formers, Mn(III) does not occur as a dissolved species.

13.12.2 Redox Reactions of Fe and Mn

A comparison of the solubility relations of the bivalent metal ions with those of higher valent oxidation states shows that oxidation of the ferrous and manganous ions to ferric oxide and higher valent manganese oxides will render them insoluble. In the presence of DO, both Mn(II) and Fe(II) are thermodynamically unstable at all pH values of natural waters:

$$2Fe^{2+} + {}^1\!/\!_2O_2 + 5H_2O \rightarrow 2Fe(OH)_3(s) + 4H^+ \quad (\log K = 5.8;\ 25°C) \qquad \textbf{(13.19)}$$

$$Mn^{3+} + {}^1\!/\!_2O_2 + H_2O \rightarrow MnO_2(s) + 2H^+ \quad (\log K = 0;\ 25°C) \qquad \textbf{(13.20)}$$

The oxygenation (oxidation by oxygen dissolved in water) reactions are accompanied by a reduction in pH. Redox potential data suggest that all organic and some inorganic substances (e.g., sulfide) are potential reductants for both Fe(III) and Mn(IV); equilibrium relations suggest that soluble iron and manganese occur in water only in the bivalent state and in the absence of oxygen. Thus iron and manganese are found in oxygen-free groundwater. Anaerobic, hypolimnetic layers of lakes and reservoirs may also hold substantial quantities in solution. Biological activity may influence the chemical reactions indirectly, for example, by creating metabolites, capable of reducing $Fe(OH)_3$ or MnO_2. Some bacteria and algae are capable of depositing manganese and ferric oxides from ferrous and manganous solutions. To be kept in mind in this connection is that organisms influence only the kinetics and not the thermodynamics of reactions.

13.12.3 Precipitation of Fe and Mn

If oxygen is kept out of the system, Fe(II) and Mn(II) can be precipitated in the same way as is calcium in lime-soda softening. Substantial reduction in soluble Fe^{2+} and Mn^{2+} can be achieved by increasing the carbonate content and pH of the water at the same time, for example, by addition of soda ash. For economic reasons the use of lime and soda ash for deferrization and demanganization is commonly restricted to hard waters that are also to be softened. Iron and manganese precipitates are removed effectively by filtration. Enclosed pressure filters generally serve this purpose.

Oxidation to insoluble ferric or manganic oxides can be accomplished with a variety of oxidants, among them in addition to DO are permanganate, ozone, and chlorine. Only 0.14 and 0.27 mg/L of O_2 are needed for the oxidation of 1 mg of Fe(II) and Mn(II), respectively. The higher valent oxides precipitated are removed by sedimentation or filtration.

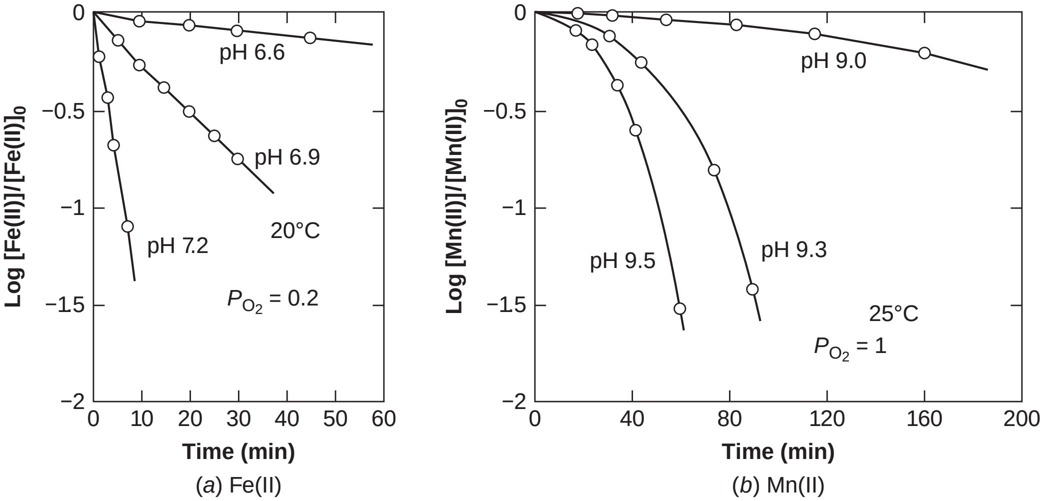
13.12.4 Kinetics of Oxygenation

Much of what has been said is based on thermodynamic considerations, but these explain only the potential behavior or iron and manganese. For an understanding of the kinetics of redox reactions, the actual behavior of these reactions (a) in different natural systems and (b) during deferrization and demanganization must also be examined. Representative kinetic experiments are brought together in Fig. 13.9. It is evident that the reaction rates are strongly pH dependent. Oxygenation of Fe(II) is seen to take place only slowly, especially below pH 6.5. Oxidation of Mn(II) is still slower and becomes measurable only above pH 8.5. The rate of reaction of ferrous iron rises a 100-fold with a unit increase in pH. Catalysts (especially Cu^{2+} and Co^{2+}), as well as anions that form complexes with Fe(III) (e.g., $HPO_4{}^{2-}$), also speed up the reaction rate significantly.

Slowly formed MnO_2 becomes a sorbent for Mn^{2+} ions. Consequently the oxidation products of manganese oxygenation are generally nonstoichiometric compounds in various average degrees of oxidation ranging from $MnO_{1.3}$ to $MnO_{1.9}$ (30–90% oxidation to MnO_2). The relative proportions of Mn(II) and Mn(IV) in the solid phase depend strongly on pH and other variables. The suggested reaction pattern tells why a substantial fraction of the Mn(II) removed from solution is not oxidized and explains the autocatalytic nature of the reaction. Filter sands coated with MnO_2 can be effective catalysts for $[Mn^{2+}]$ oxygenation.

13.12.5 Engineering Management of Oxidative Removal of Iron and Manganese

Oxidation of Fe(II) and Mn(II) is but one step in the removal process. Coagulation and sedimentation or filtration codetermine the overall removal rate or the efficiency of removal of these constituents. It can be inferred from the kinetics of



(*a*) Fe(II) (*b*) Mn(II)

Figure 13.9 Oxygenation of iron(II) and manganese(II) in bicarbonate solutions (After Fair et al., 1971).

oxygenation that the oxidation reaction may be rate controlling up to pH 7 for ferrous iron and pH 8.5 for manganous manganese.

Coagulation of iron(III) and manganese(IV) oxides is fast within neutral or slightly acid pH ranges, and removal of the oxides by sand filtration is best within the same range. In such circumstances catalysts may be of assistance—for example, Cu^{2+}, which hastens the oxygenation of Fe(II); or oxidants, such as chlorine or permanganate, both of which oxidize Fe(II) and Mn(II) readily even in the acid pH range. Hydrous oxides of Fe(III) and Mn(IV) have high sorption capacities for bivalent metal ions. Both hydrous MnO_2 and $Fe(OH)_3$, for instance, tend to sorb Mn^{2+} and Fe^{2+} ions. Sorption capacities for Mn^{2+} at pH 8 are on the order of 1.0 and 0.3 mole of Mn(II) sorbed per mole of MnO_2 and $Fe(OH)_3$, respectively. The tendencies for sorption of Mg^{2+} and Ca^{2+} ions are significantly smaller. Sorption of Fe(II) and Mn(II) onto both ferric and manganic oxide is an important feature of oxidative deferrization and demanganization. An example is the easier removal of Mn(II) from water containing substantial amounts of iron $\{[Fe(II)] > [Mn(II)]\}$ rather than Mn(II) alone. Removal then becomes predominantly a matter of sorption of Mn^{2+} on incipient precipitates of ferric oxide. In practice, removal of iron and manganese is generally hastened and made more efficient by letting water trickle downward over coke or crushed stone or rise upward through gravel or other relatively coarse, heavy materials. As the contact interfaces become coated with hydrous oxides of Fe(III) or Mn(IV), the removal of Mn(II) and Fe(II) by sorption becomes swifter and more complete. Sand and anthracite filters, too, must *mature*, that is, become coated with hydrous oxides, if they are to remove iron and manganese effectively. The alkalinity of limestones, dolomites, and magnesium oxides makes them not only useful contact media during the oxygenation of Mn(II) and Fe(II), but also powerful sorbents for unoxidized Mn(II) and Fe(II) after the surfaces have become coated with the products of oxidation. In a similar fashion, diatomaceous earth, clays, or zeolites that have been coated with MnO_2 by treating them with Mn^{2+} and MnO_4^- provide excellent interfaces for iron and manganese removal. When the sorption capacity of surfaces of this kind has been exhausted, they can, in a sense, be *regenerated*. Permanganate will do this, that is,

$$3\{Mn(II) \bullet MnO_2\}(s) + 2MnO_4^- + 2H_2O \rightarrow 8MnO_2(s) + 4H^+ \tag{13.21}$$

Sedimentation of oxide precipitates washed or escaping from the contact units can be speeded and improved by coagulation. Indeed, small amounts of iron and manganese will be removed from some waters by cationic polyelectrolytes, alum, and iron(III) alone.

EXAMPLE 13.7 APPLICATION OF AERATION TO A WATER TREATMENT PLANT FOR IRON AND MANGANESE REMOVAL FROM GROUNDWATER

Iron and manganese occur naturally in water, especially groundwater. Although soluble iron and manganese in water cause no adverse health effects, excessive amounts of iron and manganese can add a rusty taste and color to the water and can stain clothes and plumbing fixtures. In the presence of DO, soluble iron and manganese are converted to insoluble rusty ferric oxide and insoluble black manganese oxide, respectively. Aeration provides the DO needed to convert the soluble iron and manganese to their insoluble oxidized ferric and manganic forms. Since saturated DO in water is corrosive to the water distribution system, aeration is usually applied to a water treatment system before chemical feeders and rapid-mixing basin, as shown in Fig. 11.1. A filtration step is needed to remove the insoluble ferric oxide and manganese oxide. The US EPA secondary standards (aesthetic, not health related) for iron in drinking water is 0.3 mg/L, and for manganese in water is 0.05 mg/L. Recommend a groundwater treatment system for iron and manganese removal by aeration and filtration, assuming the raw ground water contains 0.8 mg/L of soluble iron and 0.2 mg/L of soluble manganese. All other water quality parameters (except coliforms) meet the drinking water standards.

Solution:

It requires 0.14 mg/L of DO to oxidize 1 mg/L of soluble iron, and 0.27 mg/L of DO to oxidize 1 mg/L of soluble manganese. The detention time before filtration should be at least 20 minutes for the aeration/oxidation reactions. The recommended groundwater treatment system for mainly removing iron and manganese, coliforms inactivation, and corrosion control should include the well field, pumping, aeration/oxidation, pressure filtration, corrosion control, disinfection (chlorination), clearwell, high service booster pumping, pressure tank, and water storage and distribution system. The flow diagram is similar to the one shown in Fig. 11.10, although screening is optional.

13.13 REMOVAL OF SPECIFIC GASES

Methane, carbon dioxide, and hydrogen sulfide are found in water as well as wastewater. The order of their respective solubilities is relatively low, moderately high, and very high.

13.13.1 Methane

Methane, CH_4, also called marsh gas and methyl hydride, is the simplest member of the paraffin hydrocarbons. In nature, methane is a common constituent of natural gas from oil and gas wells and of fire damp in coal mines. Accordingly, it may be present in deep well and mine waters. In wastewater treatment, it is the most common component of sludge gas.

A gas like methane, which is only slightly soluble in water, does not react with it, has a low boiling point, is normally absent from the ambient atmosphere, and is readily swept out of water by aeration. Good ventilation will promote its desorption and prevent the accumulation of explosive air–methane mixtures. The explosive range extends from 5.6% to 13.5% of methane by volume.

13.13.2 Carbon Dioxide

The removal of carbon dioxide from water is more complex than that of methane. The solubility of carbon dioxide, CO_2, is high (Appendix 18), and somewhat less than 1% of it reacts, at ordinary pressures and concentrations, with water to form carbonic acid, H_2CO_3, and its ionization products HCO_3^- and CO_3^{2-}. However, the boiling point of CO_2 is low, and its concentration in normal atmospheres is small. In some waters H_2CO_3 may be the only acid present; in all waters it has a strong influence on the hydrogen ion concentration, which is lowered as the gas is removed. Neither H_2CO_3 nor its ionization products can be desorbed as such by aeration.

The partial pressure of CO_2 in the atmosphere is normally about 0.033%, making for a saturation value of 0.56 mg/L in water at 20°C (78°F) and atmospheric pressure. Good ventilation will promote desorption of the gas and prevent its accumulation in asphyxiating concentrations. Exposure of water droplets to air for 2 s ordinarily lowers the CO_2 concentration in the water by 70–80%, and efficiencies as high as 90% are not unusual.

As a means of CO_2 removal, aeration enters into competition with the neutralization of CO_2 by lime (CaO), especially in conjunction with the lime-soda process of water softening. Because the rate of CO_2 desorption varies directly with its supersaturation in water, aeration is particularly efficient at high concentrations. By contrast, the efficiency of chemical neutralization is substantially independent of concentration. For this reason treatment may be optimized by aerating at high concentrations and liming at low concentrations. The dividing line generally lies at about 10 mg/L. Where water must be pumped for aeration, comparisons should be based on power costs versus chemical costs. Soda ash (Na_2CO_3) can take the place of lime, but it is more expensive in equivalently effective amounts.

13.13.3 Hydrogen Sulfide

The desorption of hydrogen sulfide, H_2S, is even more complex than the removal of CO_2. The solubility of H_2S is more than twice as high (Appendix 18), and the dissolved gas ionizes to form the sulfides HS^- and S^{-2}, neither of which can be removed directly by aeration. Moreover, aeration desorbs CO_2 faster than it does H_2S, and the consequent rapid rise in pH affects the ionization equilibria of H_2S and steps up the HS^- and S^{-2} concentration. A further complication is that the oxygen introduced by aeration reacts with H_2S to throw down elemental sulfur:

$$2H_2S + O_2 \rightarrow 2H_2O + 2S\downarrow + 2 \times 3{,}300 \text{ cal} \quad \textbf{(13.22)}$$

To be sure, there is a decrease in H_2S, but the sulfur remaining in the water exerts a chlorine demand. Usually this is undesirable. Moreover, the sulfur may be oxidized by aeration to sulfite,

$$2S + 3O_2 \rightarrow 2SO_3^{2-} + 2 \times 4{,}540 \text{ cal} \quad \textbf{(13.23)}$$

and it may be reduced back to H_2S when the water becomes anaerobic—for example, in the dead-ends of water distribution systems and in parts of wastewater systems, such as force mains. Elemental sulfur, which imparts a milky-blue cast to water, can be removed by chemical coagulation and filtration.

A low pH favors H_2S removal. To this purpose CO_2 can be bubbled through the water in advance of aeration. When flue gas is made the source, CO_2 concentrations of about 10% by volume are aimed for; the CO_2 is taken into solution in proportionate amounts; the hydrogen ion concentration is increased; and the concentration of H_2S relative to other dissolved sulfides rises. As a result sulfides are flushed out as H_2S. After that has been accomplished, unwanted CO_2 must be removed, possibly in a second aerator. The response of sulfide water is so variable that aerator design should be based on laboratory experiments, followed by plant-scale tests. It may be desirable to experiment with oxidizing chemicals, such as potassium permanganate ($KMnO_4$).

Hydrogen sulfide is an explosive and extremely toxic gas. Brief exposure (30 min or less) to concentrations as low as 0.1% by volume in air may terminate fatally. Marked symptoms of nausea, headache, and dizziness may follow exposure to 5% of the lethal concentration. Of much interest, too, is the growth of sulfur bacteria on the walls of aerators and in piping systems.

The useful removal and addition of gases other than methane, carbon dioxide, and hydrogen sulfide, including the all-important addition of oxygen, are considered in other chapters of this book.

EXAMPLE 13.8 REMOVAL OF VOLATILE ORGANIC COMPOUNDS AND HYDROGEN SULFIDE GAS FROM GROUNDWATER

The concentrations of hydrogen sulfide gas and a VOC, vinyl chloride, in a groundwater source exceed their respective maximum contaminant levels (MCLs) in the absence of adequate treatment. A community or nontransient noncommunity water system will require implementation of necessary treatment to reduce the concentrations of undesirable substances, such as H_2S and VOC, to the levels below their respective MCLs. Figure 13.10 shows an existing water treatment system consisting of screening, pumping, packed-tower air stripping, pressurized granular media filtration (dual media filtration), chemical oxidation, secondary disinfection (chlorination), corrosion control, clearwell, and water storage and distribution. Assume that the raw well water quality meets the drinking water standards except for H_2S and VOC, and the well water is not GWUDI. Is the existing water system adequate for total water treatment?

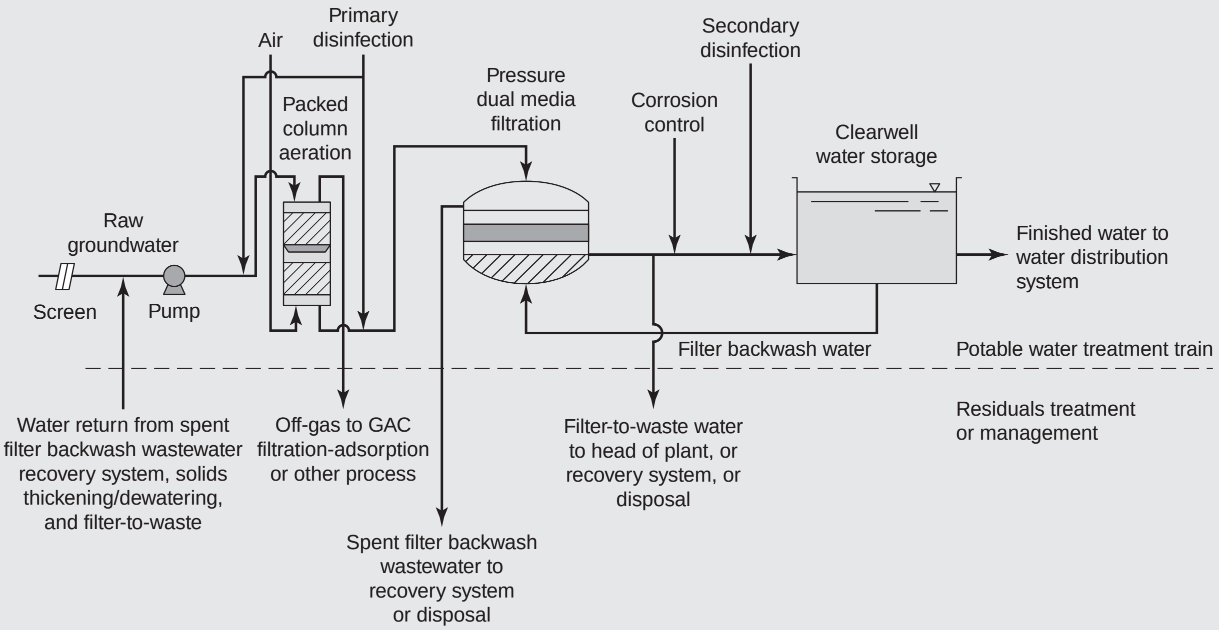


Figure 13.10 Typical process train for removal of dissolved gases and/or volatile impurities, and oxidation of iron and manganese from groundwater (A modified figure).

Solution:

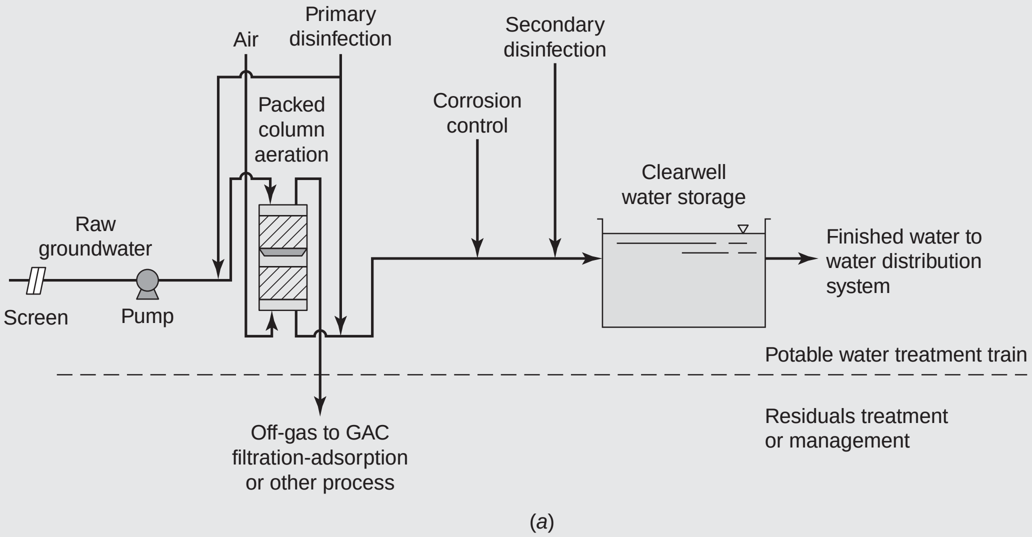
Air stripping and granular activated carbon (GAC) adsorption are the best available technologies (BATs) for removing all VOCs. However, packed-tower air stripping is the only BAT for vinyl chloride. Removal of H_2S is desirable for a public water supply system when its odor would otherwise be present in either raw water or finished water. Treatment processes for removing H_2S include any air stripping process and any chemical oxidation. Accordingly the existing water system partially shown in Fig. 13.10 should be able to adequately treat the raw groundwater aiming at the removal of vinyl chloride and hydrogen sulfide. The oxygen in air is an oxidant. Chlorine is an oxidant as well as a disinfectant. In the oxidation process (including air stripping), the soluble sulfide ion S^{2-} is converted to insoluble sulfur particles which must be subsequently removed by filtration. In general the screen shown in Fig. 11.6 may not be needed for groundwater treatment and therefore, may be eliminated.

EXAMPLE 13.9 REMOVAL OF ONLY VOLATILE ORGANIC COMPOUNDS FROM GROUNDWATER

A groundwater source is slightly contaminated by vinyl chloride (a VOC), which concentration exceeds its MCL in absence of adequate water treatment. Assume that all other water quality parameters (including hydrogen sulfide gas) of the groundwater meet the Federal and the State drinking water standards, and the groundwater is not GWUDI. Recommend a water treatment system for treating the groundwater.

Solution:

A water treatment system shown in Fig. 13.10 minus the pressurized granular media filtration (dual media filtration) will be sufficient. Specifically the recommended water system consists of optional screening, pumping, packed-tower air stripping (aeration), secondary disinfection (chlorination), corrosion control, clearwell, and water storage and distribution. Figures 13.11a and 13.11b show the aeration WTP flow diagrams and details of a packed-tower aeration process module, respectively.



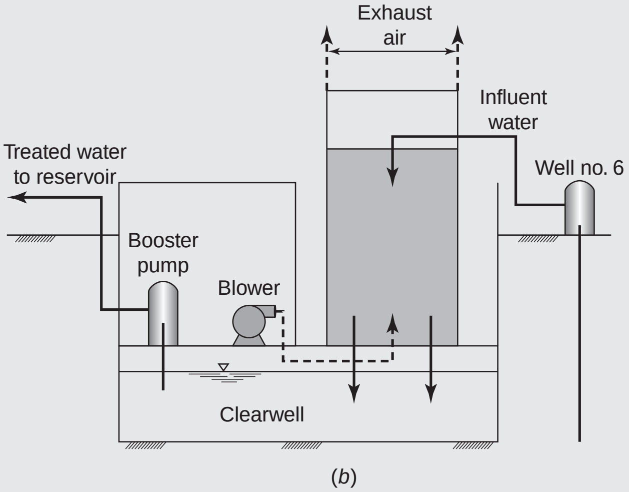


Figure 13.11 (a) Typical flow diagram of a packed column aeration system (A modified figure). (b) Vertical view of a packed column aeration unit (*Source*: U.S. EPA, 1990).

13.14 REMOVAL OF ODORS AND TASTES

The tastes and odors ascribed to algal growths are thought to be caused by essential oils included in their cells. To be desorbed, these oils must first be released to the water. A three-phase system is probably involved, because the volatile principles must be translated in succession from oil to water to air. An oil–air interface is possible but hardly probable in significant magnitude. The boiling points of most essential oils are much higher than those of most gases, and the difficulty of volatilizing most algal odors and tastes is correspondingly greater. Odor removal of but 50% has been reported, for instance, when *Synura* was the odor- and taste-producing organism. On the other hand, appreciable reductions in odors from industrial wastes have been achieved by spray aeration at high pressures.

The taste and odor principles of phenolic substances and many other industrial chemicals are seldom volatile enough to be removed by aeration. The boiling point of phenol itself, C_6H_5OH, for example, is 182°C, whereas substances with boiling points above 0°C are probably not desorbed from water at their common temperatures unless the solubility of the given substance is quite low. Even chlorine, which has a boiling point of −34°C, is so soluble that it does not respond well to aeration. Moreover, it hydrolyzes, ionizes, and reacts with other substances that may be present. Marginal chlorination of phenolic waters may intensify odor and taste troubles without improving the volatility of the chlorophenols formed. Other treatment methods, including breakpoint chlorination, must be resorted to for their control.

A few odors and tastes are reduced in intensity after water has been aerated and allowed to stand for a day or two. Slow oxidation of organic substances may be responsible for observations of this kind. On the other hand, odors due to chlorine reaction products have been known to intensify on standing.

PROBLEMS/QUESTIONS

13.1 What are the aims or objectives of gas transfer?

13.2 What are the four main types of aerators?

13.3 What are the factors that govern gas transfer? How can they be used to optimize a gas transfer operation?

13.4 What are the most common gases that are found in water?

13.5 The gas transfer process is often used to either remove from water or add to water the following same gases:

(i) Oxygen
(ii) Carbon dioxide
(iii) Ammonia

(a) Explain how it is possible to use the process for removal and addition of the same gases.

(b) Discuss the importance of adding each of the above gases in water and wastewater treatment.

(c) Discuss the importance of removing each of the above gases in water and wastewater treatment

13.6 Describe briefly, but in specific terms, the effects of the following factors on gas transfer:

(a) Presence of solutes
(b) Depth of basin
(c) Temperature
(d) Agitation
(e) Partial pressure of the gas

13.7 Determine the saturation concentration of DO in fresh water at 29°C under normal barometric pressure of 760 mm Hg.

13.8 Determine the saturation concentration of DO in saline water at 29°C under normal barometric pressure of 760 mm Hg and containing 5000 mg/L chloride.

13.9 Determine (a) the water vapor pressure at 29°C and (b) the saturation concentration of DO in brackish water containing 5000 mg/L chloride, at 29°C water temperature and under 700 mm Hg pressure.

13.10 Determine the water vapor pressure using the two vapor pressure equations when the water temperatures are 10°C and 0°C.

13.11 Based on Fick's law, $dW/dt = EA\, dc/dx$ (where E is the diffusion or transfer coefficient), derive the following relationship for the gas transfer rate:

$$dc/dt = K_g(c_s - c_t)$$

where

K_g = overall mass transfer coefficient
c_s = saturation concentration of gas in liquid
c_t = concentration of gas in liquid
t = time

13.12 Explain the experimental and mathematical techniques to be followed in the determination of the overall gas transfer coefficient, K_g, in the relationship

$$c_t = c_s - (c_s - c_o)10^{-k_g t}$$

13.13 The following data were collected during an aeration test that was designed for the evaluation of an air diffusion system:

c_t (mg/L)	1.5	2.7	3.9	4.8	6.0	7.0	8.2
dc/dt (mg/L/h)	8.4	7.5	5.3	4.9	4.2	2.8	2.0

(a) Determine the overall oxygen-transfer coefficient, 1/h.

(b) Find the saturation concentration of dissolved oxygen, mg/L.

13.14 What is the equilibrium concentration of oxygen in pure water at 20°C exposed to air under a barometric pressure of 750 mm Hg?

13.15 Find the approximate DO saturation value at 20°C and 760 mm Hg of (1) distilled water and (2) salt water containing 17,500 mg/L of chloride; and also (3) the percentage saturation of these waters when their DO content is 7 mg/L.

13.16 In an experiment on the removal of a gas from water sprayed into the air in spherical droplets 0.25 cm in diameter, the initial supersaturation of the water with the gas was 45 mg/L. After 2 s of exposure this was reduced to 16.5 mg/L. Find the coefficient of gas transfer.

13.17 Find the time of exposure of water falling through a distance of 12 ft (3.66 m) in

(a) Single descent

(b) Four descents

(c) Six descents

13.18 Groundwater under direct influence of surface water (GWUDI) is to be treated as surface water for selection of water treatment system. If the raw GWUDI contains excessive amount of soluble iron (0.9 mg/L) and soluble manganese (0.09 mg/L), moderate turbidity (1.2 NTU), color (16 color units), virus, and coliforms, recommend a water treatment system.

13.19 Recommend a water treatment system for treating a groundwater (not GWUDI), which contains hydrogen sulfide gas concentration higher than its MCL in the absence of adequate treatment. Assume that all other water quality parameters (including VOC) of the raw groundwater meet the government's drinking water standards.

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Chapter 14

Coagulation

14.1 INTRODUCTION

The objective of coagulation and flocculation is to bring together the colloidal particles and other finely divided matter and have them agglomerated to form larger size particles that can be subsequently separated in a more efficient fashion by sedimentation, flotation, or filtration.

The traditional usage of coagulation has been primarily for the removal of turbidity from potable water. However, more recently, coagulation has been shown to be an effective process for the removal of many other contaminants that can be adsorbed by colloids such as metals, toxic organic matter, viruses, and radionuclides. Enhanced coagulation is an effective method to prepare the water for the removal of certain contaminants in order to achieve compliance with US EPA's (United States Environmental Protection Agency) newly proposed standards. These contaminants include arsenic, emerging pathogens such as *Cryptosporidium* and *Giardia*, and humic materials. Humic substances are the precursors of trihalomethanes (THMs) and other disinfection by-products (DBPs) formed by disinfection processes.

Amirtharaja and O'Melia broke down the coagulation process into three distinct and sequential steps:

1. Coagulant formation
2. Particle destabilization
3. Interparticle collisions

The first two steps are usually fast and take place in a rapid-mixing tank. The third step, interparticle collisions, is a slower process that is achieved by fluid flow and slow mixing. This is the process that brings about the agglomeration of particles and it takes place in the flocculation tank (see Chapter 12).

Coagulation is usually achieved through the addition of inorganic coagulants, such as aluminum- or iron-based salts, and/or synthetic organic polymers commonly known as polyelectrolytes. Coagulant aids are available to help in the destabilization and agglomeration of difficult and slow to settle particulate material.

14.2 THE COLLOIDAL STATE

Colloids are very small particles that have extremely large surface area. Colloidal particles are larger than atoms and ions but are small enough that they are usually not visible to the naked eye. They range in size from 0.001 to 10 micrometer resulting in a very small ratio of mass to surface area. One cm^3 of a colloid composed of 10^{18} cubical particles 10 μm on a side has a surface area of 6,500 ft^2 (604 m^2). If the same volume is drawn into a square filament 10 μm on a side in cross-section, the surface area is reduced to 4,300 ft^2 (400 m^2) and if the same volume is flattened into a film 10 μm in thickness, the surface area is still 2,200 ft^2 (204 m^2). The consequence of this smallness in size and mass and largeness in surface area is that in colloidal suspensions (a) gravitational effects are negligible and (b) surface phenomena predominate.

Because of their tremendous surface, colloidal particles have the tendency to adsorb various ions from the surrounding medium that impart to the colloids an electrostatic charge relative to the bulk of surrounding water. The developed electrostatic repulsive forces prevent the colloids from coming together and consequently contribute to their dispersion and stability. Thus the important surface properties of colloids are

1. Their tendency to concentrate substances from the surrounding medium, that is, their adsorptive properties, and
2. Their tendency to develop charge in relation to the surrounding medium, that is, their electrokinetic properties.

14.2.1 Electrokinetic Properties of Colloids

The electrokinetic properties of colloids can be attributed to the following three processes:

1. Ionization of groups within the surface of particles,
2. Adsorption of ions from water surrounding the particles, and
3. Ionic deficit or replacement within the structure of particles.

Water Engineering: Hydraulics, Distribution and Treatment, First Edition. Nazih K. Shammas and Lawrence K. Wang.

Organic substances and bacteria acquire their surface charges as a result of the ionization of the amino and carboxyl groups as shown below:

$$\text{R—NH}_3^+ \rightarrow \text{R—NH}_2 + \text{H}^+ \tag{14.1}$$

$$\text{R—COOH} \rightarrow \text{R—COO}^- + \text{H}^+ \tag{14.2}$$

The resulting charge on the surface of such particles is a function of the pH. At high pH values or low hydrogen ion concentrations, the above reactions shift to the right and the colloid is negatively charged. At a low pH, the reactions shift to the left, the carboxyl group is not ionized, and the particle is positively charged due to the ionized amino group. When the pH is at the isoelectric point, the particle is neutral, that is, neither negatively nor positively charged. Proteinaceous material, containing various combinations of both amino and carboxyl groups, are usually negatively charged at pH values above 4.

Oil droplets adsorb negative ions, preferably hydroxides (OH^-), from solution and consequently they develop a negative charge. Some other neutral particles adsorb selected ions from their surrounding medium such as calcium (Ca^{2+}) or phosphate (PO_4^{3-}) ions rendering them either positively or negatively charged, respectively.

Clays and other colloidal minerals may acquire a charge as a result of a deficit or imperfection in their internal structure. This is known as isomorphic replacement. Clays consist of a lattice formed of cross-linked layers of silica and alumina. In some clays there are less metallic atoms than nonmetallic ones within the mineral lattice producing a negative charge. In others, higher valency cations may be replaced by lower valency cations during the formation of the mineral lattice that renders the clay particles negatively charged. Examples of such imperfection include (a) the substitution of an aluminum ion (Al^{3+}) by either Mg^{2+} or Fe^{2+} and (b) the replacement of Si^{3+} cation by Al^{3+}. According to Amirtharaja and O'Melia, the type and strength of the charge resulting from this imperfection in the clay structure are independent of the surrounding water properties and pH. This is in contrast to the first two processes discussed earlier, in which both pH and ionic makeup of the surrounding solution play a big role in determining the sign and magnitude of the acquired charge on colloidal particles.

14.2.2 Hydration

The extent of hydration depends on the affinity of particles for water. Colloidal particles that have water-soluble groups on their surface such as hydroxyl, carboxyl, sulfuric, and amino, exhibit high affinity for hydration and cause a water film to surround the particles. Such colloids are classified as *hydrophilic* (water loving) particles. On the other hand, colloids that do not show affinity for water and do not have bound water films are classified as *hydrophobic* (water hating). The water molecules that are sorbed on the surface of colloids are either in addition to or in place of other molecules or ions.

14.2.3 Tyndall Effect

A beam of light passing through water and hitting the particles will be reflected because colloidal particles have an index of refraction different from that of water. The turbid appearance due to this interference with the passage of light is called *Tyndall effect*. However, it should be noted that hydrophilic colloids may produce just a diffuse Tyndall cone or none at all. This behavior is attributed to the bound water layer surrounding colloids. These particles will have an index of refraction not very different from that of the surrounding water. Hence, the dispersed phase and the dispersion medium behave in a similar fashion toward the passage of light.

14.2.4 Brownian Movement

Colloids exhibit a continuous random movement caused by bombardment by the water molecules in the dispersion medium. This action, called *Brownian movement*, imparts kinetic energy to the particles that tends to cause an increase in the frequency of collisions, thus promoting coagulation. Elevated temperature increases molecular velocity resulting in more kinetic energy and more intense Brownian movement.

14.2.5 Filterability

Colloids are small enough to pass through ordinary filters, such as paper and sand, but are large relative to ions in size, diffuse very slowly, and will not pass through membranes. As a result, colloidal particles can be readily removed by ultrafiltration but require coagulation prior to their efficient removal by ordinary filtration.

14.3 COLLOIDAL STRUCTURE AND STABILITY OF COLLOIDS

Figure 14.1a is a diagrammatic representation of charge distribution in neighborhood of the surface of particle of ferric oxide solution formed by dispersing $FeCl_3$ in water. Figure 14.1b represents a simple picture of particle surface, showing net charge q and *double-layer* distance δ. Figure 14.1c shows the potential relations corresponding to charge distribution shown in Fig. 14.1a.

The *van der Waals' force* produces the energy of attraction (A in Fig. 14.2) which varies inversely with the square of the distance between two particles. The electrostatic force produces a repulsive energy (R in Fig. 14.2), which decreases more or less exponentially with distance. Curves A and R are combined in curve S as the resultant energy of interaction. A state of equilibrium is reached at the point of minimum

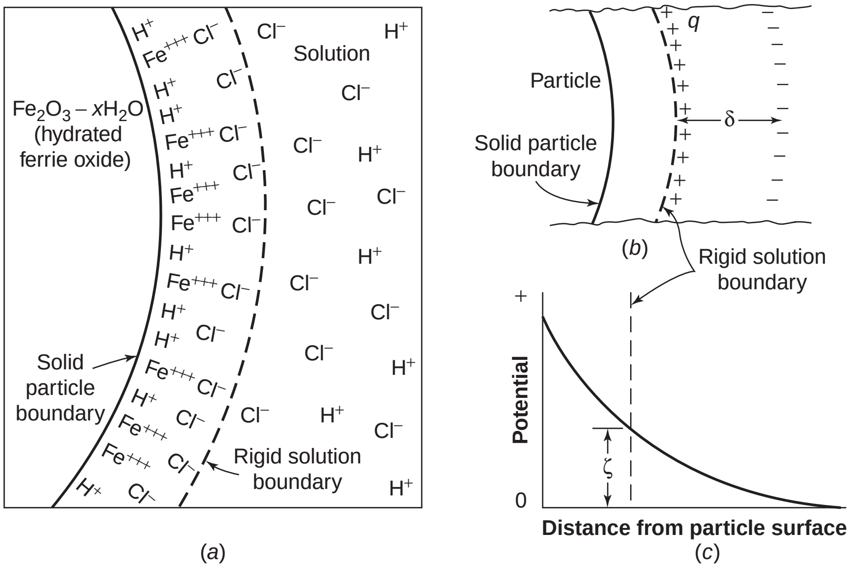


Figure 14.1 Electrical relations at surfaces of colloids (After Fair et al., 1971).

potential energy, P_m, where the distance between the particles is approximately zero and the particles coalesce. To approach each other, the particles must surmount the *energy hill*, E_b. Only if the kinetic energy of the particles is large enough to do this will the colloids become unstable and coagulate.

The stability of colloidal particulate matter is dependent on their electrokinetic property. Colloidal particles acquiring similar primary charges develop repulsive forces that keep them apart and prevent their agglomeration. The primary electrical charges could be either negative or positive. However, the majority of colloids that exist in aqueous systems are negatively charged. A colloidal system as a whole does not have a net charge. Negative primary charges on colloidal particles are balanced by positive counter-ions near the solid–liquid interface and in the adjoining dispersion medium. In a similar fashion, positively charged particles are counterbalanced by negative ions present in surrounding water. This

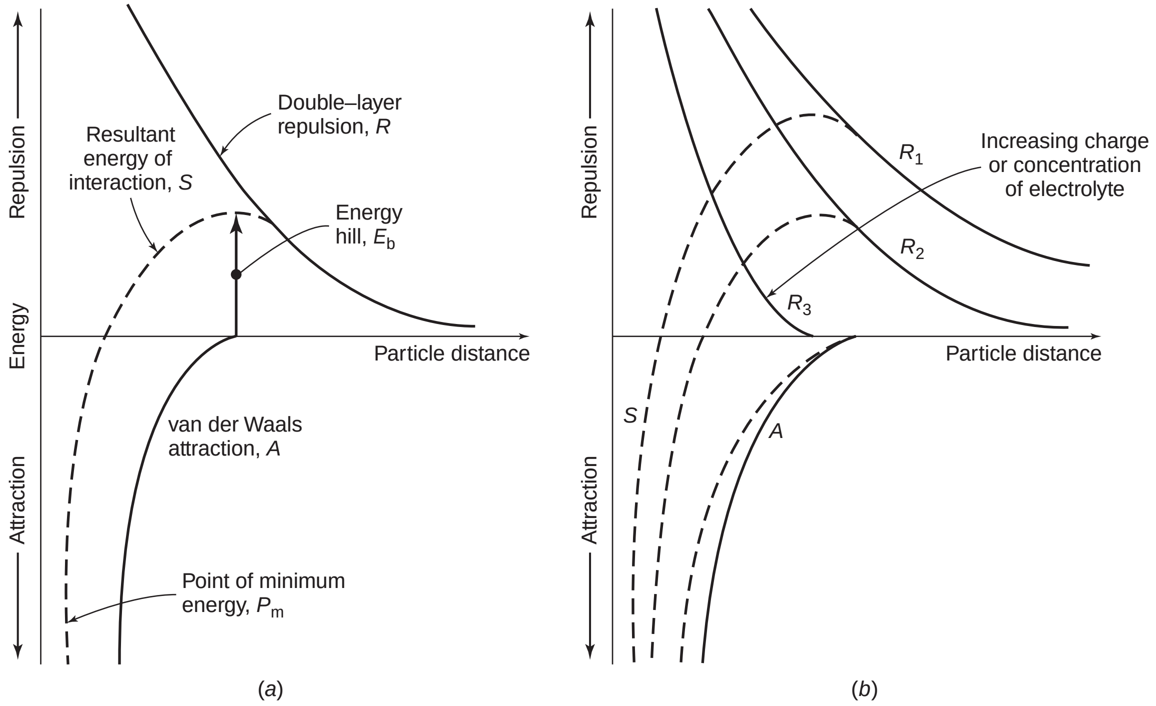


Figure 14.2 Energy of interaction between two colloidal particles in the form of attraction by van der Waals' forces and repulsion by electrostatic forces (After Fair et al., 1971).

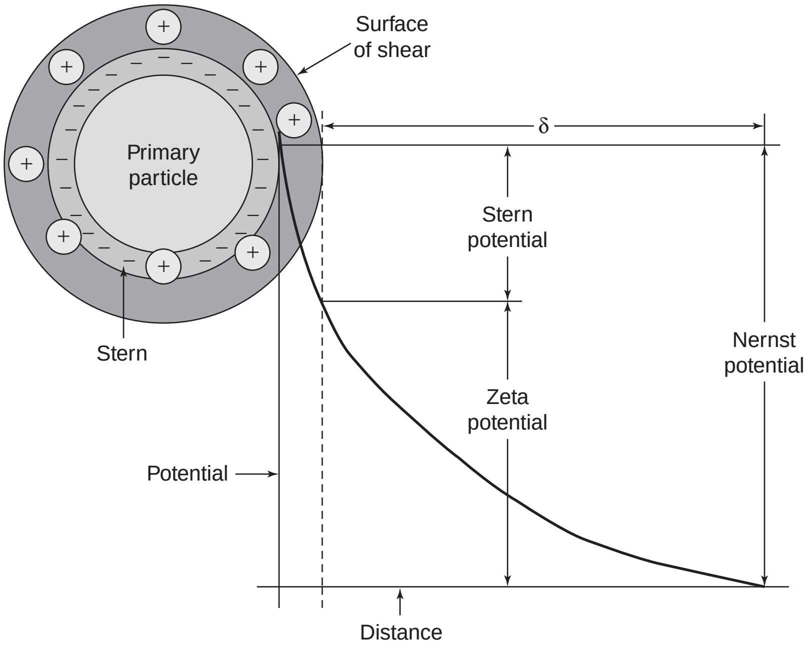


Figure 14.3 The electrical potential of a negatively charged colloidal particle (After Shammas, 2005).

natural inclination toward achieving electrical neutrality and counterbalance of charges results in the formation of an electric double layer around colloidal particles.

The electric double layer, which comprises the charged particle and surrounding counter-ions, is illustrated in Fig. 14.3. The total potential at the surface of the primary charged particle is termed the *Nernst potential*. The dense layer of counter-ions fixed on the surface of the primary particle is called the *Stern layer*. The outer limit of this layer is defined by the *surface of shear* that separates the mobile portion of the colloid from the surrounding mixture of diffuse ions. In an electric field, the ions within the surface of shear will move with particle as a unit. The concentrated counter-ions within the surface of shear reduce the net charge on the particle by an amount that is usually referred to as the *Stern potential*. Consequently, the potential is maximum at the surface of primary particle, Nernst potential, that decreases rapidly through the Stern layer resulting in a net overall charge on the particle at the surface of shear called the *zeta potential*. This potential determines the extent of repulsion between similarly charged particles that is commonly considered to be the major cause leading to the stability of a colloidal system. Further away from the surface of shear both the concentration and potential gradients continue decreasing, but at a more gradual drop, until the potential approaches the point of electrical neutrality in the surrounding solution.

The counter-ions of the Stern layer are concentrated in the interfacial region due to electrostatic attraction. However, these ions tend to be more loosely attached, as they are located at distances further away from particle surface as a result of the potential gradient. Consequently, any thermal agitation may cause these less strongly held ions to diffuse away toward the bulk of the dispersion medium. These two opposite forces, electrostatic attraction and diffusion, give rise to the distribution of the potential over distance such that the highest concentration of counter-ions occurs at the particle interface and drops gradually with increasing distance. When the dispersion medium contains low concentration of ions (low ionic strength) the diffuse layer will be spread over a wide distance, δ, as shown in Fig. 14.1. On the other hand, when the dispersion medium possesses a high ionic strength the diffuse layer would get compressed, become thinner, and eventually extend far less into the bulk of the solution.

When two similar primary charge particles drift toward each other, their diffuse layers start to interact leading to the production of a repulsive electrostatic force. The resulting repulsion between the approaching particles increases in intensity, as the particles get closer. Such charged particles may not be able to collide at all if their charges are high enough. Ultimately, as illustrated in Fig. 14.4, the colloidal stability depends on the relative strength of the above electrostatic forces of repulsion and the forces of attraction.

The forces of attraction are due to *van der Waals' forces*. All colloidal particles, irrespective of their composition, sign or magnitude of charge, or the composition of the dispersion medium, possess such attractive forces. They arise from the following:

1. Electronegativity of some atoms is higher than for others in the same molecule.
2. Vibration of charges within one atom creates a rapidly fluctuating dipole.
3. Approaching particles induce vibrations in phase with each other.

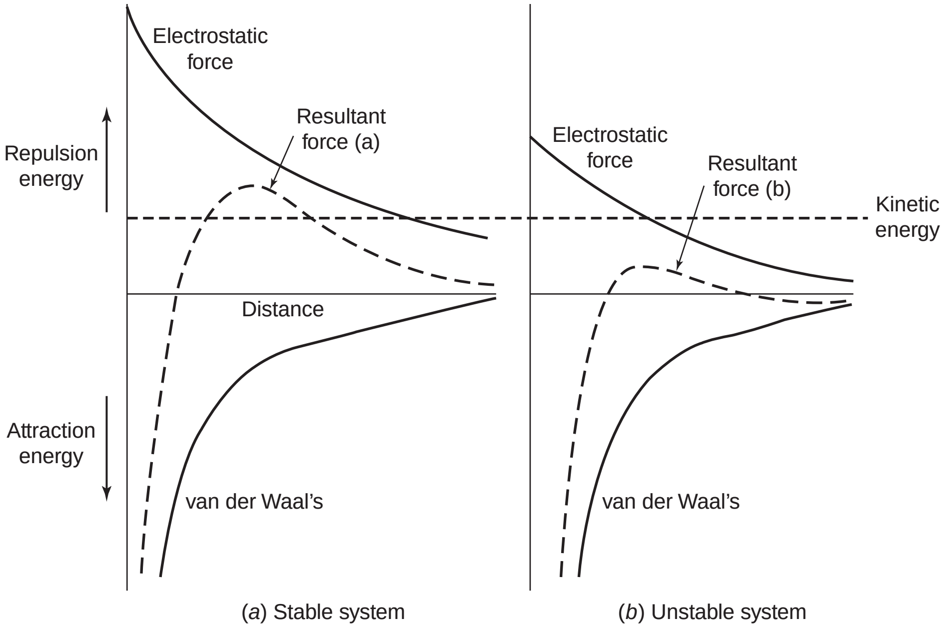


Figure 14.4 Effect of interparticle forces on the stability of a colloidal system (After Shammas, 2005).

The above three result in an attractive force between the two oppositely oriented dipoles. The magnitude of the force varies inversely with distance between particles, increasing rapidly with decreasing distance (see Fig. 14.4). If particles come close enough for these forces to take over, they will adhere.

The other factor, in addition to van der Waals' attractive forces, tending to destabilize a colloidal system is *Brownian movement*. This is due to the random motion of colloids brought about by their bombardment by molecules of the dispersion medium. The outcome of the movement is to impart kinetic energy to the colloidal particles. Higher energy particles moving in a random fashion tend to collide eventually.

Figure 14.4 illustrates the relationship of forces that exist between colloidal particles as a function of the separating distance. The net resultant force is obtained by the summation of the respective electrostatic repulsive force and Van de Waals' attractive force. When the resultant repulsion energy exceeds kinetic energy (Fig. 14.2a), particles will not coagulate and the dispersion is stable. When kinetic energy is larger than the repulsion energy (Fig. 14.2b), the dispersion is unstable and the particles will coagulate. Consequently, if it is required to destabilize and coagulate a stable dispersion, then the electrostatic repulsion energy between the particles must be lowered and/or the kinetic energy of the particles be raised.

14.4 DESTABILIZATION OF COLLOIDS

Destabilization of colloidal particles is accomplished by coagulation through the addition of hydrolyzing electrolytes such as metal salts and/or synthetic organic polymers. Upon being added to the water, the action of the metal salt is complex. It undergoes dissolution, the formation of complex highly charged hydrolyzed metal coagulants (hydroxyoxides of metals), interparticle bridging, and the enmeshment of particles into flocs. Polymers work either on the basis of particle destabilization or bridging between the particles.

The destabilization process is achieved by the following four mechanisms of coagulation:

1. Double-layer compression
2. Adsorption and charge neutralization
3. Entrapment of particles in precipitate
4. Adsorption and bridging between particles

14.4.1 Double-Layer Compression

When high concentrations of simple electrolytes are introduced into a stabilized colloidal dispersion, the added counter-ions penetrate into the diffuse double layer surrounding the particles rendering it denser and hence thinner and smaller in volume. The addition of counter-ions with higher charges, such as divalent and trivalent ions, will result in even steeper electrostatic potential gradients and more rapid decrease in charge with distance from surface of particles. The net repulsive energy (see Fig. 14.4) would become smaller or even would be completely eliminated, allowing the particles to approach each other and agglomerate.

A mathematical model that describes this coagulation mechanism is explained in detail by Verwey and Overbeek. The prediction of this model is in agreement with what is known as the *Schultze–Hardy rule*. This rule states that the coagulation of colloidal particles is achieved by ions of added electrolytes, which carry opposite charge to that of the colloids, and that the destabilization capability of the ions rises

Table 14.1 Relative coagulation power of electrolytes

	Relative power of coagulation	
Electrolyte	Positive colloid	Negative colloid
NaCl	1	1
Na_2SO_4	30	1
Na_3PO_4	1,000	1
$BaCl_2$	1	30
$MgSO_4$	30	30
$AlCl_3$	1	1,000
$Al_2(SO_4)_3$	30	>1,000
$FeCl_3$	1	1,000
$Fe_2(SO_4)_3$	30	>1,000

Source: Shammas (2005).

sharply with ion charge. Table 14.1 illustrates the relative effectiveness of various electrolytes in the coagulation of negatively and positively charged colloids. For example, the relative power of Al^{3+}, Mg^{2+}, and Na^+ for the coagulation of negative colloids is shown to vary in the ratio of 1000:30:1. A similar ratio is observed for the relative capability of PO_4^{3-}, SO_4^{2-}, and Cl^- for the coagulation of positively charged colloids.

14.4.2 Adsorption and Charge Neutralization

For all practical purposes, the ability of a chemical substance to destabilize and coagulate colloidal particles is the result of a combination of several mechanisms. Long-chained organic amines are often mentioned as being typical coagulants that function by adsorption and electrostatic neutralization. The positively charged organic amine molecules ($R–NH_3{}^+$) are easily and quickly attached to negatively charged colloidal particles. The charge on the particles get neutralized and the electrostatic repulsion is decreased or eliminated resulting in the destabilization of the colloids and hence their agglomeration. The organic amines are hydrophobic because there is a lack of interaction between the CH_2 groups in their R-chain and the surrounding water. As a result, these positively charged ions are driven out of the water and get adsorbed on the particulate interface. An overdose of $R–NH_3{}^+$ counterions, however, can lead to charge reversal from negative to positive and the restabilization of the dispersion system.

When coagulants such as metal salts are added to water they dissociate yielding metallic ions, which undergo hydrolysis and form positively charged metallic hydroxyoxide complexes. The commonly used coagulants, trivalent salts of aluminum and iron, produce numerous species because the hydrolysis products themselves tend to polymerize to give polynuclear metallic hydroxides. Examples of aluminum salt polymers are $Al_6(OH)_{15}^{3+}$ and $Al_7(OH)_{17}^{4+}$ and of iron salt polymers are $Fe_2(OH)_2^{4+}$ and $Fe_3(OH)_4^{5+}$. When such polyvalent complexes possessing high positive charges get adsorbed to the surface of the negatively charged colloids, the result is again a neutralization of the charges, decrease in the repulsion energy, and destabilization of the colloids. In a similar fashion to what occurs with the organic amines, an overdose of metallic salts could reverse the colloidal charge and restabilize the particles.

14.4.3 Entrapment of Particles in Precipitate

When the coagulants alum, $Al_2(SO_4)_3$, or ferric chloride, $FeCl_3$, are added in high enough concentration, they will react with hydroxides (OH^-) to form metal hydroxide precipitates $Al(OH)_3$ or $Fe(OH)_3$, respectively. The colloidal particles get entrapped in the precipitates either during the precipitate formation or just after. This type of coagulation by enmeshment of colloids in precipitates is commonly called sweep coagulation.

There are three elements that influence this coagulation mechanism:

1. *Oversaturation*: The rate of precipitation is a function of oversaturation with the metal hydroxide. To obtain fast precipitation and efficient sweep coagulation, high concentration of $AL(OH)_3$ or $Fe(OH)_3$ is required.
2. *Presence of anions*: The rate of precipitation is improved by the presence of various anions in water. The most effective in this respect are the sulfate ions.
3. *Concentration of colloids*: The rate of precipitation is also improved with higher concentration of colloidal particles. The reason for this is that the colloids themselves could act as nuclei for the formation of precipitates. In this case, it can be concluded that lower rather than higher coagulant dosage will be required to coagulate water having higher colloidal particle concentration.

14.4.4 Adsorption and Bridging between Particles

Polymers destabilize colloidal particles by interconnecting particles together through the formation of bridges that extend between particles. The polymers have reactive groups that bind to specific sites on the surface of colloidal particles. When a group on a polymer molecule attach to a colloid, the remainder of the long-chain molecule extends away into the water. Once the extended portion of the polymer gets attached to another colloidal particle, the two particles become tied together or bridged by the polymer. If no other particle is available or if there is an overdose of polymer, the free extended portions of the polymer molecule would wrap around the same original particle, which could effectively bring about the restabilization of the colloid. Restabilization can also occur due to aggressive mixing or extended agitation, which may break the interparticle bridging and allow the folding back of the freed polymer portions around the same original particle.

14.5 INFLUENCING FACTORS

Many factors play a role in affecting the coagulation process. In addition to mixing that was explained in great detail in Chapter 12, the following discussion covers the most important factors.

14.5.1 Colloid Concentration

Colloidal concentration has a great impact on both the required dosage and the efficiency of the coagulation process itself. The dosage of coagulants required for the destabilization of a colloidal dispersion is stoichiometrically related to the amount of colloidal particles present in solution. However, for dilute colloidal systems, the rate of coagulation is very slow because of the small number of colloids in suspension and, therefore, not enough contact between particles is available. Under such conditions increasing the concentration of particulate matter by the addition of a coagulant aid or recycling of settled sludge would improve the coagulation rate. Application of a large coagulant dosage to a dilute colloidal suspension would result in a greater chance of restabilizing the colloids.

14.5.2 Alkalinity and pH

The optimum pH range makes metallic coagulants insoluble and improves the strength of the floc, and stronger floc enhances turbidity removal through sedimentation and filtration. The water must contain sufficient alkalinity for aluminum or iron floc formation. When pH is too low, adding soda ash, lime, sodium hydroxide, or sodium bicarbonate will increase the water's pH and alkalinity. Without sufficient alkalinity, the coagulation process will not proceed to completion. Very low pH levels will also lead to high aluminum ion residual in the finished water, which is undesirable.

When the pH level is too high due to factors such as algae and aquatic plant activity, adding acid is necessary. Typical acids used for this application are sulfuric acid and hydrochloric acid.

14.5.3 Coagulant Dosage and pH

pH is a measure of H^+ and OH^- ion concentration. The presence of these ions in potential determining layer may cause particle charge to be more positive or less negative at pH values below the isoelectric point. At high pH values above the isoelectric point the reverse effect takes place whereby particle charge become more negative or less positive. The *isoelectric point* is the pH value at which charge is most nearly neutralized. The isoelectric point for aluminum hydroxide is around pH 8. It varies with the ionic strength in solution but is normally in the pH range of 7–9.

Table 14.2 Optimum pH values for metallic coagulants

Coagulant	pH
Aluminum sulfate	4.0–7.0
Ferrous sulfate	8.5 and above
Ferric chloride	3.5–6.5 and above 8.5
Ferric sulfate	3.5–7.0 and above 9.0

Source: Shammas (2005).

The solubility of colloidal dispersions is affected radically by pH. $Al(OH)_3$ is amphoteric in nature and is soluble at low and high pH. The greatest adsorption occurs in the pH range where there is minimum solubility. Examples of optimum pH ranges for metallic salts are shown in Table 14.2. Researchers reported that optimal coagulation with alum takes place at pH values near to 5 and 7. At these points, the positively charged aluminum hydroxide neutralizes the negatively charged turbidity producing colloidal particles, resulting in zero zeta potential. However, in the pH range from 5 to 7 the colloidal particles are restabilized due to charge reversal brought about by excess adsorption of the positively charged aluminum hydroxide species. pH also plays a part in affecting the amount of aluminum residual in the treated water.

The influence of pH on polymers behavior and effectiveness in coagulation is particularly important because of the interaction between pH and the charge on the electrolyte. The extent of charge change with pH is a function of the type of active group on the polymer (carboxyl, amino, etc.) and the chemistry of those groups.

The *dissolved* polynuclear hydroxo complexes that are intermediates in the hydrolytic transition of iron and aluminum ions to hydrous oxides are efficient coagulants. Under favorable conditions of pH, temperature, and time of aging, the metal ion hydrolysis products are more strongly sorbed on colloidal dispersions than are nonhydrolyzed metal ions. Figure 14.5a shows as a function of pH the critical coagulation concentration for a dispersion of amorphous SiO_2, that is, the minimal concentration of coagulant, Fe(III) in this instance, necessary to produce a floc. The following conclusions may be drawn: (1) at pH values below 3, where Fe(III) is not hydrolyzed, coagulation of SiO_2 is brought about by Fe^{3+} ions; (2) at higher pH values, lower dosages of Fe(III) cause the coagulation of the SiO_2 sol, highly charged polynuclear hydroxo complexes being formed and strongly sorbed on the surfaces of the colloids where they are able to reverse the charge of the colloids when they are present at a somewhat higher concentration than the critical coagulation concentration; (3) it follows from the charge-reversal concentration that the hydrolyzed species, and not the free Fe^{3+}, causes charge reversal; (4) at large concentrations of Fe(III), ferric oxide is precipitated within the time period of observation; and

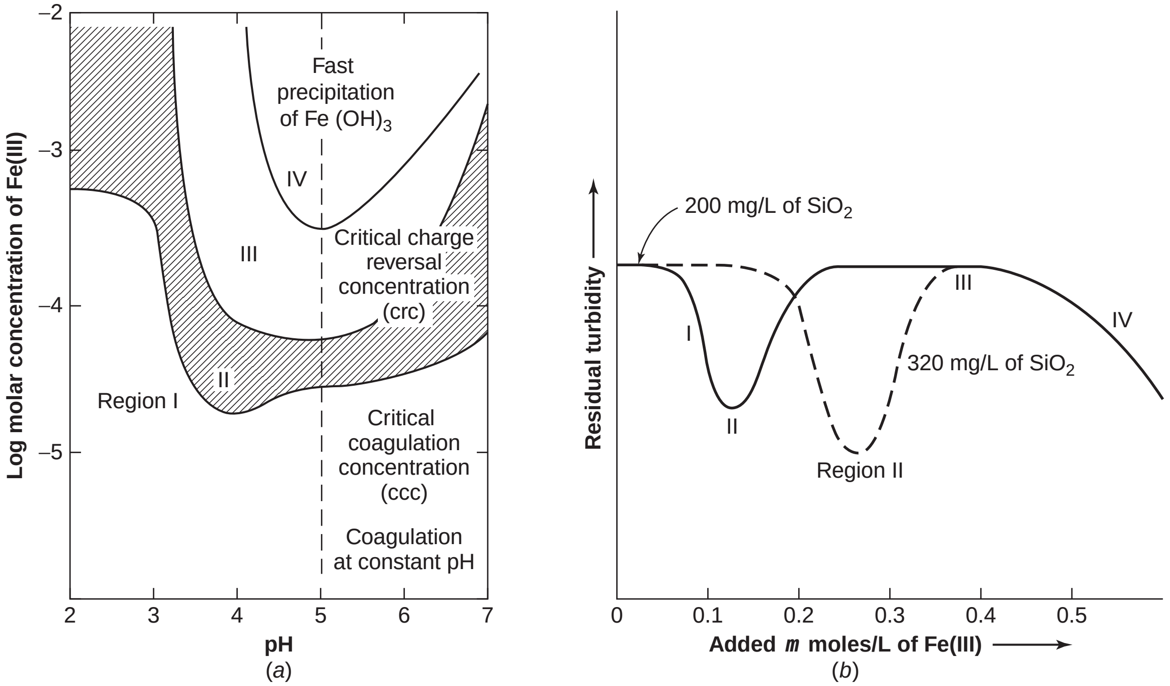


Figure 14.5 Coagulation of amorphous silica of diameter 15 μ with Fe(III). (a) Concentration–pH domain for sol stability (region I); coagulation and charge reversal (region II); stability of charge reversed sol (region III) and of $Fe(OH)_3$ precipitation (region IV). (b) Effect of dosage on residual turbidity at a constant pH of 5 (After Fair et al., 1971).

(5) the stability and instability domains of an SiO_2 suspension in the presence of ferric iron are identified in Fig. 14.3 as region I, in which SiO_2 is stable as a negatively charged sol; region II, shaded, in which SiO_2 is coagulated by Fe(III); region III, in which SiO_2 is restabilized as a positive sol by the adsorbed Fe(III) hydrolysis species; and region IV, in which hydrous iron oxide precipitates and entraps some of the suspended SiO_2.

Coagulation data of this kind can be obtained by running coagulation tests and plotting the information obtained. Suitable variables are residual turbidity and color in relation to coagulant dose. A coagulation curve at a constant and controlled pH of 5 is shown in Fig. 14.5b for the system described in Fig. 14.5a. Initial increments of Fe(III) added to the dispersion of negatively charged SiO_2 are seen to have little effect on the stability of SiO_2 in phase I, which corresponds to region I of Fig. 14.5a. In phase II flocculation occurs as soon as the critical coagulation concentration is exceeded. In phase III, however, stability of the sol is restored upon further addition of coagulant and as soon as the critical restabilization concentration is reached. Finally, in phase IV, an excess of coagulant either precipitates hydrous ferric oxide or flocculates the positive sol because a critical concentration of anions has been established in the system. The charge-reversal concentration and, to a smaller extent, the critical coagulation concentration depend on the concentration or surface area of the colloids to be coagulated. The coagulating dose may be proportional to the concentration of the colloidal impurities. In practice, however, coagulation is not always induced by dosages in the charge-reversal concentration range (phase II). The rate of coagulation may be too slow, especially when dispersions are dilute. Instead of this, coagulation becomes successful only when concentrations of metal ions are high enough to precipitate metal hydroxides that sweep the colloidal dispersions together (phase IV).

14.5.4 Zeta Potential

The zeta potential represents the net charge of colloidal particles. Consequently, the higher the value of the zeta potential, the greater is the magnitude of the repulsive power between the particles and hence the more stable is the colloidal system. The magnitude of zeta potential is determined from electrophoretic measurement of particle mobility in an electric field.

14.5.5 Affinity of Colloids for Water

Hydrophilic (water loving) colloids are very stable. Because of their hydration shell, chemicals cannot readily replace sorbed water molecules and consequently they are difficult to coagulate and remove from suspension. The stability of hydrophilic dispersions depends more on their love for water than on their electrostatic charge. It has been estimated that suspensions containing such particles require 10–20 times more coagulant amounts than what is normally needed to destabilize hydrophobic particles. Typical examples are the color-producing material in surface water and organic colloids present in wastewater. On the other hand, examples of hydrophobic (water hating) particles are metal oxides that can be easily coagulated and destabilized. However, the bulk of colloidal particles in turbid water, usually exhibit a mixture of hydrophobic–hydrophilic properties, resulting in suspensions that are intermediate in the degree of their difficulty to coagulate.

14.5.6 Anions in Solution

As explained below, one of the constraints in using alum and iron as coagulants is the occurrence of charge reversal and restabilization of colloids. However, this behavior can be suppressed or eliminated in the presence of high concentrations of anions such as sulfate, silicate, and phosphate. It was found that background concentration of SO_4^{2-} in excess of 10–14 mg/L has the ability to prevent restabilization. Coagulation with alum is brought about by various species of positively charged aluminum hydroxyoxides. Aluminum hydroxide possesses its lowest charge and lowest solubility at its isoelectric point that lies in the pH range of 7–9. As a result, when the alum dosage is increased within this pH range, sweep coagulation takes place due to the formation of the aluminum hydroxide precipitate. However, at lower pH values (5–7), higher dosages of alum will tend to increase the positively charged alum species that get adsorbed on particles interface leading to charge reversal and the restabilization of the colloidal particles. Similar concepts and conclusions are applicable to iron coagulants.

14.5.7 Cations in Solution

The presence in raw water of divalent cations, such as Ca^{2+} and Mg^{2+}, is commonly considered not only to be helpful in the coagulation of negatively charged colloidal clay particles by anionic polymers but also to be necessary. Three reasons have been suggested to be behind this beneficial effect:

1. Compression of the colloidal double layer
2. Reduction of the colloidal negative charge and minimization of repulsive potential
3. Reduction in the range of repulsive barrier between adsorbed polymers

14.5.8 Temperature

Coagulation by metallic salts is adversely affected by low temperature. However, the effect has been reported to be more pronounced in using alum, hence the recommendation to switch to iron salts when operating under low water temperatures. Another alternative option is to add bentonite as a coagulant aid. The addition of the negatively charged clay particles will enable the coagulation process to proceed as a result of charge neutralization rather than by sweep coagulation. The increase in rate and effectiveness of coagulation at higher temperatures can be attributed to the following reasons:

1. Increase in velocity of molecules and hence in kinetic energy
2. Increase in rate of chemical reactions
3. Decrease in time of floc formation
4. Decrease in viscosity of water
5. Alteration in the structure of the flocs resulting in larger agglomeration

14.6 COAGULANTS

Coagulants, that is, chemicals that are added to the water to achieve coagulation should have the following three properties:

1. Trivalent metallic cations or polymers whose effectiveness as coagulants has been determined.
2. Nontoxic and without adverse physiological effects on human health.
3. Insoluble or low solubility in the pH ranges common in water treatment practice. This is necessary in order to have an efficient coagulation process and to be able to leave the lowest possible residual of the chemical in the treated water.

The most commonly used coagulants in water and wastewater treatment include aluminum sulfate (alum), ferric chloride, ferric sulfate, ferrous sulfate (copperas), sodium aluminate, polyaluminum chloride (PACl), and organic polymers.

The selection of a particular coagulant depends on the required level of effectiveness. A standard jar test is the recommended method to determine the relative effectiveness of various coagulants for a particular raw water supply. If different coagulants are equally effective, then the factors that should be considered are cost; availability; overall safety; and ease of storage, handling, and application. Metallic salts, for example, are much less expensive than polymers.

Alum is the most widely used coagulant because of its availability, low cost, ease of use, and ease of storage. Ferric chloride, other metallic salts, and polymers are less widely used. Alum's performance, however, is greatly affected by the pH of the influent. The recommended dosage of alum ranges from 5 to 150 mg/L, but the problem of sludge disposal increases with higher alum dosages. Due to health concerns about aluminum, some water utilities are considering switching to ferric chloride. Although ferric chloride is not always as effective as alum in reducing trihalomethane formation potential (THMFP) and total organic carbon (TOC), it is more effective than alum for water with high dissolved color, low turbidity, and a moderate pH.

In applications where polymers are effective, dosages are generally lower than alum dosage for the same effect. Typical polymer dosages range from 1.5 to 10 mg/L. Consequently, polymer coagulants produce less residual sludge than alum. To avoid large sludge volumes, there is a trend toward replacing alum with cationic polymers, cationic polymers plus alum, or sodium aluminate. The use of cationic polymers instead of alum is most applicable to raw water with turbidity of less than 5 NTU and where direct filtration is feasible.

14.6.1 Aluminum Salts

The chemistry of metallic salts is a complex one. It involves dissolution, hydrolysis, and polymerization reactions.

1. *Dissolution.* All metal cations in water are present in a hydrated form as aquocomplexes. The simple aluminum variety Al^{3+} does not exist as such in an aqueous solution. Rather, the aluminum species is present in the aquometal form as $Al(H_2\,O)_6{}^{3+}$:

$$Al_2(SO_4)_3 + 12H_2O \rightarrow 2Al(H_2O)_6^{3+} + 3SO_4^{2-} \quad \textbf{(14.3)}$$

2. *Hydrolysis.* The aquometal ions formed in the dissolution of alum in water are acidic or proton donors. This is demonstrated by the following hydrolytic reactions:

$$Al(H_2O)_6^{3+} + H_2O \rightarrow Al(H_2O)_5(OH)^{2+} + H_3O^+ \quad \textbf{(14.4)}$$

$$Al(H_2O)_5(OH)^{2+} + H_2O \rightarrow Al(H_2O)_4(OH)_2^+ + H_3O^+ \quad \textbf{(14.5)}$$

$$Al(H_2O)_4(OH)_2^+ + H_2O \rightarrow Al(H_2O)_3(OH)_3 + H_3O^+ \quad \textbf{(14.6)}$$

$$Al(H_2O)_3(OH)_3 + H_2O \rightarrow Al(H_2O)_2(OH)_4^- + H_3O^+ \quad \textbf{(14.7)}$$

3. *Polymerization.* The hydroxocomplexes formed as products of hydrolysis may combine to form a variety of hydroxometal polymers such as

$Al_6(OH)_{15}^{3+}$

$Al_7(OH)_{17}^{4+}$

$Al_8(OH)_{20}^{4+}$

$Al_{13}(OH)_{34}^{5+}$

The net result of adding alum to an aqueous environment is the formation of large positively charged complexes that are insoluble and the generation of hydrogen ions. The actual exact variety of species present in water, following the addition of the coagulant, is determined by both pH and the extent of the applied dosage. As the dosage is increased such that it exceeds the solubility of alum in water, hydrolysis takes place. At further increase in dosage, a variety of hydroxocomplexes are formed, followed by the production of hydroxometal polymers, and finally the formation of the aluminum hydroxide precipitates.

Since the dissolution of alum in water increases the concentration of hydrogen ions, the net effect is a drop in pH or the consumption of present alkalinity:

$$Al_2(SO_4)_3{\cdot}14.3H_2O + 3Ca(HCO_3)_2 \rightarrow \underline{2Al(OH)_3} + 3CaSO_4 + 14.3H_2O + 6CO_2 \quad \textbf{(14.8)}$$

As shown in the above reaction, each additional mole of alum dosage consumes 6 moles of alkalinity (as $HCO_3{}^-$) and produces 6 moles of carbon dioxide. This means that each mg/L of alum will decrease water alkalinity by 0.50 mg/L (as $CaCO_3$) and will produce 0.44 mg/L of carbon dioxide. As long as adequate natural alkalinity (buffering capacity) is present in water, the tendency of alum to lower the pH does not create an operational problem.

When natural alkalinity is not sufficient to react with the alum dosage, lime or soda ash can be added to cover the deficit:

$$Al_2(SO_4)_3{\cdot}14.3H_2O + 3Ca(OH)_2 \rightarrow \underline{2Al(OH)_3} + 3CaSO_4 + 14.3H_2O \quad \textbf{(14.9)}$$

$$Al_2(SO_4)_3{\cdot}14.3H_2O + 3Na_2CO_3 + 3H_2O \rightarrow \underline{2Al(OH)_3} + 3Na_2SO_4 + 14.3H_2O + 3CO_2 \quad \textbf{(14.10)}$$

Lime is the most commonly used chemical because of its lower cost. However, soda ash has an advantage over lime in that it does not increase water hardness. The optimum pH for coagulation with alum is around 6 with an effective operational range between pH 5 and 8.

EXAMPLE 14.1 DETERMINATION OF DAILY AMOUNT OF LIME REQUIRED FOR COAGULATION WITH ALUM

A plant treating 50 MGD (189.25 MLD) of water is using 50 mg/L of alum [$Al_2(SO_4)_3{\cdot}14.3H_2O$] as a coagulant. The water contains a natural alkalinity of 30 mg/L as $3Ca(HCO_3)_2$. Compute the number of pounds of lime as CaO required per day.

Solution 1 (US Customary Units):

From Eq. (14.8):

$$Al_2(SO_4)_3 \cdot 14.3H_2O + 3Ca(HCO_3)_2 \rightarrow \underline{2Al(OH)_3} + 3CaSO_4 + 14.3H_2O + 6CO_2$$

Molecular weight of $Al_2\ (SO_4)_3 \cdot 14.3H_2O = 2 \times 27 + 3 \times 32 + 12 \times 16 + 14.3 \times 18 = 600$.
Molecular weight of $3Ca\ (HCO_3)_2 = 3\ [40 + 2 \times 1 + 2 \times 12 + 6 \times 16] = 3 \times 162 = 486$.
600 mg/L alum reacts with 486 mg/L calcium bicarbonate.
? mg/L alum will react with the 30 mg/L available calcium bicarbonate.
mg/L alum = 30 × 600/486 = 37 mg/L.
Left over alum in solution = 50 − 37 = 13 mg/L.

From Eq. (14.9):

$$Al_2(SO_4)_3 \cdot 14.3H_2O + 3Ca(OH)_2 \rightarrow \underline{2Al(OH)_3} + 3CaSO_4 + 14.3H_2O$$

Molecular weight of $3Ca(OH)_2 = 3\ [40 + 2 \times 16 + 2 \times 1] = 3 \times 74 = 222$.
600 mg/L alum reacts with 222 mg/L calcium hydroxide.
13 mg/L remaining alum will need ? mg/L calcium hydroxide.
mg/L needed calcium hydroxide = 13 × 222/600 = 4.8 mg/L.

$CaO + H_2O \rightarrow Ca(OH)_2$
56 74
? 4.8
mg/L as CaO = 4.8 × 56/74 = 3.62 mg/L.

Since 1 mg/L = 8.34 lb/MG.

Weight of CaO needed per day of operation = 50 MGD × 3.62 × 8.34 lb/MG
= **1,509 lb/d**.

Solution 2 (SI Units):

The solution is identical to the US customary units for the calculation of CaO requirement = 3.62 mg/L.

Weight of CaO needed per day of operation = $(189.25 \times 10^6\ \text{L/d} \times 3.62\ \text{mg/L})/10^6\ \text{mg/kg}$
= **685 kg/d**.

14.6.2 Iron Salts

Ferric salts (ferric chloride and ferric sulfate) when added to water behave in a similar fashion to alum. As illustrated in the following reactions for ferric sulfate, the dissolution, hydrolysis, and polymerization reactions are identical to that of alum:

$$Fe_2(SO_4)_3 + 12H_2O \rightarrow 2Fe(H_2O)_6^{3+} + 3SO_4^{2-} \quad \textit{Dissolution} \qquad \textbf{(14.11)}$$

$$Fe(H_2O)_6^{3+} + H_2O \rightarrow Fe(H_2O)_5(OH)^{2+} + H_3O^+ \quad \textit{Hydrolysis} \qquad \textbf{(14.12)}$$

$$Fe_2(OH)_2^{4+} \quad \text{(example of a ferric dimer)} \quad \textit{Polymerization}$$

Iron salts are also acidic in solution and consume alkalinity:

$$Fe_2(SO_4)_3 + 3Ca(HCO_3)_2 \rightarrow \underline{2Fe(OH)_3} + 3CaSO_4 + 6CO_2 \qquad \textbf{(14.13)}$$

If natural alkalinity is not sufficient, lime or soda ash can be added:

$$Fe_2(SO_4)_3 + 3Ca(OH)_2 \rightarrow \underline{2Fe(OH)_3} + 3CaSO_4 \qquad \textbf{(14.14)}$$

$$Fe_2(SO_4)_3 + 3Na_2CO_3 + 3H_2O \rightarrow \underline{2Fe(OH)_3} + 3Na_2SO_4 + 3CO_2 \qquad \textbf{(14.15)}$$

Ferric coagulants may have some advantages when coagulating certain types of water. First, coagulation is effective over a wider pH range, usually from pH 4 to 9. However, best performance is between pH 3.5 and 6.5 and above 8.5. Second, a strong and heavy floc is produced which can settle rapidly. Third, ferric salts are more effective for removing color-, taste-, and odor-producing matter.

EXAMPLE 14.2 DETERMINATION OF DAILY AMOUNT OF LIME REQUIRED FOR COAGULATION WITH FERRIC SULFATE

A water treatment plant has a flow of 6 MGD (22.7 MLD). The plant uses 25 mg/L of ferric sulfate [$Fe_2(SO_4)_3$] as a coagulant. If the water contains a natural alkalinity of 18 mg/L as $Ca(HCO_3)_2$, compute the weight of lime as CaO required per day.

Solution 1 (US Customary Units):

Iron salts are acidic in solution and consume alkalinity:

$$Fe_2(SO_4)_3 + 3Ca(HCO_3)_2 \rightarrow \underline{2Fe(OH)_3} + 3CaSO_4 + 6CO_2 \quad \textbf{(14.13)}$$

Molecular weight of $Fe_2(SO_4)_3 = 2 \times 56 + 3 \times 32 + 12 \times 16 = 400$.
Molecular weight of $3Ca(HCO_3)_2 = 3\,[40 + 2 \times 1 + 2 \times 12 + 6 \times 16] = 3 \times 162 = 486$.
400 mg/L ferric sulfate reacts with 486 mg/L calcium bicarbonate.
? mg/L ferric sulfate will react with the 18 mg/L available calcium bicarbonate.
mg/L ferric sulfate = 18 × 400/486 = 14.8 mg/L.
Left over ferric sulfate in solution = 25 − 14.8 = 10.2 mg/L.
Since natural alkalinity is not sufficient, lime as $Ca(OH)_2$ is added:

$$Fe_2(SO_4)_3 + 3Ca(OH)_2 \rightarrow \underline{2Fe(OH)_3} + 3CaSO_4 \quad \textbf{(14.14)}$$

Molecular weight of $3Ca(OH)_2 = 3\,[40 + 2 \times 16 + 2 \times 1] = 3 \times 74 = 222$.
400 mg/L ferric sulfate reacts with 222 mg/L calcium hydroxide.
10.2 mg/L remaining ferric sulfate will need ? mg/L calcium hydroxide.
mg/L needed calcium hydroxide = 10.2 × 222/400 = 5.7 mg/L.

$CaO + H_2O \rightarrow Ca\,(OH)_2$
56 74
? 5.7
mg/L as CaO = 5.7 × 56/74 = 4.28 mg/L.

$$\textbf{Weight of CaO needed per day of operation} = 6\text{ MGD} \times (4.28 \times 8.34\text{ lb/MG})$$
$$= \textbf{214 lb/d}.$$

Solution 2 (SI Units):

The solution is identical to the US customary units for the calculation of CaO requirement = 4.28 mg/L.

$$\textbf{Weight of CaO needed per day of operation} = (22.7 \times 10^6\text{ L/d} \times 4.28\text{ mg/L})10^6\text{ mg/kg}$$
$$= \textbf{97 kg/d}.$$

The specific gravity of chemical sludge, such as alum sludge and iron sludge, can be determined by the following equation:

$$\frac{1}{S_s} = \sum_{i=1}^{n} \frac{w_i}{S_i} \quad \textbf{(14.16)}$$

where

S_s = specific gravity of sludge
W_i = weight fraction of *i*th component of sludge
S_i = specific gravity of *i*th component of sludge

EXAMPLE 14.3 DETERMINATION OF DAILY AMOUNT OF REQUIRED ALUM

A raw water supply of 2 MGD (7.57 MLD) is treated with 26 mg/L of alum. Determine the daily alum requirement in lb and kg assuming the alum dosage has been determined by a standard jar test.

Solution 1 (US Customary System):

26 mg/L = 26 × 8.34 lb/MG.
2 MGD (26 × 8.34 lb/MG) = **433.68 lb/d.**

Solution 2 (SI System):

7.57 MLD = 7.57×10^6 L/d.

26 mg/L = 26×10^{-6} kg/L.

$(7.57 \times 10^6$ L/d$)(26 \times 10^{-6}$ kg/L$)$ = **196.82 kg/d.**

EXAMPLE 14.4 DETERMINATION OF NATURAL ALKALINITY REQUIREMENT

Raw water contains 24 mg/L (as $CaCO_3$) bicarbonate alkalinity. Is this natural alkalinity enough for reaction with 26 mg/L of added alum?

Solution:

From Eq. (14.8):

$$\underset{(600)}{Al_2(SO_4)_3{\cdot}14.3H_2O} + \underset{(3\times162)}{3Ca(HCO_3)_2} \rightarrow \underline{2Al(OH)_3} + 3CaSO_4 + 14.3H_2O + 6CO_2$$

Based on the chemical reaction, 1 mole of alum (600) reacts with 3 moles of bicarbonate alkalinity (3 × 162)

$$\begin{aligned}\text{So 1 mg/L of alum will react with } 3\times162/600 &= 0.81 \text{ mg/L of } Ca(HCO_3)_2\\ &= 0.81\times[CaCO_3/Ca(HCO_3)_2]\\ &= 0.81\times[100/162]\\ &= 0.50 \text{ mg/L as } CaCO_3.\end{aligned}$$

26 mg/L alum will then react with 26 × 0.50 = 13 mg/L alkalinity as $CaCO_3$.

Since raw water contains 24 mg/L alkalinity as $CaCO_3$, which is greater than the 13 mg/L it reacts with, there is no need to add any additional alkalinity.

EXAMPLE 14.5 DETERMINATION OF THE QUANTITY OF GENERATED SLUDGE

A flow of 2 MGD (7.57 MLD) of raw water containing sufficient amount of alkalinity is treated with 26 mg/L of alum. Determine the volume of alum sludge $Al(OH)_3$ generated if the sludge is collected at 2% consistency. Assume that the dry solids have a specific gravity of 2.2 which is determined in the laboratory.

Solution (US Customary and SI Systems):

From Eq. (14.8):

$$\underset{(600)}{Al_2(SO_4)_3{\cdot}14.3H_2O} + \underset{(3\times162)}{3Ca(HCO_3)_2} \rightarrow \underset{(2\times78)}{\underline{2Al(OH)_3}} + 3CaSO_4 + 14.3H_2O + 6CO_2$$

1 mole of alum (600) produces 2 moles of $Al(OH)_3$ (2 × 78).

a. 1 lb of alum will produce (2 × 78)/600 = 0.26 lb $Al(OH)_3$.

b. 1 kg of alum will produce (2 × 78)/600 = 0.26 kg $Al(OH)_3$.

From Example 14.3, it was determined that treating 2 MGD (7.57 MLD) with 26 mg/L alum will need 433.68 lb/d (196.82 kg/d) of alum. Thus the added alum will produce the following quantities of $Al(OH)_3$:

a. 0.26 × 433.68 = 112.8 lb/d.

b. 0.26 × 196.82 = 51.2 kg/d.

The specific gravity of alum sludge at 2% solids content is determined from Eq. (14.16):

$$\frac{1}{S_s} = \sum_{i=1}^{n} \frac{W_i}{S_i}$$

$$\frac{1}{S_s} = \frac{0.98}{1} + \frac{0.02}{2.2}$$

$$S_s = 1.01.$$

The volume of alum sludge is then calculated as follows:

a. (11.8 lb/d)(100/2)[1/(8.34 lb/gal)](1/1.01) = **669 gal.**

b. (51.2 kg/d)(100/2)[1/(1 kg/L)](1/1.01) = **2,532 L.**

14.6.3 Sodium Aluminate

The main difference between sodium aluminate and other common coagulants, alum and iron salts, is its being alkaline rather than acidic in solution. It reacts with the natural carbon dioxide acidity and produces aluminum hydroxide floc:

$$2NaAlO_2 + CO_2 + 3H_2O \rightarrow \underline{2Al(OH)_3} + Na_2CO_3 \quad \textbf{(14.17)}$$

Sodium aluminate is most commonly used in combination with alum in the treatment of boiler water. This combination in water also produces aluminum hydroxide floc in a similar fashion to the previous reaction of sodium aluminate with carbon dioxide acidity:

$$6NaAlO_2 + Al_2(SO_4)_3 \cdot 14.3H_2O \rightarrow \underline{8Al(OH)_3} + 3Na_2SO_4 + 2.3H_2O \quad \textbf{(14.18)}$$

Sodium aluminate can be produced by dissolving alumina in sodium hydroxide. The main deterrent to the wide scale use of this coagulant is its relatively high cost.

14.6.4 Magnesium Coagulant

Magnesium ions can be precipitated as magnesium hydroxide; therefore, it can be used as an effective coagulant for removal of color, turbidity, and other impurities from natural waters. The chemistry of magnesium coagulation is a combination of that of water softening and coagulation.

Similar to other metal coagulants, such as alum and ferric chloride, sufficient lime or equivalent must be added to satisfy the carbon dioxide demand, to convert all bicarbonate alkalinity to carbonate alkalinity, and to precipitate gelatinous magnesium hydroxide. The following are the chemical reactions of magnesium coagulation:

$$Ca(OH)_2 + CO_2 \rightarrow H_2O + CaCO_3 \quad \textbf{(14.19)}$$

$$Ca(OH)_2 + Ca(HCO_3)_2 \rightarrow 2CaCO_3 + 2H_2O \quad \textbf{(14.20)}$$

$$Ca(OH)_2 + Mg(HCO_3)_2 \rightarrow CaCO_3 + MgCO_3 + 2H_2O \quad \textbf{(14.21)}$$

$$Ca(OH)_2 + MgCO_3 \rightarrow CaCO_3 + \underline{Mg(OH)_2} \quad \textbf{(14.22)}$$

In water treatment practice, pH must be maintained at 10.7 level during rapid mixing, coagulation, flocculation, and clarification. After clarification (sedimentation or dissolved air flotation), carbon dioxide is added to the clarified effluent for pH adjustment and stabilization before filtration, disinfection, water storage, and consumption.

The magnesium coagulant, such as magnesium carbonate trihydrate, forms a sludge containing magnesium hydroxide, calcium, carbonates, and impurities; the $Mg(OH)_2$ portion of the sludge can be recovered for reuse by recarbonation:

$$\underline{Mg(OH)_2} + CO_2 \rightarrow Mg(HCO_3)_2 \quad \textbf{(14.23)}$$

The solubilized magnesium bicarbonate can be vacuum filtered and recycled to the rapid mixer for water treatment again. The $CaCO_3$ portion of the sludge can be treated by flotation and centrifugation, and recalcined in a kiln for lime and CO_2 recovery:

$$CaCO_3 + \text{heat} \rightarrow CaO\ (\text{lime}) + CO_2 \quad \textbf{(14.24)}$$

14.6.5 Polymeric Inorganic Salts

Polymeric ferric and aluminum salts are being increasingly used to coagulate turbid waters. PACl is used on a large scale in the treatment of potable water in the United States, Japan, France, Germany, China, South Korea, and Taiwan. Although commercial preparations are available in the market, they are more commonly prepared on site by the addition of a base to neutralize concentrated solutions of ferric and aluminum salts. The polymerization is affected by

1. The concentration of the salt solution
2. The type and concentration of the base solution
3. Ionic strength
4. Temperature

Polyferric sulfate (PFS) is another polymeric inorganic salt, which is used for water and wastewater treatment in the United States, Canada, and China. Its chemical formula is

$$[Fe_2(OH)_n(SO_4)_{3-(n/2)}]_x$$

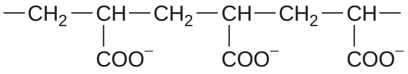
where $n < 2$ and $x > 10$.

PFS is commercially supplied as a reddish brown solution. It complies with the AWWA Standard for ferric sulfate (B406) and is NSF/ANSI Standard 60 certified for use in potable water.

14.6.6 Organic Polymers

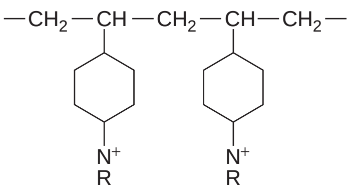
Polyelectrolytes are linear or branched chains of small identical subunits—sometimes of two or three different kinds of subunits. The subunits contain ionizable -COOH, -OH, $-POP_3 H_2$, $-NH_2^+$, and $R_1NR_2^+$ groups. Polymers of this kind differ from nonelectrolyte polymers in the same way that low molecular weight electrolytes differ from nonelectrolytes. They are soluble in water, conduct electricity, and are affected by the electrostatic forces between their charges. Proteins, protamines, nucleic, pectic, and alginic acids, polysaccharides, and numerous poly acids are natural polyelectrolytes. Synthetic polyelectrolytes are formed by polymerization of simple substances, known as monomers. Long-chain molecules of this kind may be designed to contain from 2–3 to nearly 106 subunits. Cationic, anionic, and ampholytic (cationic–anionic) polyelectrolytes can be prepared by choosing suitable monomers. Examples follow:

Polyacrylate exemplifies an anionic polyelectrolyte:

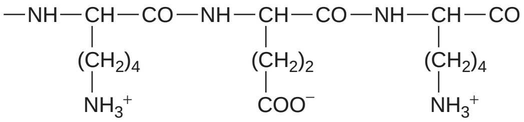


$$-CH_2-\underset{COO^-}{\underset{|}{CH}}-CH_2-\underset{COO^-}{\underset{|}{CH}}-CH_2-\underset{COO^-}{\underset{|}{CH}}-$$

Polyvinylpyridinium salts exemplify cationic polyelectrolytes:



Polyamino acids, such as polylysine-glutamic acid, exemplify polyampholites:



$$-NH-\underset{\underset{NH_3^+}{(CH_2)_4}}{CH}-CO-NH-\underset{\underset{COO^-}{(CH_2)_2}}{CH}-CO-NH-\underset{\underset{NH_3^+}{(CH_2)_4}}{CH}-CO$$

In principle these polymers are not unlike some polyelectrolytic colloids found in natural waters. Particle charge depends on the degree of ionization and, consequently, on the pH of the medium. Natural and synthetic macromolecules have a pronounced effect on the stability of colloidal dispersions.

Polymers function as excellent coagulants due to their ability to destabilize particles by charge neutralization, interparticle bridging, or both. Anionic and nonionic polymers destabilize negatively charged colloidal particles through their bridging effect. Cationic polymers, on the other hand, are able to destabilize and coagulate such particles by both charge neutralization and interparticle bridging.

Factors that play a role in the effectiveness of polymers in accomplishing their function as coagulants include the following:

1. *Polymer properties*
   - (a) Functional groups on polymers. The type of groups is important for specific bonding to sites on the surface of colloidal particles.
   - (b) Charge density.
   - (c) Molecular weight and size. Large size is important for the effectiveness of anionic and nonionic polymers.
   - (d) Degree of branching.
2. *Solution characteristics*
   - (a) pH. It can affect the charge on both polymers and colloidal particles.
   - (b) Concentration of divalent cations (Ca^{2+}, Mg^{2+}). These are necessary to enable anionic polymers to effectively destabilize negatively charged colloids.

Accurate and precise control of dosage is very important for feeding of polymers in treatment plants. There is a narrow range for maximum performance. Concentrations lower than necessary will not produce effective coagulation; while overdosing of polymers will result in charge reversal and restabilization of the colloidal system. Also polymers are more expensive compared to metallic salts. However, this is usually more than compensated for by the lower polymer dosage as well as the reduced sludge production.

The flocculation of bacteria with cationic polyelectrolytes is illustrated in Fig. 14.6. With increasing amounts of added polyelectrolyte, a greater fraction of the microbial surface becomes covered until flocculation becomes optimal. Beyond this point there is mutual repulsion and redispersion of the cells.

Flocculation by polyelectrolytes proceeds by a bridging mechanism, so that close-range chemical interaction, that is, formation of hydrogen and coordinative bonds, is much more important than electrostatic interaction. Accordingly flocculation may be efficient even when the polyelectrolytes carry a net charge of the same sign as the particles being flocculated. Negatively charged clays, for instance, may be flocculated expediently with negatively charged—that is, anionic—polyacrylamides. Coagulation of colloidal dispersions by neutral salts (e.g., $CaC1_2$) is an entirely different matter. Dispersions agglomerate because the repulsive potential of the double layer surrounding the particles is reduced. The agglomerates produced by the two mechanisms possess different properties and, therefore, respond differently to settling and filtration.

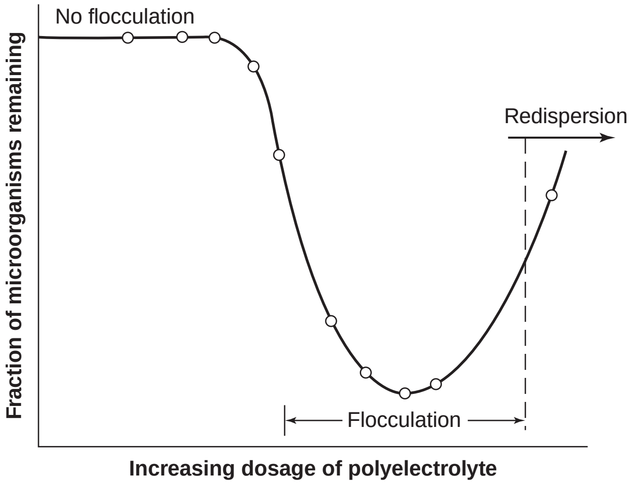


Figure 14.6 Flocculation of dispersed microorganisms by a cationic polyelectrolyte (Purifloc 602, Dow Chemical Company) at constant pH.

14.6.7 Coagulant Aids

Coagulant aids are sometimes used to achieve optimum conditions for coagulation and flocculation. The aim is to obtain faster floc formation, produce denser and stronger flocs, decrease the coagulant dosage, broaden the effective pH band, and improve the removal of turbidity and other impurities. Coagulant aids include four typical types:

1. *Alkalinity addition.* Alkalinity must be added to waters that may not have sufficient natural alkalinity to react with the acidic metallic coagulants to produce a good floc. Alkalinity is commonly supplemented in the form of the hydroxide ion by the addition of hydrated lime, $Ca(OH)_2$, or in the form of the carbonate ion by the addition of soda ash, Na_2CO_3.
2. *pH adjustment*. Acids and alkalis are used to adjust the pH of the water to fall within the optimal pH range for coagulation. pH reduction is usually done by the addition of sulfuric and phosphoric acids. Increasing the pH is achieved by the addition of lime, sodium hydroxide, and soda ash.
3. *Particulate addition*. The addition of bentonite clays and activated silica (sodium silicate treated with sulfuric acid or alum) is very useful in coagulating low turbidity waters. These coagulant aids, when added in sufficient amounts, can increase the particulate concentrations to such an extent that more rapid coagulation will take place. In addition both activated silica and clays serve as weighing agents that produce denser and better settling floc.
4. *Polymers.* The use of organic polymers has recently replaced activated silica as a coagulant aid. Polymers can produce the same impact on coagulation and they are applied at much lower concentrations and are easier to use. Anionic and nonionic polymers are used with ferric and aluminum salts to provide the interparticulate bridging for effective coagulation. Polymers will tend to produce stronger and faster settling flocs and can reduce the metallic salt dosage that would have been required without polymers.

14.7 COAGULATION CONTROL

Theoretical analysis of coagulation is essential for understanding the process, for knowing how it works and what it can achieve as well as for discerning how to obtain the maximum performance out of it. There are four types of colloidal systems:

1. *Type I. High colloidal concentration, low alkalinity*. This is the least complicated system to treat. At low 4–6 pH levels metallic salts in water produce positively charged hydroxometal polymers. These in turn destabilize the negatively charged colloids by adsorption and charge neutralization. The high concentration of particulate material provides an ample opportunity for contact and building of good flocs. As a result, one has to determine only one variable—the optimum coagulant dosage.
2. *Type II. High colloidal concentration, high alkalinity.* Destabilization can also be accomplished, as in type I, by adsorption and charge neutralization. However, in order to overcome the high alkalinity, there are two possible approaches. One alternative is to feed a high coagulant dosage that is sufficient to consume the excess alkalinity as well as to form the positively charged hydroxometal polymers. The second alternative is to add an acid to lower the pH before feeding the coagulant. In this case one has to determine two variables—the optimum coagulant dosage and optimum pH.
3. *Type III. Low colloidal concentration, high alkalinity.* Because of the low chance of interparticle contacts due to the low colloidal concentration, the feasible approach in this case is to achieve sweep coagulation by feeding a high coagulant dosage that results in the entrapment of the colloidal particles in the metal hydroxide precipitate. A second alternative approach is to add a coagulant aid that will increase particle concentration and hence the rate of interparticle contact. A lower coagulant dosage will then be needed to achieve coagulation by charge neutralization.
4. *Type IV. Low colloidal concentration, low alkalinity.* This is the most difficult case to deal with. The low colloidal concentration and depressed rate of interparticle contacts do not allow effective coagulation by adsorption and charge neutralization. On the other

hand, the low alkalinity and low pH of the suspension do not enable rapid and effective destabilization by sweep coagulation. Coagulation in this system can be achieved by the addition of a coagulant aid (increase colloidal concentration), addition of lime or soda ash (increase alkalinity), or the addition of both but at lower concentrations.

However, because the process is so complex and the number of variables so large, in most cases it is not feasible neither to predict the best type of coagulant and optimum dosage nor the best operating pH. The most practical approach is to simulate the process in a laboratory setting using the *jar test*. Other available alternatives and/or supplementary techniques include the *zetameter* (electrophoretic measurement) and the *streaming current detector* (*SCD*).

14.7.1 Jar Test

The jar test is the most valuable tool available for developing design criteria for new plants, for optimizing plant operations, and for the evaluation and control of the coagulation process. A jar test apparatus is a variable speed, multiple station or gang unit that vary in configuration depending on the manufacturer. The differences, such as the number of test stations (usually six), the size (commonly 1000 mL), and shape of test jars (round or square), method of mixing (paddles, magnetic bars, or plungers), stirrer controls, and integral illumination, do not exert an appreciable impact on the performance of the unit (see Fig. 14.7).

The jar test can be run to select each of the following:

1. Type of coagulants
2. Dosage of coagulants
3. Coagulant aid and its dosage
4. Optimum operating pH
5. Sequence of chemical addition
6. Optimum energy and mixing time for rapid mixing
7. Optimum energy and mixing time for slow mixing

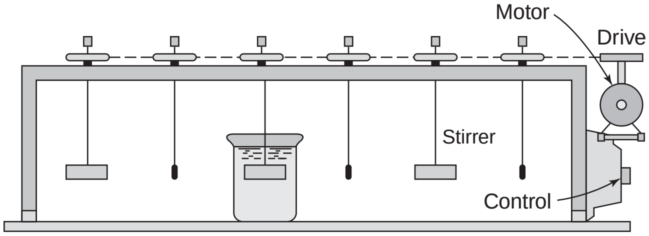


Figure 14.7 Laboratory apparatus for determining optimum dosage of coagulating chemicals and needed stirring (After Fair et al., 1971).

The detailed procedure for the setting up, running, and interpreting a jar test is explained in various publications. Basically, for dosage optimization samples of water/wastewater are poured into a series of jars, and various dosages of the coagulant are fed into the jars. The coagulants are rapidly mixed at a speed of 60–80 rpm for a period of 30–60 seconds then allowed to flocculate at a slow speed of 25–35 rpm for a period of 15–20 minutes. The suspension is finally left to settle for 20–45 minutes under quiescent conditions. The appearance and size of the floc, the time for floc formation, and the settling characteristics are noted. The supernatant is analyzed for turbidity, color, suspended solids, and pH. With this information in hand, the optimum chemical dosage is selected on the basis of best effluent quality and minimum coagulant cost.

Jar tests are simple, but certain aspects require attention. Because jar tests are not very accurate for low turbidity water, coagulant dosages for influent turbidities less than 5 NTU are difficult to ascertain with only jar test results based on settled water turbidity. In such cases, it may be appropriate to filter settled water through filter paper before measuring turbidity. Careful attention is necessary to ensure that the jar test water temperature is the same as that of the plant influent. In addition, jar tests are inappropriate for establishing expected performance of direct filtration plants because the sedimentation process simulated in the jar test is not part of a direct filtration system.

14.7.2 Zetameter

The *zeta potential* measures the net charge of the colloidal particle, and it is dependent on the distance through which the charge is effective:

$$\zeta = 4\pi\delta q/\varepsilon \qquad \textbf{(14.25)}$$

where

ζ = zeta potential
q = charge at the shear surface
δ = thickness of diffuse layer
ε = dielectric constant of the liquid

Most naturally occurring colloidal particles are negatively charged. The more negative is the charge, the higher is the zeta potential, and the greater will be the repulsive force between the particles and hence the greater is the stability of the system. The reverse is also true. As the zeta potential approaches zero, the charges become so low that the repulsion becomes less effective and conditions become ideal for flocculation. Zeta potential after coagulation ranges from +5 to −10 millivolts (mV). Organic colloids require near-zero zeta potentials for coagulation, while clay colloids require negative zeta potentials. The relationship between colloidal stability and zeta potential is shown in Table 14.3.

Table 14.3 Degree of coagulation as a function of zeta potential

Degree of coagulation	Zeta potential (mV)
Maximum	+3 to 0
Excellent	−1 to −4
Fair	−5 to −10
Poor	−11 to −20
Virtually none	−21 to −30

Source: Shammas (2005).

The zeta potential of a given suspension can be determined by using the Helmholtz-Smoluchowski equation:

$$\zeta = K\mu v/\varepsilon E \tag{14.26}$$

where

ζ = zeta potential, mV
K = constant
μ = viscosity
v = measured velocity of colloids, μm/s
ε = dielectric constant
E = applied electric field gradient, V/cm

Since the dielectric constant and viscosity are temperature dependent, the measurement should be made at the water operating temperature, otherwise a correction must be applied to compensate for the temperature difference. For example at 25°C, the zeta potential can be found from the following relationship:

$$\xi = 12.9v/E \tag{14.27}$$

The zeta meter, an instrument used for the determination of the zeta potential, is based on electrophoretic mobility measurement. An electric field is applied across an electrophoresis cell containing the colloidal suspension. The transfer of the negatively charged particles toward the anode is observed through a microscope. A prism situated between the eyepiece of the microscope and the cell is rotated until the colloidal particles appear to be stationary. At this point the prism rotation exactly cancels the transfer velocity of the particles. The unit is provided with an averaging computer equipped for digital read-out in millivolts of zeta potential.

Measurements of zeta potential can provide a good indication of a coagulant effectiveness in charge neutralization, and hence can help in the control and optimization of the coagulation process. However, the reader must be reminded that although this technique is helpful it cannot replace the jar test. The reason being that the zeta potential is unable to predict the enmeshment of particles that leads to sweep coagulation.

14.7.3 Streaming Current Detector

The *SCD* serves the same function as the zeta meter in that it measures the colloidal charge. This instrument consists of a piston that slides inside a cylinder. Two electrodes are attached to the ends of the cylinder to transmit the alternating (streaming) current that is generated from a colloidal suspension. When the piston is moved in a reciprocating motion up and down inside the cylinder, the water sample moves into and out of the annular space between the piston and cylinder. The alternating current generated at the electrodes is directly proportional to the charge on the colloidal particles.

There is a strong correlation between the charge measurements performed by the SCD and the zeta meter. Either instrument can be used to evaluate the extent of charge neutralization. An advantage of the SCD is its suitability for on-line installation. This gives the treatment plant operator the feasibility to have automatic feedback control for adjusting the chemical dosage and, hence, allows an effective and optimized coagulation process. However, it should be noted that one drawback of the SCD is that its measurements are a function of the water pH so that variations in pH require corresponding adjustments in the charge readings. Use of an SCD improves plant operations, but is not a substitute for an effective operator. The matching of the zeta potential with specific plant output must be carefully observed because, while there is a high correlation between zeta potential and finished water quality, there is no causal relationship. The lack of careful scrutiny of the relationship between particle charge observation test results and plant effluent will lead to false conclusions about the finished water's clarity.

14.7.4 Colloid Titration for Polyelectrolyte Determination and Coagulation Control

The colloidal titration method developed by Wang et al. (1978) has proven to be a reliable procedure for determining the concentration of natural and synthetic polyelectrolytes. The titration method is based on a color change in the toluidine blue indicator. Initially the cationic polyelectrolyte (analyte) and potassium polyvinylsulphate (titrant) form a preferential complex or colloid. When all of the analyte has been consumed, the excess titrant reacts with the indicator resulting in a color change from blue to blue-violet. This color change is then measured spectrophotometrically. For coagulation process control involving the use of polyelectrolytes, as shown in Fig. 14.8, when the residual added polyelectrolytes become zero, the coagulated water is destabilized and is ready for clarification and filtration.

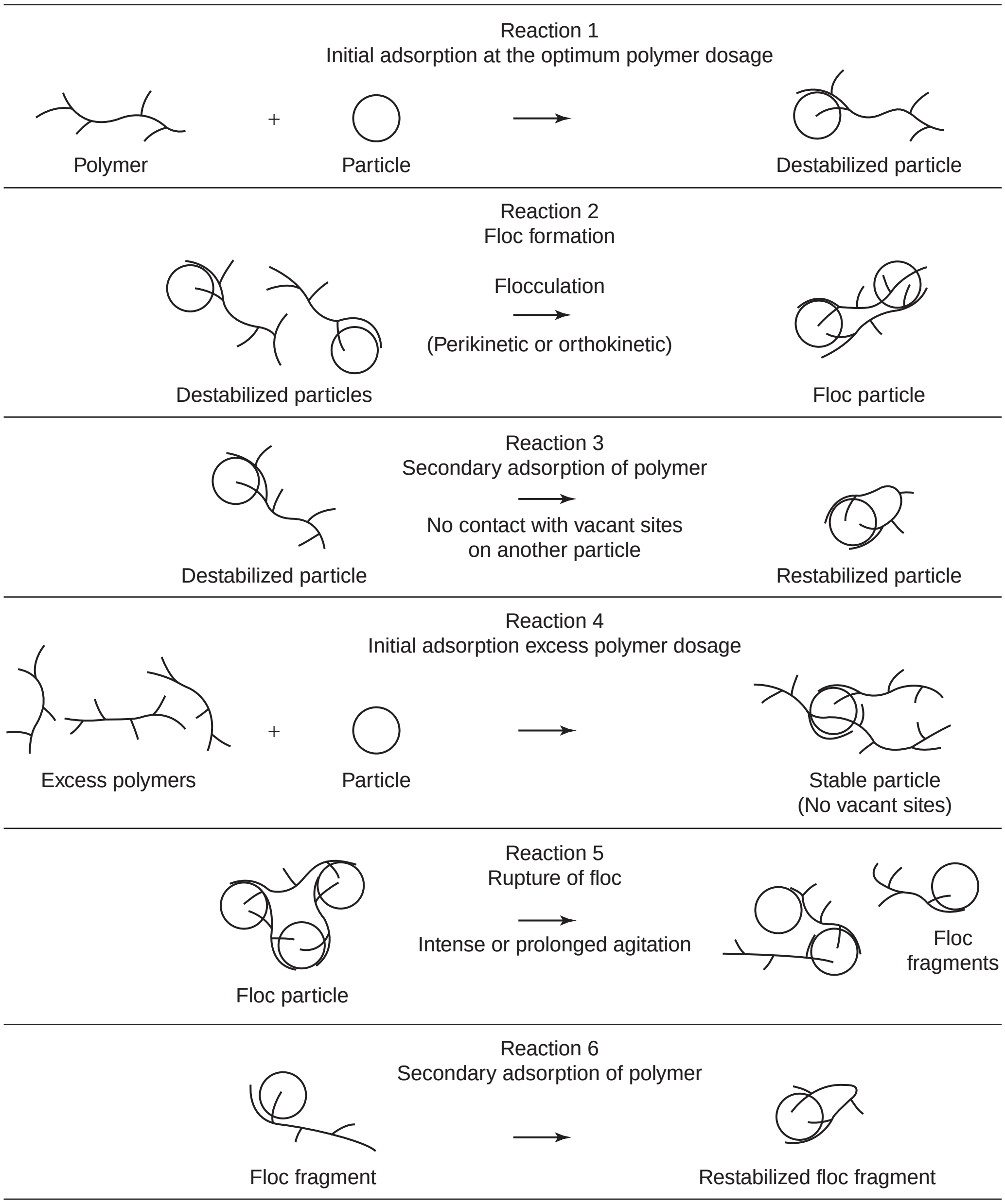


Figure 14.8 Schematic representation of the bridging model for the destabilization of colloids by polymers (US EPA 1999).

PROBLEMS/QUESTIONS

14.1 Define what the colloidal particles are and explain their properties.

14.2 What are the three processes that lead to the electrokinetic properties of colloids?

14.3 Explain the electric double-layer theory and the instability of colloidal particles.

14.4 The forces of attraction between colloidal particles are due to van der Waals' forces. What are the sources of these forces?

14.5 What are the four mechanisms of coagulation?

14.6 Explain the mechanism of colloidal destabilization by entrapment of particles in precipitate. What are the elements that influence this destabilization mechanism?

14.7 How do bridges between particles lead to their destabilization? How can one avoid particles restabilization?

14.8 Many factors play a role in affecting the coagulation process. List what is conserved as the most important of these factors.

14.9 The presence in raw water of divalent cations, such as Ca^{2+} and Mg^{2+}, is commonly considered not only to be helpful in the coagulation of negatively charged colloidal clay particles by anionic polymers but also to be necessary. What are the suggested reasons behind this beneficial effect?

14.10 To what reasons can the increase in rate and effectiveness of coagulation at higher temperatures be attributed?

14.11 What is the main difference between sodium aluminate and other common coagulants, alum and iron salts? Explain with the help of chemical reaction equations.

14.12 Coagulant aids are sometimes used to achieve optimum conditions for coagulation and flocculation. The aim is to obtain faster floc formation, produce denser and stronger flocs, decrease the coagulant dosage, broaden the effective pH band, and improve the removal of turbidity and other impurities. List the typical types of coagulant aids and briefly explain their functions.

14.13 Name and explain the four types of colloidal systems.

14.14 A water treatment plant uses ferric chloride ($FeCl_3$) for coagulation. If the water supply source contains an insufficient amount of bicarbonate alkalinity $Ca(HCO_3)_2$, indicate what kind of chemicals are needed and write complete and balanced chemical equations for all reactions that take place in the coagulation process.

14.15 A plant treating 25 MGD of water is using 30 mg/L of alum [$Al_2(SO_4)_3 \cdot 18H_2O$] as a coagulant. The water contains a natural alkalinity of 15 mg/L as $Ca(HCO_3)_2$. Compute the number of pounds of lime as CaO required per day.

14.16 (SI system) A plant treating 95 ML/d of water is using 30 mg/L of alum [$Al_2(SO_4)_3 \cdot 18H_2O$] as a coagulant. The water contains a natural alkalinity of 15 mg/L as $Ca(HCO_3)_2$. Compute the number of kilograms of lime as CaO required per day.

14.17 A water treatment plant has a flow of 20 MGD. The plant uses 15 mg/L of ferric sulfate [$Fe_2(SO_4)_3$] as a coagulant. If the water contains a natural alkalinity of 12 mg/L as Ca $(HCO_3)_2$, compute the weight of lime as CaO required in lb/day.

14.18 (SI system) A water treatment plant has a flow of 76 ML/d. The plant uses 15 mg/L of ferric sulfate [$Fe_2(SO_4)_3$] as a coagulant. If the water contains a natural alkalinity of 12 mg/L as $Ca(HCO_3)_2$, compute the weight of lime as CaO required in kg/day.

14.19 A water treatment plant is treating 4 MGD of water. If it is using 25 mg/L of ferrous sulfate ($FeSO_4 \cdot 7H_2O$) for coagulation, compute

1. The amount (lb/day) of chlorine needed to oxidize the ferrous element to ferric in lb/d.
2. The amount (lb/day) of natural alkalinity $Ca(HCO_3)_2$ required to be present in the water for efficient coagulation.

14.20 (SI system) A water treatment plant is treating 15 ML/d of water. If it is using 25 mg/L of ferrous sulfate ($FeSO_4 \cdot 7H_2O$) for coagulation, compute

1. The amount (kg/day) of chlorine needed to oxidize the ferrous element to ferric in kg/day.
2. The amount (kg/day) of natural alkalinity $Ca(HCO_3)_2$ required to be present in the water for efficient coagulation.

14.21 A water treatment plant has a capacity of 10 MGD. If it is using 20 mg/L of ferric chloride ($FeCl_3$) as a coagulant, and the water contains a bicarbonate alkalinity [$Ca(HCO_3)_2$] of 10 mg/L as $CaCO_3$, compute the number of pounds of lime as CaO required per day.

14.22 A water treatment plant has a capacity of 38 ML/d. If it is using 20 mg/L of ferric chloride ($FeCl_3$) as a coagulant, and the water contains a bicarbonate alkalinity [$Ca(HCO_3)_2$] of 10 mg/L as $CaCO_3$, compute the number of pounds of lime as CaO required in kg/day.

SPECIAL REFERENCE

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Chapter 15

Screening, Sedimentation, and Flotation

15.1 TREATMENT OBJECTIVES

The unit operations discussed in this chapter are designed to remove successively smaller suspended matters from water. Leaves, the floating debris of lakes and streams, and other sizeable clogging substances respond to removal from water by *screening*. Relatively heavy suspended matter visible to the naked eye can normally be removed from water by *sedimentation*. Light and fine water solids respond selectively to removal by *flotation*.

15.2 SCREENING

Screening intercepts particles that are larger than the smallest openings of wire mesh, parallel bars (racks), or other screen patterns developed for use (1) in surface water intakes and (2) ahead of water pumps.

Trash racks are often included in dams and other intake structures. To facilitate cleaning, they are placed on slopes of 3–6 vertical to 1 horizontal below a working platform from which long-tined rakes can engage the bars to pull up the rakings. Three screens equipped with or without self-cleaning mechanisms are illustrated in Fig. 15.1: (a) a vertical screen constructed either of wire mesh or solely of vertical bars in order to screen out leaves and other debris, (b) a fine screen that can be tilted for cleaning, and (c) a hydraulically cleaned microstrainer. A common arrangement of a different kind is to slide a pair of removable screens into vertical grooves in the walls and bottom of an inlet channel and to leave one in place while the other is being lifted out to flush off the collected debris with high-velocity water jets. Intake screens are normally constructed to 2- to 4-mesh screening, more rarely of screening with 6 meshes to the inch. An exception is fine-mesh screens called microstrainers. In one such screen, pairs of warp and weft stainless steel wires create 160,000 openings per square inch, approximately 23 μ in size. At Denver, CO, lake water flowing radially outward through the screen deposited in excess of 90% of its filter-clogging microscopic organisms and 25% of its turbidity on the screen. About 3% of the water strained had to be recycled as washwater to keep the strainer in service.

15.3 SEDIMENTATION

The erosion of the land by runoff from rainstorms carries vast amounts of soil and debris into streams and other water courses. To the resulting suspensions of mineral soil and organic debris are added community and industrial wastes that are transported through their collecting systems, spilled from storm drains and the overflows of combined systems of sewerage, or left within the treated effluents from separate and combined sewers. Some of the eroded particles and the jetsam of human life and enterprise are heavy enough to settle when flood waters subside, often to be picked up again at rising river stages, to be redeposited farther downstream in successive waves, and eventually to reach the ocean. In this way mud and sludge banks are formed, destroyed, and shifted downstream. When river waters come to rest in ponds, lakes, and impoundages, gravitational forces also succeed in pulling fine or otherwise light particles to the bottom. There they accumulate and build up deltas in inland bodies of water as well as in the sea.

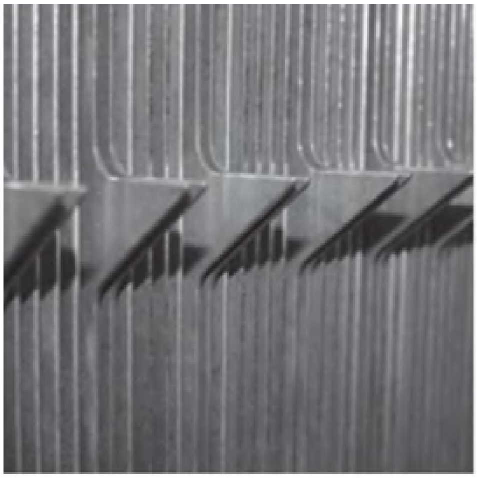
In water treatment, *sedimentation*, or the removal by gravitational settling of suspended particles heavier than water is perhaps the most widely useful operation. When the impurities are separated from the suspending fluid by gravitation and natural aggregation of the settling particles, the operation is called *plain sedimentation*. When chemical or other substances are added to induce or hasten aggregation and settling of finely divided suspended matter, colloidal substances, and large molecules, the operation is called *coagulation*. When chemicals are added to throw dissolved impurities out of solution, the operation is described as *chemical precipitation*.

15.4 TYPES OF SEDIMENTATION

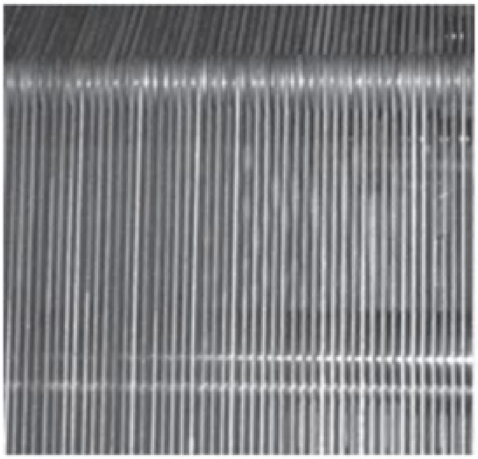
Clarification occurs in various sedimentation regimes, the particular regime of sedimentation would depend mainly on the nature of the settling particles and the concentration of particles in suspension. In general, there are four sedimentation regimes as shown in Fig. 15.2, plotted after Fitch.

Class 1 clarification is a kind of clarification in which particles with little tendency to coalesce settle out of a dilute

Water Engineering: Hydraulics, Distribution and Treatment, First Edition. Nazih K. Shammas and Lawrence K. Wang.



(*a*) Coarse screen



(*b*) Fine screen

(*c*) Microstrainer

Figure 15.1 Racks and screens on water intakes (Courtesy of Hydro-Dyne Engineering with permission).

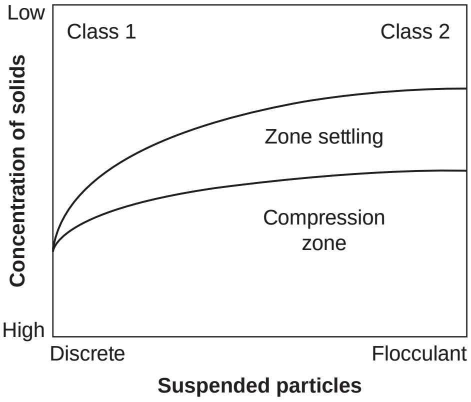


Figure 15.2 Types of settling.

suspension. Such non-flocculating, discrete particles do not alter their size, shape, or weight during settling. Sedimentation of such particles in dilute suspensions is unhindered by the presence of other settling particles. If the particles flocculate, the sedimentation regime in a dilute suspension is called class 2 clarification.

When the flocculant suspension is of an intermediate concentration instead of a dilute one, the particles are closer together and the interparticle forces hold the particles in a fixed position relative to each other. This results in subsidence of the mass of particles as a whole. Such a sedimentation regime is called zone settling. At still higher concentrations, the particles come in actual contact with one another and the weight of the particles is partly supported by the flocculated mass. This type of settling is called compaction or compression.

15.4.1 Settling Velocities of Discrete Particles—Class 1 Clarification

A discrete particle is one that, in settling, is not altered in size, shape, or weight. In falling freely through a quiescent fluid, such a particle accelerates until the frictional resistance,

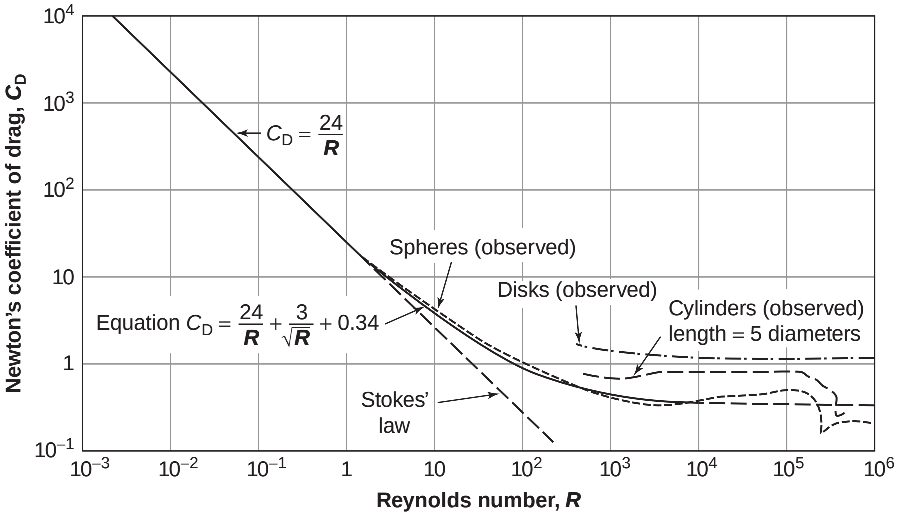


Figure 15.3 Newton's coefficient of drag for varying magnitudes of Reynolds number (After Camp, 1946 with permission).

or drag, of the fluid equals the weight of the particle in the suspending fluid. Thereafter, the particle settles at a uniform (terminal) velocity, which is an important hydraulic attribute or characteristic of the particle. Accordingly,

$$F_I = (\rho_s - \rho)gV \qquad \textbf{(15.1)}$$

where F_I is the impelling force, g the gravity constant, V the volume of the particle, and ρ_s and ρ are, respectively, the mass density of the particle and the fluid.

The drag force F_D of the fluid, on the other hand, is a function of the dynamic viscosity μ and mass density ρ of the fluid, and of the velocity v_s and a characteristic diameter d of the particle. To be fully representative, this diameter must reflect (1) the orientation of the particle relative to its direction of motion, represented, for example, by its cross-sectional area or projected area at right angles to motion, and (2) the relative frictional surface of the particle in contact with the fluid, represented, for example, by its surface area in relation to its volume. Dimensionally, therefore, $F_D = \phi\ (v_s, d, \rho, \mu)$ or, designating dimensional relations by square brackets, $[F_D] = [v_s^x d^y \rho^p \mu^q]$. Introducing the fundamental units of mass m, length l, and time t of the various parameters into this equation, $[\text{mlt}^{-2}] = [m^{p+q} l^{x+y-3p-q} t^{-x-q}]$, and solving for x, y, and p in terms of q,

$$F_D = v_s^2 d^2 \rho \phi \left(v_s d\rho/\mu\right) \qquad \textbf{(15.2)}$$

$$F_D = v_s^2 d^2 \rho \phi (\boldsymbol{R}) \qquad \textbf{(15.3)}$$

$$\boldsymbol{R} = v_s d\rho/\mu = v_s d/\upsilon \qquad \textbf{(15.4)}$$

where $\boldsymbol{R}$ is the **R**eynolds number and the values of ρ, μ, and υ are given in Appendix 3. This dimensionally derived relationship for the frictional drag has been verified experimentally.

By substituting the cross-sectional, or projected, area A_c at right angles to the direction of settling for d^2, the dynamic pressure $\rho v_s^2/2$ for ρv_s^2, and Newton's drag coefficient C_D for $\phi\ (\boldsymbol{R})$,

$$F_D = C_D A_c \rho v_s^2/2 \qquad \textbf{(15.5)}$$

The magnitude of C_D is not constant, but varies with $\boldsymbol{R}$ as shown in Fig. 15.3. For spheres, the observational relationships between C_D and $\boldsymbol{R}$ are approximated by the following equation (upper limit of $\boldsymbol{R} = 10^4$):

$$C_D = \frac{24}{\boldsymbol{R}} + \frac{3}{\boldsymbol{R}^{1/2}} + 0.34 \qquad \textbf{(15.6)}$$

Equations (15.3) and (15.5) can be combined to establish a general relationship for the settling or rising of free and discrete particles, as follows:

$$v_s = \{(2g/C_D)[(\rho_s - \rho)/\rho](V/A_c)\}^{1/2} \qquad \textbf{(15.7)}$$

or, for spherical particles, $V = (\pi/6)d^3$ and $A_c = (\pi/4)d^2$,

$$v_s = \{(4/3)(g/C_D)[(\rho_s - \rho)/\rho]d\}^{1/2}$$

or, approximately,

$$v_s = [(4/3)(g/C_D)(s_s - 1)d]^{1/2} \qquad \textbf{(15.8)}$$

Here s_s is the specific gravity of the particle and $d = 3/2\ V/A_c = 6\ V/A$, where A is the surface area of the particle.

For *eddying resistance* at high **R**eynolds numbers ($\boldsymbol{R} = 10^3$–10^4), C_D assumes a value of about 0.4, and

$$v_s \approx [(3.2g(s_s - 1)d]^{1/2} \qquad \textbf{(15.9)}$$

For viscous resistance at low **R**eynolds numbers ($\boldsymbol{R} < 0.5$), $C_D = 24/\boldsymbol{R}$, and Eq. (15.6) reads as follows:

$$v_s = (g/18)[(\rho_s - \rho)/\mu]d^2 \qquad \textbf{(15.10a)}$$

and,

$$v_s = (g/18)[(s_s - 1)/\upsilon]d^2 \qquad \textbf{(15.10b)}$$

where v_s = settling velocity of particles, ft/s (m/s); g = acceleration of gravity = 32.2 ft/s^2 = 9.81 m/s^2; d = diameter of particle, ft (m); ρ_s = density of particle, lb-s^2/ft^4 (kg/m^3);

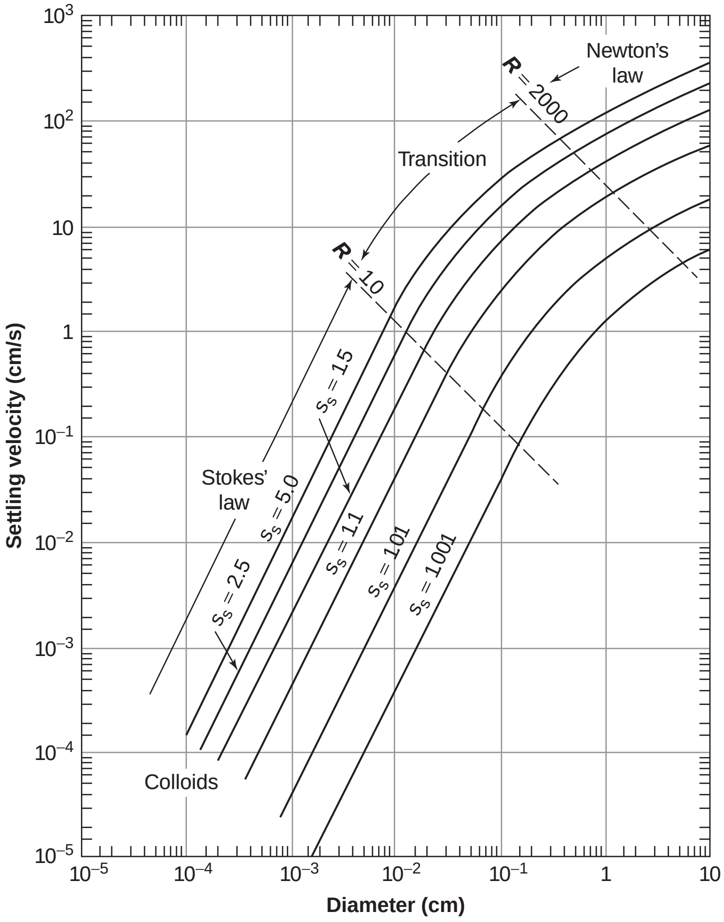


Figure 15.4 Settling and rising velocities of discrete spherical particles in quiescent water at 10°C. For other temperatures, multiply the Stokes values by $\nu/(1.31 \times 10^{-2})$, where ν is the kinematic viscosity at the stated temperature (After Fair et al., 1971).

ρ = density of water, lb-s^2/ft^4 (kg/m^3); μ = absolute viscosity of water, lb-s/ft^2 (N-s/m^2); υ = kinematic viscosity of water, ft^2/s (m^2/s); s_s = specific gravity of particles; and 1 = specific gravity of water.

Equations (15.10a/15.10b) are the Stokes' law, υ being the kinematic viscosity with values shown in Appendix 3. To span the region between the Stokes range and the turbulent range, the curves shown in Fig. 15.4 are useful.

EXAMPLE 15.1 DETERMINATION OF SETTLING VELOCITY OF SPHERICAL PARTICLES IN WARM TEMPERATURE WATER

Find the settling velocity in water at 20°C (68°F) of spherical silica particles of specific gravity 2.65 and a diameter of 5×10^{-3} cm (5×10^{-5} m; 16.4×10^{-5} ft).

Solution 1 (US Customary System):

From Appendix 3, $\nu = 1.080 \times 10^{-5}\,\text{ft}^2/\text{s}$ at 68°F.

From Eq. (15.10b),

$$\begin{aligned} v_s &= (g/18)[(s_s - 1)/\upsilon]d^2 \\ v_s &= (32.2/18)[(2.65 - 1)/(1.080 \times 10^{-5})](16.4 \times 10^{-5})^2 \\ &= \mathbf{7.35 \times 10^{-3}\ ft/s}. \end{aligned}$$

Hence

$$\begin{aligned} \boldsymbol{R} &= v_s d/\upsilon \\ &= (7.35 \times 10^{-3})(16.4 \times 10^{-5})/(1.080 \times 10^{-5}) \\ &= \mathbf{1.1 \times 10^{-1}}, \text{ and } \textbf{Stokes' law applies}. \end{aligned}$$

Solution 2 (SI System):

From Appendix 3, $\nu = 1.010 \times 10^{-6} \mathrm{m}^2/\mathrm{s}$ at 20°C.

From Eq. (15.10b),

$$v_s = (g/18)[(s_s - 1)/\upsilon]d^2$$

$$v_s = (9.81/18)[(2.65 - 1)/(1.010 \times 10^{-6})](5 \times 10^{-5})^2$$

$$= \mathbf{2.22 \times 10^{-3}\ m/s = 0.222\ cm/s}.$$

Hence

$$\boldsymbol{R} = v_s d/\upsilon$$

$$= (2.22 \times 10^{-3})(5 \times 10^{-5})/(1.010 \times 10^{-6})$$

$$= \mathbf{1.1 \times 10^{-1}}, \text{ and } \textbf{Stokes' law applies}.$$

EXAMPLE 15.2 DETERMINATION OF SETTLING VELOCITY OF SPHERICAL FLOC PARTICLES IN COLD TEMPERATURE WATER

Determine the settling velocity of a 10 μm chemical floc particle having a density of 1,100 kg/m^3 in winter at a water temperature of 4°C (39.2°F).

Solution:

At 4°C, $\upsilon = 1.666 \times 10^{-6}\ \mathrm{m}^2/\mathrm{s}$.

$$v_s = (g/18)[(s_s - 1)/\upsilon]d^2$$

$$= (9.81/18)[(1.1 - 1)/1.666 \times 10^{-6}](10 \times 10^{-6})^2$$

$$= 0.0328 \times 10^{-4}$$

$$= \mathbf{3.28 \times 10^{-6}\ m/s}.$$

Check Reynolds number:

$$\boldsymbol{R} = v_s d/\upsilon$$

$$= (3.28 \times 10^{-6})(10 \times 10^{-6})/(1.666 \times 10^{-6})$$

$$= 19.7 \times 10^{-6}$$

$$= \mathbf{1.97 \times 10^{-5}}, \text{ and } \textbf{Stokes' law applies}.$$

It is noted that the chemical floc settles very slowly in winter at low temperatures.

The suspended matter in water and wastewater is seldom spherical. The irregular particles generally composing suspensions possess greater surface area per unit volume than do spheres and, because of this, settle more slowly than do spheres of equivalent volume. Moreover, the frictional drag changes with the orientation of particles relative to the direction of motion. As shown in Fig. 15.5, irregularities in shape exert their greatest influence on drag at high values of $\boldsymbol{R}$. At low values ($\boldsymbol{R} < 10$), the settling velocities of rod-like and disk-like spheroidal particles are, respectively, 78% and 73% of the velocity of an equal-volume sphere. For particles of irregular shape, in general, $A/V = 6/\psi d = S/d$, where ψ is called the sphericity of the particle and $S = 6/\psi$ is called the shape factor.

15.4.2 Hindered Settling of Discrete Particles—Class 2 Clarification

When a discrete particle settles through a liquid in *free fall*, the liquid displaced by the particle moves upward through an area large enough not to interpose friction. In *hindered* settling, by contrast, particles are spaced so closely that the friction rises as the velocity fields around the individual particles interfere. In laboratory observations of attained velocities, the walls of narrow containers in which even a single particle is settling may have a similar hindering influence. However, this *wall effect* becomes negligible at high **R**eynolds numbers or when the particle diameter is less than about 1% of the cylinder diameter. Laminar flow, or its equivalent, may

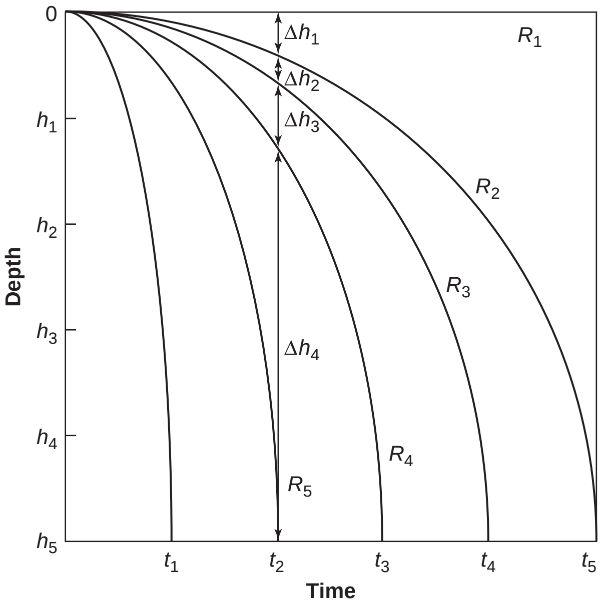


Figure 15.5 Equal percent removal curves for clarification of flocculating suspension.

persist over much of the range of grain size and pore space of interest in sedimentation and filtration, even though flow is forced to pursue irregular paths because of the exterior interference of the particles in suspension.

To determine the effect of flocculation on sedimentation it is necessary to perform settling column analysis or sedimentation tests, since no suitable rational design procedure is available to evaluate such effects. The settling column may be of any diameters but its length should be equal to the depth of the settling tank to be designed. The settling column usually used in laboratories is 6 in. (150 mm) in diameter and about 10 ft (3 m) deep. It contains sampling ports at 2 ft (60 cm) intervals.

To perform sedimentation tests, a flocculating suspension similar in character to the one to be used in the settling tank is placed in the column in such a way as to ensure a uniform distribution of particles of all sizes from top to bottom. Samples are withdrawn from all the ports at various time intervals and the amount of suspended solids contained in the samples is determined. The percent removals of suspended solids by sedimentation are then calculated. For example, if a suspension containing 500 mg/L of suspended solids was introduced into the column and the sample showed a concentration of 100 mg/L, then the percent removed would be 80%.

These percentages are plotted on a depth versus time graph. Points of equal percent removals are joined and a plot as shown in Fig. 15.5 is obtained. Overall removals of suspended solids at a certain detention time (t_2) are given by

$$R_{t2} = \frac{\Delta h_1}{h_5}\left(\frac{R_1+R_2}{2}\right) + \frac{\Delta h_2}{h_5}\left(\frac{R_2+R_3}{2}\right) + \frac{\Delta h_3}{h_5}\left(\frac{R_3+R_4}{2}\right) + \frac{\Delta h_4}{h_5}\left(\frac{R_4+R_5}{2}\right) \tag{15.11}$$

where

R_{t2} = the percent removal at time t_2

$R_1, R_2,\ldots, R_5$ are percent removals

h_5 = the total depth of water in column

Δh_1, Δh_2, Δh_3, Δh_4 are depth increments to successive percent removal curves at time t_2

In order to obtain removals in settling tanks comparable to those indicated by a settling column analysis, it is recommend that the detention time be multiplied by 1.75–2.0 and the overflow rate or design settling velocity by 0.65.

EXAMPLE 15.3 DESIGN OF SEDIMENTATION TANK BASED ON SETTLING COLUMN ANALYSIS

The data shown in Table 15.1 were obtained from a settling column analyses. Find the detention time for a settling tank with 8 ft (2.44 m) effective depth to remove 79% of the suspended solids.

Table 15.1 Suspended solids removal in % for Example 15.3

Depth ft (m)	Time (min)						
	10	20	30	45	60	90	120
2 (0.6)	40	58	69	75	80	84	85
4 (1.2)	24	47	60	69	75	81	84
6 (1.8)	16	39	55	65	70	79	82
8 (2.4)	14	33	52	63	68	76	81

Solution:

The first step in solving the problem is to plot the data on time–depth graph in the same manner as elevations are plotted on a topographic survey map. Points of equal percentage removals are joined to obtain percentage removal lines as shown in Fig. 15.6. These lines are drawn similar to contours on a survey map.

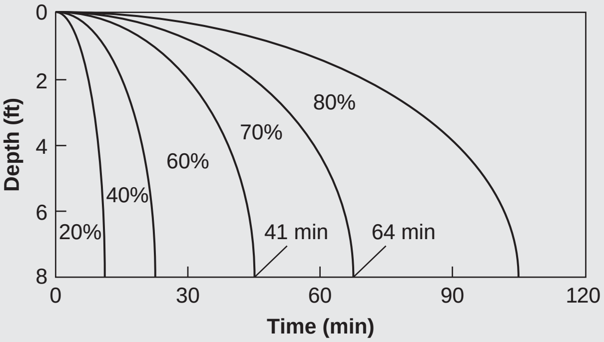


Figure 15.6 Equal percent removal curves for Example 15.3. Conversion factor: 1 ft = 0.3048 m.

To obtain detention time for 75% suspended solids removal we calculate overall percent removals, as per Eq. (15.10), at two different times, one which would give more than 75% removal and the other which gives less than 75% removal. The time required for 75% removal is then obtained by proportion. Examination of the percent removal curves indicate that overall removal at 64 min should be over 75% and that at 41 min should be less than 75%. So calculate overall removals at these two points only.

Overall removal at 64 min equals:

$$R_{t2} = \frac{\Delta h_1}{h_5}\left(\frac{R_1 + R_2}{2}\right) + \frac{\Delta h_2}{h_5}\left(\frac{R_2 + R_3}{2}\right) + \frac{\Delta h_3}{h_5}\left(\frac{R_3 + R_4}{2}\right) + \frac{\Delta h_4}{h_5}\left(\frac{R_4 + R_5}{2}\right)$$

$$= \frac{5.7}{8} \times \frac{70 + 80}{2} + \frac{2.3}{8} \times \frac{80 + 100}{2} = 79.3\%.$$

Overall removal at 41 min equals:

$$R_{t2} = \frac{4.9}{8} \times \frac{70 + 60}{2} + \frac{1.9}{8} \times \frac{80 + 70}{2} + \frac{1.2}{8} \times \frac{80 + 100}{2} = 71.1\%.$$

By proportion 75% removal will be obtained in

$$t = 41 + \frac{75 - 71.1}{79.3 - 71.1} \times (64 - 41)$$

$$= 41 + \frac{3.9}{8.2} \times (64 - 41) = 51.94 \text{ min, say 52 min.}$$

A factor of safety of 2.0 or more is usually employed to this theoretical detention time to obtain design detention time for a settling tank. So **detention time** for the settling tank = 52 × 2 = **104 min.**

15.4.3 Settling of Flocculent Suspensions—Zone Settling

Organic matter and the flocs formed by chemical coagulants or by zoogleal growths tend to agglomerate. The resulting clusters settle more rapidly than the constituent particles or flocs when the amount of included water is small.

Concentrated suspensions have somewhat different settling characteristics than those of dilute suspensions. Figure 15.7 has been plotted to illustrate the effect of a high concentration of suspended solids on settling rates. Curve (*X*) is a plot showing settling of a particle in dilute suspensions of different-size particles. Initially the subsidence of the particle is unhindered and it settles at its own individual velocity of subsidence. The particle accelerates as it moves downward until point B is reached when it starts decelerating. Somewhere between B and C the particle becomes part of the sludge. From C to D the particle is in the compression regime area.

When the concentration of particles is increased, the fast-settling particles form a zone at some distance from the start and settle collectively at a reduced velocity. With further increase in the concentration of particles in the suspension, a point is reached where even the initial subsidence is collective. This is shown in curve (*Y*). The particles settle as a zone or "blanket" leaving a relatively clear liquid above the zone-settling region. Some particles are invariably left behind which settle in the relatively clear water as discrete or

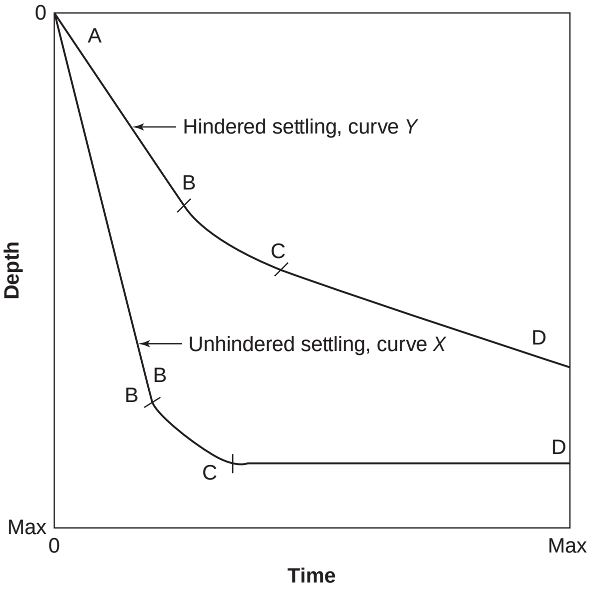


Figure 15.7 Effect of high concentration of suspended solids on settling rates.

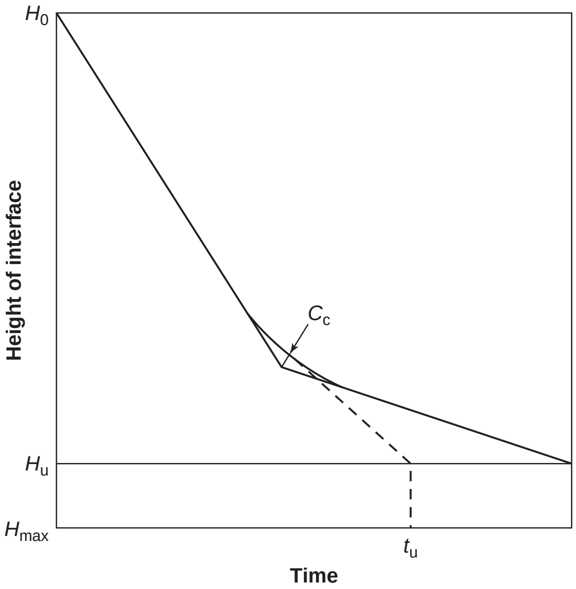


Figure 15.8 Graphical determination of t_u, time required to obtain the desired underflow concentration.

flocculated particles. A distinct interface is formed between the relatively clear liquid and the zone-settling region.

Settling of particles from a suspension with high concentration of suspended solids usually involves both zone settling and compression settling in addition to free settling. The compression-settling region is formed under the zone-settling region in a settling column. Settling tests are usually required to determine the sedimentation characteristics of suspensions where zone settling and compression settling occur. Talmadge and Fitch developed a method to determine the area required for a solids handling system from the results of settling tests. This method is described below.

A settling test is performed with suspension of solids of uniform concentration (c_0) in a settling column of height (H_0). The position of the interface with time is determined and is plotted on a depth time graph (see Fig. 15.8).

The critical area for a solid handling system is given by the equation

$$A = \frac{Qt_u}{H_0} \tag{15.12}$$

where

A = area required for the solids handling thickener, ft^2 (m^2)

Q = volumetric flow rate into thickener, ft^3/s (m^3/s)

t_u = time required to attain underflow concentration (c_u), s

H_0 = initial column height of the interface in the settling column, ft (m)

In the equation above, Q and H_0 are known and t_u is found graphically from the settling curve. First, the point of critical concentration c_c is determined by bisecting the angle formed by extending the tangents to the hindered settling and the compression-settling regions of the settling curve. The bisector cuts the settling curve near the point where compression settling starts. The critical concentration corresponds to the largest cross-section area required for a solids handling system. The value of t_u can be determined by drawing a vertical line to the time axis from the intersection of the tangent at c_c and the horizontal line drawn at depth H_u. H_u is the depth at which all solids are at the desired underflow concentration (c_u):

$$H_u = \frac{c_0 H_0}{c_u} \tag{15.13}$$

where

H_u = the depth at which all solids are at the desired underflow concentration (c_u), ft (m)

c_u = underflow concentration, mg/L or %

c_0 = initial concentration at depth H_0, mg/L or %

Knowing the value of t_u, the area required for solids handling system can be obtained from the equation.

15.4.4 Compression Settling

In the zone of compression, particles come into physical contact with each other and are supported by the layers below. The volume required for sludge in the compression region is also found by settling tests. The approximate rate of consolidation is proportional to the difference in sludge depth at time t and the final depth of sludge after a long period of settling:

$$-\frac{dH_t}{dt} = K(H_t - H_\infty) \tag{15.14}$$

where

H_t = depth of sludge at any time (t), ft (m)

H_∞ = final depth of sludge after a long period of settling, ft (m)

K = constant for a given suspension

Integrating Eq. (15.14) between the limits H_{t1} and H_{t2}, the depths of sludge at times t_1 and t_2, respectively,

$$-\int_{H_{t_1}}^{H_{t_2}} \frac{\mathrm{d}H_t}{H_t - H_\infty} = K\int_{t_1}^{t_2} \mathrm{d}t$$

$$\ln\left(\frac{H_{t_1} - H_\infty}{H_{t_2} - H_\infty}\right) = K(t_1 - t_2)$$

$$H_{t1} - H_\infty = \left(H_{t2} - H_\infty\right)\mathrm{e}^{K(t_1 - t_2)} \quad \textbf{(15.15)}$$

H_∞ or the final depth of sludge, after a long period of time, can be calculated from the above relationship.

Gentle stirring generally compacts sludge in the compression region by breaking up the flocs and permitting water to escape. Mechanical rakes are usually employed to provide the required stirring in the sedimentation tank. In settling column tests it may also be proper to provide stirring if the results are to be used in the design of thickeners with mechanical rakes. Stirring also improves the settling efficiency in the zone-settling region.

15.5 SETTLING BASINS

15.5.1 Efficiency of Ideal Settling Basins

For convenience of discussion, a continuous-flow basin can be divided into four zones: (1) an *inlet zone*, in which influent flow and suspended matter disperse over the cross-section at right angles to flow; (2) a *settling zone*, in which the suspended particles settle within the flowing water; (3) a *bottom zone*, in which the removed solids accumulate and from which they are withdrawn as underflow; and (4) an *outlet zone*, in which the flow and remaining suspended particles assemble and are carried to the effluent conduit (Fig. 15.9).

The paths taken by *discrete* particles settling in a horizontal-flow, rectangular or circular basin are shown in Fig. 15.9. They are determined by the vector sums of the settling velocity v_s of the particle and the displacement velocity v_d of the basin. All particles with a settling velocity $v_s \geq v_0$ are removed, v_0 being the velocity of the particle falling through the full depth h_0 of the settling zone in the detention time t_0. Also, $v_0 = h_0/t_0$, $t_0 = V/Q$, and $V/h_0 = A$, where Q is the rate of flow, V the volumetric capacity of the settling zone, and A its surface area. Therefore, $v_0 = Q/A$ is the surface loading or overflow velocity of the basin. In vertical-flow basins, particles with velocity $v_s < v_0$ do not settle out. By contrast, such particles can be removed in horizontal-flow basins if they are within vertical striking distance $h - vt_0$ from the sludge zone. If y_0 particles possessing a settling velocity $v_s \leq v_0$ compose each size within the suspension, the proportion y/y_0 of particles removed in a horizontal-flow tank becomes

$$y/y_0 = h/h_0 = (v_s t_0)/(v_0 t_0) = v_s/v_0 = v_s/(Q/A) \quad \textbf{(15.16)}$$

These relationships follow also from the geometry of Fig. 15.9.

Equation (15.16) states that, for *discrete* particles and *unhindered* settling, basin efficiency is solely a function of the settling velocity of the particles and of the surface area of the basin relative to the rate of flow, which, in combination, constitute the surface loading or overflow velocity. The efficiency is otherwise independent of basin depth and displacement time or detention period. It follows that particles with settling velocity $v_s \geq v_0$ are removed and that particles with velocity $v_s < v_0$ can be fully captured in horizontal-flow basins if false bottoms, trays, or stacks of hexagonal tubes or similar structures are inserted at vertical intervals $h = v_s t_0$. The larger the number of inserts the smaller can be the settling velocity of particles. Conceptually, therefore, filters are approached by settling basins with a very large number of inserts.

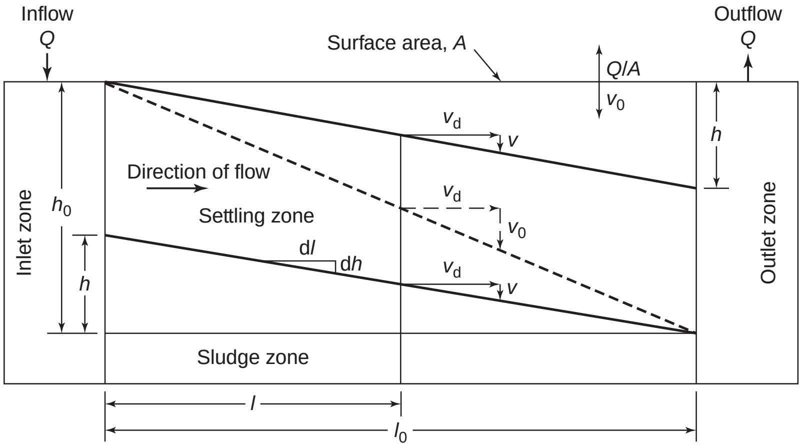


Figure 15.9 Settling paths of discrete particles in a horizontal-flow tank (idealized). The capacity of the settling zone is V, and its surface area is A (After Fair et al., 1971).

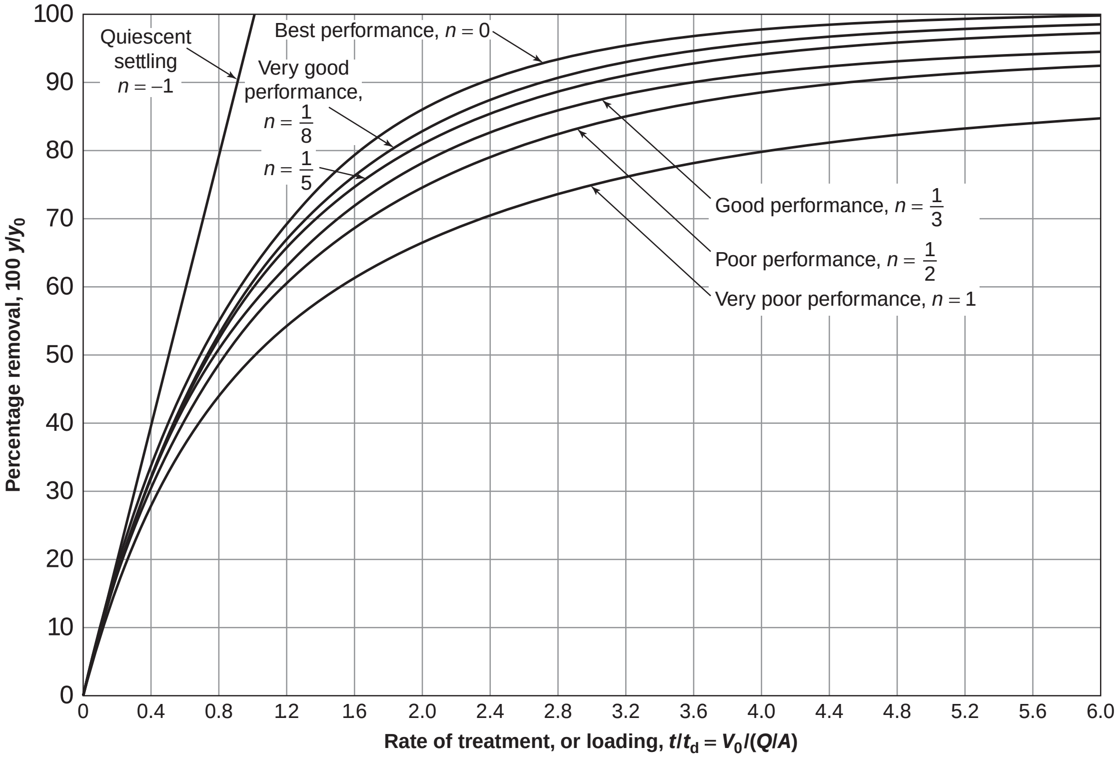


Figure 15.10 Performance curves for settling basins of varying effectiveness. (After Hazen, 1904 with permission).

15.5.2 Reduction in Settling Efficiency by Currents

Settling-basin efficiency is reduced by currents that short-circuit their flows. Among them are (1) eddy currents, set up by the inertia of the incoming fluid; (2) surface currents, wind induced in open basins; (3) vertical convection currents, thermal in origin; (4) density currents, causing cold or heavy water to underrun a basin and warm or light water to flow across its surface; and (5) currents induced by outlet structures. Also, in horizontal-flow tanks, as material settles to the bottom, a downward current is induced near the tank inlet with a corresponding upwelling near the outlet.

In accordance with the concepts of longitudinal change in treatment response, the amount of sediment of settling velocity v_0 reaching the tank bottom in time t is $y/y_0 = 1 - (1 + nkt)^{-1/n}$, where n is a coefficient that identifies basin performance and $k = 1/t_0 = v_0/h_0$ is a coefficient that characterizes the settleability of the sediment in terms of the time t_0 required for a particle with settling velocity v_0 to settle through the filled depth h_0 of the basin. The basin coefficient n has a lower limit of 0 and an upper limit of 1. Because $t/t_0 = v_0/(Q/A)$, moreover,

$$y/y_0 = 1 - [1 + nv_0/(Q/A)]^{-1/n} \tag{15.17}$$

The validity of this equation is supported by Hazen's theory of sedimentations. Settling curves for various values of n are shown in Fig. 15.10.

From a mathematical analysis of longitudinal mixing in settling tanks, Thomas and Archibald have concluded that the value of n can be approximated by the ratio of the difference between the mean and modal flowing-through periods to the mean flowing-through period (Fig. 15.11). Common values of n are 0, $^1/_8$, $^1/_4$, $^1/_2$, and 1, respectively, for ideal, very good, good, poor, and very poor performance.

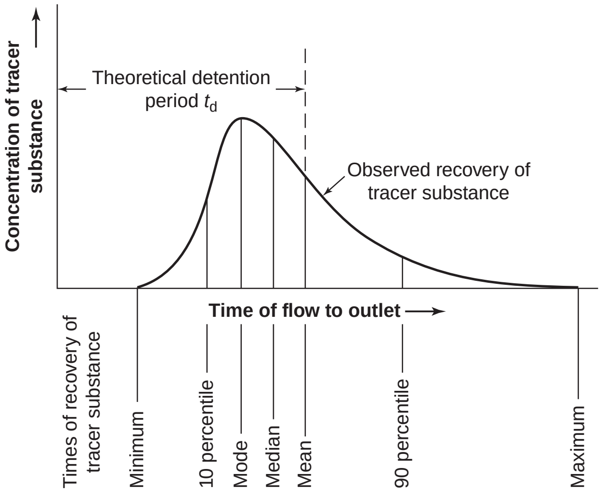


Figure 15.11 Dead spaces and short-circuiting in a settling basin are reflected in the concentration and time of recovery of tracer substances (After Fair et al., 1971).

EXAMPLE 15.4 DETERMINATION OF SETTLING VELOCITY AND SIZE OF PARTICLES AT A SPECIFIED REMOVAL RATE

Find the settling velocity and size of particles of specific gravity 1.001, of which 80% are expected to be removed in a very good settling basin at an overflow rate of 1,000 gpd/ft^2 (40.8 m^3/d/m^2), if the water temperature is 10°C (50°F).

Solution 1 (US Customary System):

From Fig. 15.10,

$$\begin{aligned} Q/A &= 1{,}000\,\text{gpd/ft}^2\,[(1.547\times10^{-6}\,\text{ft}^3/\text{s})/(\text{gpd})]\times30.48\,\text{cm/ft} = 4.72\times10^{-2}\,\text{cm/s} \\ v_0/(Q/A) &= 1.8\ \text{for}\ n = \ \text{and}\ y/y_0 = 80\% \\ v_0 &= 1.8(4.72\times10^{-2}\,\text{cm/s}) \\ &= 8.\mathbf{5}\times\mathbf{10^{-2}\,cm/s}. \end{aligned}$$

From Fig. 15.4, therefore, ***d* = 0.15 cm.**

Solution 2 (SI System):

$$\begin{aligned} Q/A &= 40.8\,\text{m}^3/\text{d/m}^2 = 40.8\,\text{m/d} = 4{,}080\,\text{cm}/\,(1{,}440\times60)\ \text{s} \\ &= 0.0472\,\text{cm/s} \\ &= 4.72\times10^{-2}\,\text{cm/s}. \end{aligned}$$

From Fig. 15.10,

$$\begin{aligned} v_0/\,(Q/A) &= 1.8\ \text{for}\ n = \frac{1}{8}\ \text{and}\ y/y_0 = 80\% \\ v_0 &= 1.8\,(Q/A) = 1.8\left(4.72\times10^{-2}\,\text{cm/s}\right) \\ &= \mathbf{8.5\times10^{-2}\,cm/s}. \end{aligned}$$

From Fig. 15.4, therefore, ***d* = 0.15 cm**.

15.5.3 Short-Circuiting and Basin Stability

In an ideal basin displacement is steady and uniform, and each unit volume of fluid is detained for a time $t_d = V/Q$. Even in well-designed basins, however, some of the inflow reaches the outlet in less than the theoretical detention period and some takes much longer to do so. The degree of short-circuiting and extent of retardation can be measured by adding a single but adequate amount of dye, electrolyte, or other tracer substance to the basin influent and observing the rise in concentration of the substance reaching the outlet as time passes (Fig. 15.12). Modal, median, and mean flowing-through periods identify the central tendency of the time–concentration distribution, and percentiles reflect its variance. Relating observed times to the theoretical detention period t_d permits making comparisons between different basins.

Spaces in which the flow rotates upon itself receive no suspended solids, do no work, and reduce the effective capacity of the basin while shortening the flowing-through times relative to the theoretical detention period, t_d. In the absence of such currents, the ratio of the mean time to t_d must equal unity. In the absence of short-circuiting, the mean, median, and mode must coincide. Short-circuiting is characterized, therefore, by the ratio of the mode or median to the mean being less than unity or by the ratio of the difference between the mean and mode, or the mean and median, to the mean being large.

Figure 15.12 Rectangular sedimentation tanks (Courtesy of US EPA).

If there is some interchange of flow between ineffective spaces and active portions of the tank, the time–concentration curve becomes unduly long, because small amounts of tracer material are released but slowly for capture in the effluent. The proportion of tracer substance arriving at the outlet in a given time equals the ratio of the area under the frequency curve as far as the given time to the total area or total dose of tracer substance. Percentile ratios then identify the degree of variability of exposure to sedimentation. If the time–concentration curve of a basin does not reproduce itself reasonably well in repeated tests, flow through the tank is not stable, and tank performance may be erratic.

15.5.4 Scour of Bottom Deposits

As suggested by Ingersoll, McKee, and Brooks, fine, light, and flocculent solids settling from coagulated waters, biologically treated wastewaters, and the like may be lifted from the sludge zone when $v_s = (\tau/\rho)^{1/2}$. Here τ is the shear stress at the liquid–sludge interface and ρ is the density of the supernatant water. Because $\tau/\rho = grs$, and $rs = (f/8\ g)v_d^2$ in the Weisbach–Darcy equation, $(\tau/\rho)^{1/2} = v_s = v_d(f/8)^{1/2}$, where v_d is the displacement velocity, and

$$v_d = [(8/f)(\tau/\rho)]^{1/2} = (8/f)^{1/2} v_s \qquad \textbf{(15.18)}$$

where v_d = displacement velocity, ft/s (m/s); f = Weisbach–Darcy friction factor, dimensionless; v_s = settling velocity of particles, ft/s (m/s).

It follows that v_d should be kept well below 18 v_s for $f = 2.5 \times 10^{-2}$. Accordingly, the ratio of length to depth in rectangular basins or of surface area A to cross-sectional area a must be kept below

$$A/a = l_0/h_0 = (v_d t_d)/(v_0 t_0) = (8/f)^{1/2}(t_d/t_0)(v_s/v_0) \qquad \textbf{(15.19)}$$

for $(t_d/t_0) = 1$. Then, where $v_s = v_0$, $l_0/h_0 \le 18$.

To avoid scouring velocities, the ratio of length to depth, or surface area to cross-sectional area, must be kept below a value of

$$l_0/h_0 = A/a = V_d t_d/v_0 t_0 = (t_d/t_0)[(6k/f)C_D]^{1/2} \qquad \textbf{(15.20)}$$

where t_d/t_0 equals unity for an ideal channel.

That the removal efficiency of some suspensions in settling tanks is stepped up by gentle stirring is a not-uncommon observation. The potential of such useful power dissipation can be evaluated by the ratio of expected conjunctions in terms of von Smoluchowski's orthokinetic flocculation, on the one hand, and quiescent settling (Eq. 15.15), on the other hand. This is also a useful consideration in sludge thickening.

EXAMPLE 15.5 FINDING DISPLACEMENT VELOCITY AND TANK LENGTH-TO-DEPTH RATIO WITHOUT CAUSING FLOC SCOURING

Find for alum floc ($s_s = 1.1$), 10^{-1} cm in diameter, the displacement velocity at which the floc can be removed without danger of resuspension and the length-to-depth ratio of the settling unit in which the removal can be effected. Assume a Weisbach–Darcy friction factor $f = 3.0 \times 10^{-2}$ and a temperature of 10°C.

Solution:

By Eq. (15.18),

$$v_d = \left[(8/f)(\tau/\rho)\right]^{1/2} = (8/f)^{1/2} v_s$$

$$v_d = \left[8/\left(3 \times 10^{-2}\right)\right]^{1/2} v_s$$

$$= 16.3 v_s.$$

From Fig. 15.4, for $d = 10^{-1}$ cm and $s_s = 1.1$

$$v_s = 3.0\,\text{cm/s}$$

Accordingly,

$$v_d = 16.3 \times 3.0 = 48.9\,\text{cm/s}$$

$$v_d = \mathbf{1.60\,ft/s}$$

and by Eq. (15.19), and $v_s = v_0$

$$l_0/h_0 = (8/f)^{1/2}\left(t_d/t_0\right)\left(v_s/v_0\right)$$

$$l_0/h_0 = 16.3\left(t_d/t_0\right) = \mathbf{16.3}$$

for an ideal basin $t_d/t_0 = 1.0$.

15.5.5 Elements of Tank Design

Each of the four functional zones of sedimentation basins (Fig. 15.9) and flotation tanks (Fig. 15.23) present special problems of hydraulic and process design that depend on the behavior of the suspended matter within the tank, during removal, and after deposition as sludge or scum.

Size, density, and flocculating properties of the suspended solids, together with their tendency to entrain water, determine the geometry of the settling or rising zone. Their concentration by volume and the contemplated length of storage establish the dimensions of the bottom zone and the scum zone. In wastewater treatment, both the settling zone and the bottom and scum zones must take the possible putrescence of liquid and solids into account if the liquid is not to become septic and if gas-lifted solids are to be kept out of the effluent and away from the tank surface. Putrescence and excessive accumulations are avoided by removing sludge more or less continuously.

Removal devices affect tank design as well as tank operation. Thermal convection currents and wind-induced currents are held in check by housing or covering the tanks. The proper number of units is a matter of wanted flexibility of operation and economy of design.

The use of flocculating agents may add as many as three ancillary functions to settling units by requiring (1) rapid distribution of the agent through the water to be treated, (2) adequate reaction time or time for floc growth, and (3) return of floc to the influent for the sake of promoting flocculation.

15.6 UPFLOW CLARIFICATION

Unless vertical flow is introduced for the specific purpose of separating large or heavy, fast-settling particles from small or light, slow-settling particles by differential sedimentation, vertical-flow basins normally combine flocculation or contact filtration (also called sludge-blanket filtration) with sedimentation. If differential sedimentation or solids classification is the primary purpose, the small or light, discrete particles are purposely carried over the effluent weir of the tank while the large or heavy particles settle to the tank bottom. This is done in some grit chambers, for example. If floc buildup and removal is the primary function, entering particles are not intended to remain discrete, but to conjoin into aggregates that are eventually withdrawn from the upflowing water.

For floc buildup, vertical-flow tanks (Fig. 15.13) are differentiated zonally. Inlet and sludge zone are in close contact, and the flocculation zone is occupied in part or as a whole by a cloud or blanket of flocs, not unlike a fluidized filter bed in its general nature. Rising flocs or particles come into contact with settling flocs or particles and with a stationary cloud of flocs or particles in equilibrium with their hydraulic environment. An outlet zone at the top of the tank allows for some upward and downward displacement of the flocculation zone. In this respect it forms a buffer between tank and outlet.

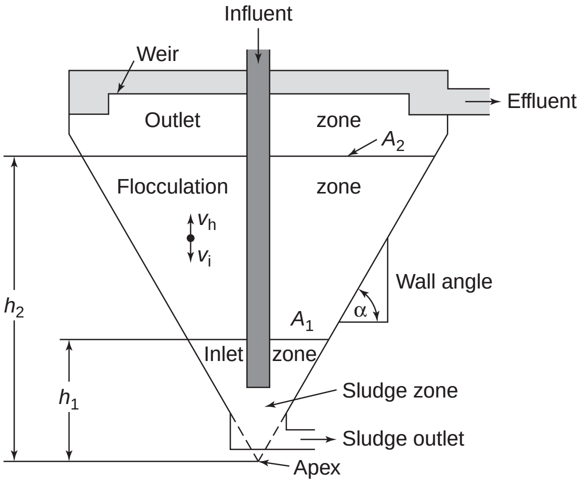


Figure 15.13 Vertical section through conical or pyramidal upflow tank (After Fair et al., 1971).

In sludge-blanket filtration, hydraulic operations are aimed at the (1) control of floc growth, (2) positioning of the flocculation zone or floc-blanket surface, and (3) regulation of the intensity of floc shear. Hydraulic control is exerted by proper dissipation of hydraulic power and adjustment of residence time in the flocculation or contact zone.

The power dissipated, P, is $\rho g h_f Q$, where ρ is the mass density of the fluid, g the gravity constant, h_f the head loss in passage through a zone of depth $h_2 - h_1$ (Fig. 15.13), and Q the rate of flow. The useful loss of head equals the weight in water of the suspended floc, or $h_f = [(\rho_s - \rho)/\rho](1 - f_e)(h_2 - h_1) = (s_s - 1)(1 - f_e)(h_2 - h_1)$, where ρ_s and s_s are, respectively, the mass density and specific gravity of the flocs, and f_e is the relative pore space of the flocculation zone. For a zonal volume or capacity V, cross-sectional area A, and wall angle α,

$$V = \int A\,\mathrm{d}h = 4\cot^2\alpha \int h^2\,\mathrm{d}h = 4/3\cot^2\alpha\left(h_2^3 - h_1^3\right)$$

for a square pyramidal tank

$$V = \int A\,\mathrm{d}h = \pi\cot^2\alpha \int h^2\,\mathrm{d}h = (\pi/3)\cot^2\alpha\left(h_2^3 - h_1^3\right)$$

for a conical tank

The detention time

$$t_d = f_e V/Q, \tag{15.21}$$

and

$$f_e = (v/v_s)^{1/5} = (v_h/v_s)^{1/4} \tag{15.22}$$

where f_e = relative pore space, dimensionless; v = face velocity, ft/s (m/s); v_s = settling velocity of particles, ft/s (m/s); v_h = hindered-settling velocity, ft/s (m/s); t_d = detention time, s; V = volume, ft^3 (m^3); Q = flow, ft^3/s (m^3/s).

If the values of v_s, v_h, v (being the settling, interstitial hindered settling and face velocities of the particles and fluid, respectively) are introduced into the relevant expressions for wanted and limiting velocity gradients, $G = \sqrt{P/\mu V}$ and the product of gradients and exposure time t_d is formed to identify contact opportunity,

$$G = [(g/\nu)(s_s - 1)(1 - f_e)(h_2 - h_1)/(V/Q)]^{1/2} \tag{15.23}$$

and, because $t_d = f_e V/Q$,

$$Gt_d = f_e[(g/\nu)(s_s - 1)(1 - f_e)(h_2 - h_1)(V/Q)]^{1/2} \tag{15.24}$$

where G = velocity gradient, s^{-1}; g = acceleration of gravity = 32.2 ft/s^2 = 9.81 m/s^2; ν = kinematic viscosity, ft^2/s (m^2/s); s_s = specific gravity of particles, dimensionless; f_e = relative pore space, dimensionless; $h_2 - h_1$ = zone depth between h_2 and h_1, ft (m); V = volume, ft^3 (m^3); Q = flow, ft^3/s (m^3/s); t_d = detention time, s.

The relationship between dynamic viscosity and kinematic viscosity is represented by

$$\nu = \mu/\rho \tag{15.25}$$

where ν = kinematic viscosity, ft^2/s (m^2/s); μ = dynamic viscosity, lb-s/ft^2 (N-s/m^2); ρ = mass density of fluid, lb-s^2/ft^4 (kg/m^3); and N = Newton force, kg-m/s^2.

If G and Gt_d are to be controlled, f_e, h, and V are the manageable variables and must be selected to suit wanted floc growth and clarification. To assure zonal stability by preventing floc rise and escape, the cross-sectional area (or width) of the tank may, as a practical matter, be enlarged in the direction of flow. At the same time, a useful initial G value should be imposed to promote floc building, while a destructive terminal G value is avoided to keep the floc formed from being broken up and swept over the effluent weir. For adequate contact opportunity, the period of residence should be long enough to accomplish wanted results.

Cross-sectional area is increased in the direction of flow by providing a wall angle of 45–65° with the horizontal (2 cot α = 2.00–0.93) to create a diameter of circular tanks or width of square tank as large as 2.00–0.93 times the distance from the apex. At a wall angle of 63°26′, incidentally, diameter D and width B equal the apical distance, that is, $D = B = h$.

EXAMPLE 15.6 DESIGN OF UPFLOW FLOCCULATION-SEDIMENTATION TANK

Given an upward flow of 0.5 ft^3/s (323,000 gpd) or 14.2 L/s (0.0142 m^3/s) in a square 45° pyramidal tank, with an effective flocculating zone between 2 ft (0.61 m) above the apex (1 ft or 0.30 m above the tank bottom) and 9 ft (2.74 m) above the apex, find

1. The average G and Gt values and the limiting G values (G_2 and G_1) at the top and bottom of the flocculating zone, and
2. The average settling velocities and diameters of the flocs when the average floc concentration is 40%, the temperature 50°F (10°C), and the average specific gravity of the floc s_s = 1.10. Assume that the settling is hindered and approximated by Stokes' law.

Solution 1 (US Customary System):

1. From Appendix 3,

$$\nu = 1.41 \times 10^{-5} \text{ ft}^2/\text{s at 50°F.}$$

Moreover,

$$g = 32.2 \text{ ft/s}^2$$

$$(s_s - 1) = 1.1 - 1 = 0.10 = 10^{-1}$$

$$(1 - f_e) = 0.40 = 4 \times 10^{-1} \text{ and } f_e = 0.60 = 6 \times 10^{-1}$$

$$Q = 0.5 \text{ ft}^3/\text{s}.$$

For the recurrent products in Eqs. (15.23) and (15.24)

$$[(g/\nu)(s_s - 1)(1 - f_e)(h_2 - h_1)]^{1/2}$$

$$= \{[32.2/(1.41 \times 10^{-5})]10^{-1}(4 \times 10^{-1})(9 - 2)\}^{1/2} = 8.00 \times 10^2$$

$$V = (4/3) \cot^2 \alpha \left(h_2^3 - h_1^3\right)$$

where cot 45° = 1

$$(V/Q)^{1/2} = [(4/3)(9^3 - 2^3)/0.5]^{1/2} = 43.9$$

$$t_d = f_e V/Q = 0.60(43.9)^2 = 1{,}160 \text{ s.}$$

Consequently from Eq. (15.23)

$$G = [(g/v)(s_s - 1)(1 - f_e)(h_2 - h_1)/(V/Q)]^{1/2}$$

$$G = 8.00 \times 10^2/43.9 = \mathbf{18.2\ s^{-1}}.$$

From Eq. (15.24)

$$Gt_d = f_e[(g/v)(s_s - 1)(1 - f_e)(h_2 - h_1)(V/Q)]^{1/2}$$

$$Gt_d = 6 \times 10^{-1} \times 8.00 \times 10^2 \times 43.9 = \mathbf{2.1 \times 10^4}.$$

For G in upper 1 ft, $h_2 - h_1 = 9 - 8 = 1$, and $h_2^3 - h_1^3 - = 9^3 - 8^3 = 729 - 512 = 217$ whence, in proportion to G,

$$G = [(1/7)/(217/721)]^{1/2} \times 18.2 = \mathbf{12.5\,s^{-1}}.$$

For G in lower 1 ft, $h_2 - h_1 = 3 - 2 = 1$, and $h_2^3 - h_1^3 - = 3^3 - 2^3 = 19$ whence again, in proportion to G,

$$G = [(1/7)/(19/721)]^{1/2} \times 18.2 = \mathbf{42.5\ s^{-1}}.$$

2. For the upper 1 ft,

The average value of h at the center of gravity = 8.63 ft.
The average area $A = 4 \times (8.63)^2 = 298\ \text{ft}^2$.
Hence the face velocity $v = (0.5/298)\ 30.5 = 5.12 \times 10^{-2}$ cm/s.
For $f_e = 0.6$

$$f_e = (v/v_s)^{1/5}$$

$$f_e^5 = 0.6^5 = 7.78 \times 10^{-2} = v/v_s, \text{ and}$$

$$v_s = 5.12 \times 10^{-2}/(7.78 \times 10^{-2}) = \mathbf{0.657\,cm/s}.$$

or from Fig. 15.4 $\boldsymbol{d} = \mathbf{4.0 \times 10^{-2}\ cm}$ by Stokes' law as an approx. to hindered settling.

For the lower 1 ft, the average value of $h = 1.63$ ft and the average area $A = 4(1.63)^2 = 10.6\ \text{ft}^2$
Hence

$$v = 1.44\,\text{cm/s and}$$

$$v_s = \mathbf{18.5\,cm/s}\ \text{or}$$

From Stokes' law

$$v_s = (g/18)[(s_s - 1)/\upsilon]d^2$$

$$18.5 = (981/18)(0.1/1.31 \times 10^{-2})d^2$$

$$18.5 = 4.16 \times 10^{-2}d^2$$

$\boldsymbol{d} = \mathbf{2.1 \times 10^{-1}\ cm}$ by Stokes' law as an approximation to hindered settling.

If the upflow tank serves as a flocculation as well as a settling unit, the magnitudes of G and Gt_d suggest (1) that the flocs should grow well at average values of $G = 18.2\ \text{s}^{-1}$ and $Gt_d = 2.1 \times 10^4$; (2) that the floc formed is not likely to be destroyed by shear in the upper foot of the tank, where $G = 42.5\ \text{s}^{-1}$; and (3) that, to settle into the lower foot, the floc must grow until it has a settling velocity of 18.5 cm/s or its diameter is 2.1×10^{-1} cm approximately.

The surface loading (SL) of the upflow tank

$$\begin{aligned} \text{SL} &= Q/A \\ &= 323{,}000\,\text{gpd}/(2 \times 9\,\text{ft})^2 \\ &= 997\,\text{gpd/ft}^2. \end{aligned}$$

That is not very far from the normal loading of settling units.

Solution 2 (SI System):

1. From Appendix 3,

$$\nu = 1.31 \times 10^{-6}\ \mathrm{m^2/s}\ \text{at}\ 10°\mathrm{C}.$$

Moreover,

$$g = 9.81\ \mathrm{m/s^2}$$
$$(s_s - 1) = 1.1 - 1 = 0.10 = 10^{-1}$$
$$(1 - f_e) = 0.40 = 4 \times 10^{-1}\ \text{and}\ f_e = 0.40 = 4 \times 10^{-1}$$
$$Q = 0.0142\ \mathrm{m^3/s}.$$

For the recurrent products in Eqs. (15.23) and (15.24)

$$[(g/\nu)(s_s - 1)(1 - f_e)(h_2 - h_1)]^{1/2} = \{[9.81/(1.31 \times 10^{-6})]10^{-1}(4 \times 10^{-1})(2.74 - 0.61)\}^{1/2}$$
$$= 8.00 \times 10^2$$
$$V = (4/3)\cot^2\alpha\left(h_2^3 - h_1^3\right)$$

where cot 45° = 1

$$(V/Q)^{1/2} = [(4/3)(2.74^3 - 0.61^3)/0.0142]^{1/2} = 43.9$$
$$t_d = f_e V/Q = 0.60\,(43.9)^2 = 1160\ \mathrm{s}.$$

Consequently from Eq. (15.23)

$$G = [(g/\nu)(s_s - 1)(1 - f_e)(h_2 - h_1)/(V/Q)]^{1/2}$$
$$G = 8.00 \times 10^2/43.9 = \mathbf{18.2\ s^{-1}}.$$

From Eq. (15.24)

$$Gt_d = f_e[(g/\nu)(s_s - 1)(1 - f_e)(h_2 - h_1)(V/Q)]^{1/2}$$
$$Gt_d = 6 \times 10^{-1} \times 8.00 \times 10^2 \times 43.9 = \mathbf{2.1 \times 10^4}.$$

For G in upper 0.30 m,

$$h_2 - h_1 = 2.74 - 2.44 = 0.30\ \mathrm{m},\ \text{and}\ h_2^3 - h_1^3 = 2.74^3 - 2.44^3 = 6.14$$

whence, in proportion to *G*,

$$G = [(18.2/\mathrm{s})/(1/7 \times 6.14)/20.42]^{1/2} = \mathbf{12.5\ s^{-1}}.$$

For *G* in lower 0.30 m,

$$h_2 - h_1 = 0.91 - 0.61 = 0.30,\ \text{and}\ h_2^3 - h_1^3 = 0.91^3 - 0.61^3 = 0.54$$

whence again, in proportion to *G*,

$$G = (18.2/\mathrm{s})[(1/7)/(0.54/20.42)]^{1/2} = \mathbf{42.5\ s^{-1}}.$$

2. For the upper 0.30 m,

The average value of *h* at the center of gravity = 2.63 m.

The average area $A = 4 \times (2.63)^2 = 27.68\ \mathrm{m}^2$.

Hence the face velocity $v = (0.0142\ \mathrm{m^3/s}\ /27.68) \times 100 = 5.12 \times 10^{-2}\ \mathrm{cm/s}$.

For $f_e = 0.6$,

$$f_e = (v/v_s)^{1/5}$$
$$f_e^5 = 7.78 \times 10^{-2} = v/v_s,\ \text{and}$$
$$v_s = 5.12 \times 10^{-2}/(7.78 \times 10^{-2}) = \mathbf{0.657\ cm/s}$$

or from Fig. 15.4 $\boldsymbol{d = 4.0 \times 10^{-2}}$ **cm** by Stokes' law as an approx. to hindered settling.

For the lower 0.30 m, the average value of $h = 0.50$ m and the average area $A = 4(0.50)^2 = 1$ m^2. Hence

$$v = 1.44\,\mathrm{cm/s}\ \text{and}$$

$$v_s = \mathbf{18.5\,cm/s}\ \text{or}$$

from Stokes' law

$$v_s = (g/18)[(s_s - 1)/\nu]d^2$$

$$18.5 = (981/18)(0.1/1.31 \times 10^{-2})d^2$$

$$18.5 = 4.16 \times 10^{-2} d^2.$$

$\boldsymbol{d = 2.1 \times 10^{-1}}$ **cm** by Stokes' law as an approximation to hindered settling

If the upflow tank serves as a flocculation as well as a settling unit, the magnitudes of G and Gt_d suggest (1) that the flocs should grow well at average values of $G = 18.2\ \mathrm{s}^{-1}$ and $Gt_d = 2.1 \times 10^4$; (2) that the floc formed is not likely to be destroyed by shear in the upper foot of the tank, where $G = 42.5\ \mathrm{s}^{-1}$; and (3) that, to settle into the lower foot, the floc must grow until it has a settling velocity of 18.5 cm/s or its diameter is 2.1×10^{-1} cm approximately.

The surface loading (SL) of the upflow tank

$$\mathrm{SL} = Q/A$$

$$= [(0.0142\,\mathrm{m^3/s}) \times 60 \times 60 \times 24\,\mathrm{m^3/d}]/(2 \times 2.74\,\mathrm{m})^2 = \mathbf{40.9\,m^3/d/m^2}.$$

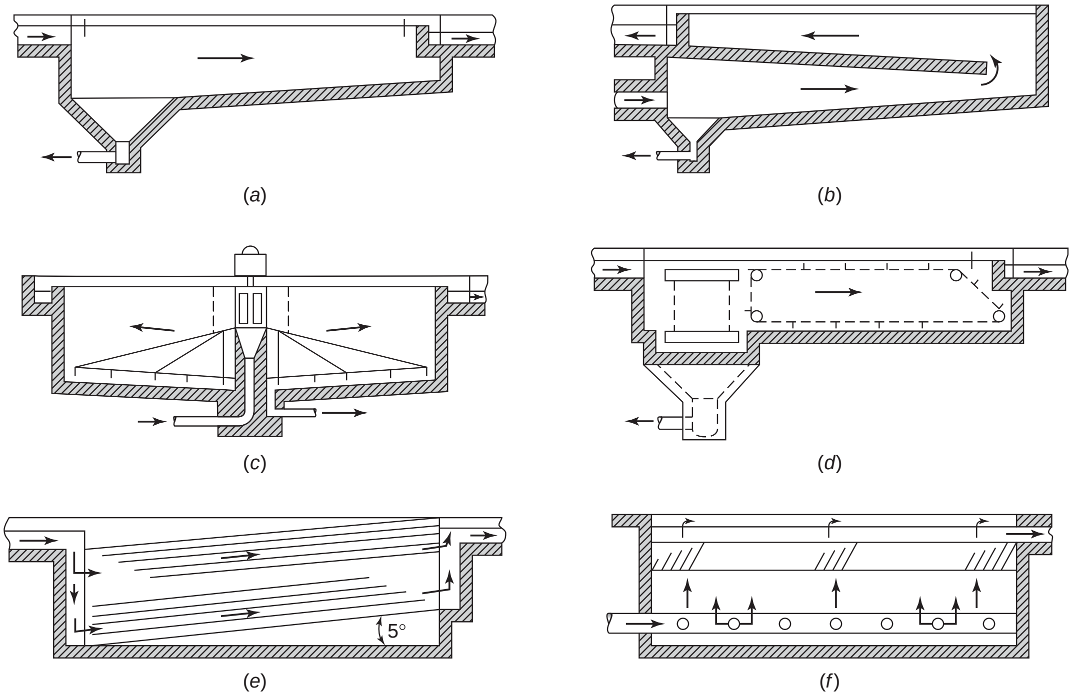
That is not far from the normal loading of settling units.

15.7 GENERAL DIMENSIONS OF SETTLING TANKS

Horizontal-flow tanks (Fig. 15.12) and upward-flow tanks (Fig. 15.14) have been constructed in great variety. Sketches of representative designs are shown in Figs. 15.15 and 15.16. Circular, square, or rectangular in plan, they vary in depth from 7 to 15 ft (2.13–4.57 m), 10 ft (3.05 m) being a preferred value. Circular tanks are as much as 200 ft (60 m) in diameter, but they are generally held to a 100 ft (30 m) maximum to reduce wind effects. Square tanks are generally smaller. A side length of 70 ft (21.34 m) is common. Rectangular tanks have reached lengths of almost 300 ft (90 m), but a 100 ft (30 m) limit is generally imposed on them, too. The width of mechanically cleaned, rectangular tanks is dictated in part by the available length of scrapers. This is 16 ft (4.88 m) for wooden scrapers, but scrapers can be operated in parallel in the same tank. A width of 30 ft (9 m) is not unusual. The diameter of mechanically cleaned, circular tanks is governed by the structural requirements of the trusses supporting the scrapers. Except for steep-sided sludge hoppers, the bottom of most settling tanks slopes gently. Common slopes lie close to 8% for circular or square tanks and 1% for rectangular tanks. Foothold on a slippery surface becomes precarious at a slope of $1^1/_2$ in. per ft (12.5%). The slopes of sludge hoppers range from 1.2:1 to 2:1 (vertical:horizontal). They should be steep enough for the solids to slide to the bottom.

Figure 15.14 Upflow settling basins (http://yosemite.epa.gov/R10/CLEANUP.NSF/9f3c21896330b4898825687b007a0f33/8572edc919ef3f31882569f50002f069/$FILE/P4140080.JPG).

In upflow clarification, incoming flows are forced to rise with reasonable uniformity either by flaring the tank outward (Fig. 15.16a) or by providing distributing grids (Fig. 15.16b). Grids may be quite like the underdrainage piping of filters and like them, also serve a dual purpose by acting as inflow structures during normal upflow, in this instance, and as sludge collectors during sludge removal. Overflow from structures of this kind may be to a circumferential or a central weir, to a perforated collector, or to a pipe grid that is the mirror image of the inlet arrangement and will assure a uniform rate of rise and concurrent tank loading. Designs such as those illustrated in Figs. 15.16c and 15.16d combine mechanical flocculation with upflow flocculation and sludge separation.



(a) (b) (c) (d) (e) (f)

Figure 15.15 Representative designs of horizontal and vertical displacement settling tanks. (a) Rectangular tank with longitudinal flow. Tank is thrown out of operation for cleaning. Solids are flushed to sump for removal from the dewatered tank. (b) Tank a, equipped with a single tray. (c) Circular tank with radial flow. Solids are plowed to central sump and withdrawn during operation. The rotary mechanism carries plows. (d) Tank a, mechanized. (e) Tank a, filled with tubular settlers at small angle (5°) to the horizontal. (f) Tank provided with pipe grid for distribution of entering flow and with large-angle (45–60°) tubes near surface of tank. Solids are either scraped or flushed to influent or effluent end, thence to sump to be withdrawn (After Fair et al., 1971).

15.8 SLUDGE REMOVAL

To be flushed clean, if not provided with mechanical sludge removal mechanisms, tanks must be cut out of service and unwatered. The accumulated solids are washed into a sump, whence they are withdrawn by gravity or pumping before the tank is refilled or by hydrostatic pressure after it has been refilled. Needed water is bled from neighboring tanks or from a pressure line. If the line transports water for general plant or municipal uses, backflow preventers must be installed to keep the water supply from being contaminated. This is a serious responsibility. Many water-purification tanks are cleaned by hand. The settled floc and solids are small in volume and often quite stable, even in warm weather.

In the treatment of municipal and industrial wastewaters, however, sludge deposits are usually so large in volume and generally so putrescible that they must be removed more or less continuously. Mechanical removal is usually warranted. It is often economical, too, in the treatment of silt-laden and softened waters.

Scrapers or plows are normally attached to rotating arms (Fig. 15.15c) or endless chains (Fig. 15.15d). At the same time surface scum can be collected with the scraper mechanism. Wide rectangular tanks may be equipped with cross-conveyors to concentrate sludge and scum withdrawal in one spot. To keep solids from escaping back into the cleaned liquid, the scrapers should preferably operate at velocities below 1 ft/min (0.3 m/min). Power requirements are about 1 hp for 10,000 ft^2 (0.80 kW for 1,000 m^2) of tank area, but straight-line collectors must be furnished with motors about 10 times that strong in order to master the starting load. Sludge pipes operating much like suction cleaners can take the place of plows when the sludge is feathery and light (Fig. 15.15c).

Sludge is withdrawn from vertical-flow tanks before the sludge blanket rises high enough to be carried over the effluent weir. Hydraulic and biochemical stability are important considerations. Sludge recirculation by pumping or by a system of baffles may promote flocculation (Fig. 15.16).

15.9 INLET HYDRAULICS

For high efficiency, inlets must distribute flow and suspended matter more or less equally into batteries of tanks and within individual tanks. For hydraulic equality, the dividing flow must encounter equal frictional resistances or be subjected to a controlling loss of head, that is, a head large in comparison with the frictional resistances between inlets or inlet openings. The water levels of parallel tanks are held at the same elevation by regulating the outflow. If suspended matter travels along the bottom of the influent conduit, hydraulic

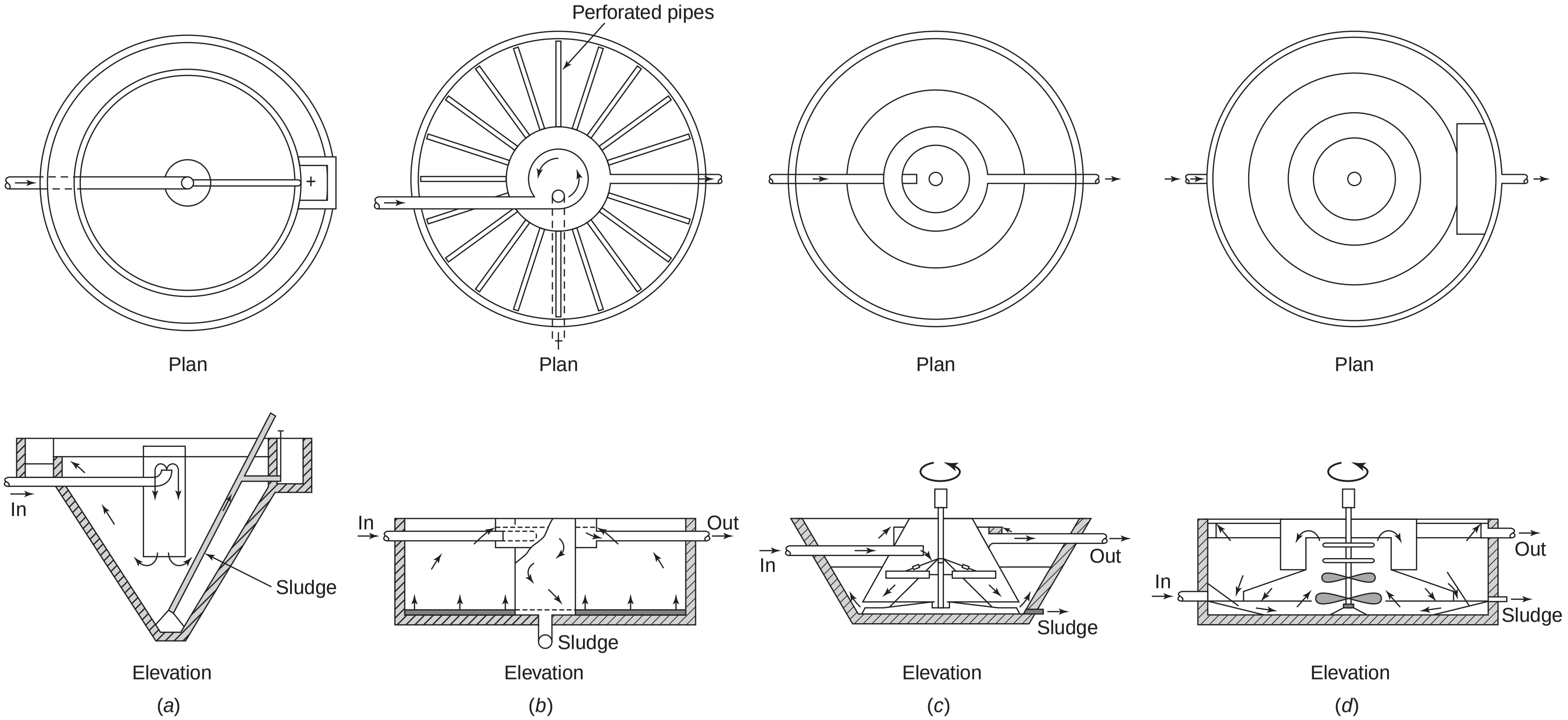


Figure 15.16 Representative design of upflow settling tanks. (a) Flaring circular or square tank. Sludge is removed during operation; in this case hydrostatically. (b) Circular tank with central mixing and flocculating chamber. Pipe grid distributes flow and intermittently collects sludge. (c) Circular, flaring tank with central mixing and flocculator chamber. (d) Circular tank with central mixing and flocculating chamber and built-in sludge recirculation. (After Fair et al., 1971).

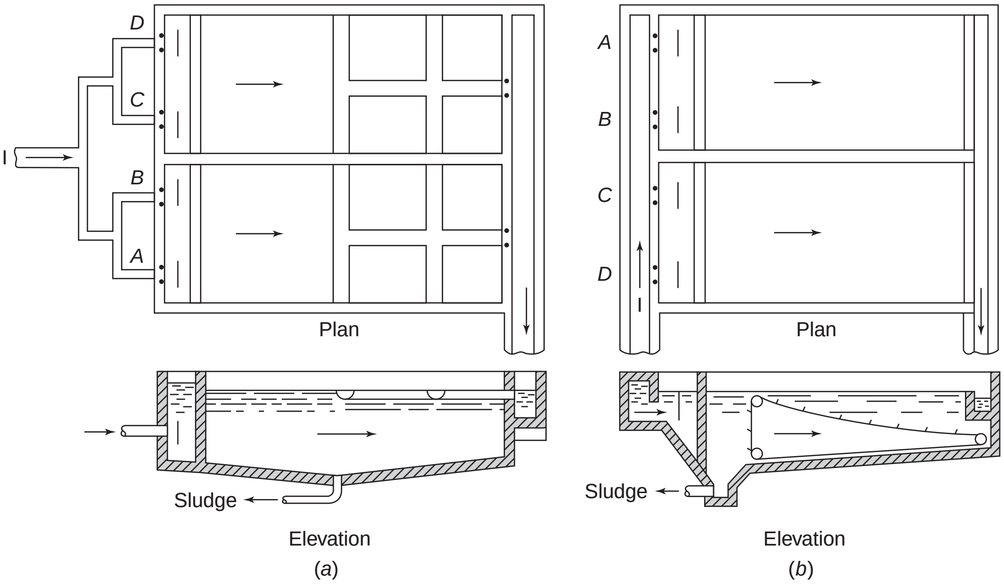


Figure 15.17 Inflow and outflow structures of settling tanks. (a) Uniformity of inflow is secured by equality of resistance. (b) Uniformity of inflow is secured by control of resistance (After Fair et al., 1971).

equality does not necessarily ensure loading equality. Necessary adjustments can then be made only by trial.

In reference to Fig. 15.17, the principles of flow regulation can be identified as follows. The flow originating at I in Fig. 15.17a traverses identical paths before it is discharged at A, B, C, and D. Consequently flow is evenly distributed between the tanks and within them except when the water-surface elevation of the two tanks is not the same. Equality is then confined to points in the individual tanks.

The flow originating at I in Fig. 15.17 is subdivided in such manner that the discharge q_n through any inlet orifice is held to mq_1, where $m < 1$ and q_1 is the discharge through the first orifice. The head on the nth orifice must then be

$$h_n = kq_n^2 = k(mq_1)^2 = m^2 h_1 \tag{15.26}$$

or, if h_f is the head lost between points of discharge (1 and n), $h_n = h_I - h_f = m^2 h_1$, and

$$h_1 = h_f/(1 - m^2) \tag{15.27}$$

The magnitude of h_f can be estimated from friction losses and velocity changes.

EXAMPLE 15.7 HEADLOSSES IN INLETS TO SEDIMENTATION TANKS TO OBTAIN EQUALITY IN FLOW DISTRIBUTION

The inlet farthest from the point of supply of a settling tank is to discharge 99% of the flow delivered by the nearest inlet. Find, in terms of the friction head h_f,

1. The required head loss through the nearest inlet and
2. The associated head loss through the farthest inlet

Solution:

1. The required head loss through the nearest inlet and from Eq. (15.27),

$$h_1 = h_f/(l - m^2)$$
$$h_1 = h_f/[1 - (0.99)^2]$$
$$= \mathbf{50.3}\,\boldsymbol{h_f}.$$

2. The associated head loss through the farthest inlet from Eq. (15.26),

$$h_n = m^2 h_1$$
$$h_n = (0.99)^2 h_1$$
$$= 0.980\,h_1$$
$$= \mathbf{49.3}\,\boldsymbol{h_f}.$$

Baffle boards in front of inlet openings will destroy much of the kinetic energy of the incoming water and assist in distributing the flow laterally and vertically over the basin. Training or dispersion walls perforated by holes or slots (Fig. 15.17) operate on the principle demonstrated in Eq. (15.20) by introducing a controlling head loss. Frictional resistance in advance of the openings is a function of the velocity head of the eddy currents. Baffles of this kind promote flow stability. However, by creating and destroying velocity, baffles can quickly build up head losses that are not offset by increased basin efficiency. Conduit and orifice velocities should be high enough to prevent deposition of solids, yet low enough to keep fragile floc from being destroyed when that is important.

Model analysis of inlet structures can provide rewarding guide lines.

15.10 OUTLET HYDRAULICS

Outflow is generally controlled by a weir attached to one or both sides of single- or multiple-outlet troughs. If weirs in adjacent tanks are placed at the same elevation and discharge freely over the same length, the loading of equal basins should stay within the limits of inflow variation. If effluent weirs are submerged, the degree of submergence will vary along the trough. Draw-off then becomes unequal and induces short-circuiting, unless a training wall similar to the inlet training wall again introduces a controlling loss of head.

Outlet troughs are lateral spillways. Required dimensions are given by the drawdown curve of the water surface in the trough in much the same way as for the washwater troughs of rapid filters.

Weir length relative to surface area determines the strength of outlet currents. By experience, this should lie below 30,000 gpd/ft (372,000 L/d/m) to avoid surges. To prevent nonuniform discharge caused at low rates of flow by wind, obstruction to flow by stranded solids, and slight variations in weir level, the weir may be subdivided into triangular notches. This will also ventilate the nappe of the jets issuing from the notches and avoid changes in rates of flow caused by departures from free flow. Adjustable weir plates can be leveled from the water level in the tank.

In wastewater treatment a scum board, or shallow baffle plate, paralleling outlet weirs, will keep floating solids, grease, and oil from stranding on the weirs or reaching the effluent. In ordinary circumstances, variations in flow are of little concern in the design and operation of settling tanks. Provided that the maximum design flow is not exceeded, the mass of water they contain is large enough to smooth out normal flow variations. In grit chambers, on the other hand, allowable variations fall within relatively narrow limits and call for the use of flow-control devices. Tubular inserts in advance of outlet weirs (Figs. 15.15e and 15.15f) may intercept and deposit solids that might otherwise escape.

15.11 SEDIMENTATION TANK LOADING, DETENTION, AND PERFORMANCE

Except when tanks receive suspensions composed of discretely settling particles of known size and density, it is advisable to base their design on the results of experimental settling-velocity analyses. Nevertheless, it is useful to know the order of magnitude of tank loadings for common types of suspensions, such as those shown in Table 15.2.

Departure from the ideal either decreases permissible loadings and increases requisite detention periods or lowers tank efficiencies. Hazen's theory of sedimentation will give a clue to attainable or required values. In general, coagulated and lime-softened waters, as well as municipal and industrial wastewaters subjected to plain sedimentation or coagulation in primary tanks, are provided detention periods of about 2 h or surface loadings of 900 gpd/ft^2 (43 m^3/d/m^2) in tanks 10 ft (3 m) deep above the sludge zone.

15.11.1 Sedimentation Tank Performance

Most settling basins incorporated in water-purification works include or follow coagulation or chemical precipitation. Tank performance depends in considerable measure on the effectiveness of these preparatory processes and is not a unique measure of settling efficiency. Only when turbid river waters are subjected to plain sedimentation is it common practice to report the percentage removal of suspended solids. Otherwise, interest centers on the removal of such items as color, turbidity, hardness, and iron.

Table 15.2 Common loadings and detention periods of settling tanks

Composition of suspension	Specific gravity	Size (cm)	Settling velocity at 10°C (cm/s)	Surface loading (gpd/ft^2)	Minimum detention period in 10 ft basin (h)
Alum and iron floc containing water	1.002	$\geq 10^{-1}$	8.3×10^{-2}	1,800	1.0
Activated sludge	1.005	10^{-1}	2×10^{-1}	1,200	1.5
Calcium precipitate	1.2	10^{-2}	—	900	2.0
Wastewater organics	≥ 1.001	10^{-1}	4.2×10^{-2}	900	2.0
Slit, clay	2.65	$\geq 10^{-3}$	6.9×10^{-3}	150	12

Conversion factors: 1 gpd/ft^2 = 0.0408 m^3/d/m^2, 1 ft = 0.3048 m.

The weight of suspended matter in turbid river waters may run into thousands of mg/L. Recorded efficiencies of removal by plain sedimentation vary widely for different rivers and different stretches of the same river. As little as 30% and as much as 75% of the suspended matter may settle out in 1 h, and as little as 50% and as much as 90% in 2 h. Very fine silt may not settle out even during many months of storage.

15.11.2 Regulations and Standards

Recommended standards for water works, popularly known as Ten State Standards are the most widely used standards for the design of sedimentation tanks. The most important provisions of these standards are listed below:

1. Plants designed for processing surface waters should provide duplicate units and permit operation of basins in series or parallel.
2. Presedimentation tanks should be designed for a minimum detention period of 3 h.
3. Settling tanks that are provided following flocculation and with conventional sedimentation shall have a minimum of 4 h of settling time. The detention time may be reduced to 2 h for lime-soda softening facilities treating groundwater. Reduced sedimentation time may also be approved when equivalent effective settling is demonstrated or when overflow rate is not more than 0.5 gpm/ft^2 (1.2 m/h).
4. Inlet devices: Inlets shall be designed to distribute the water equally and at uniform velocities. A baffle should be constructed across the basin close to the inlet end and should project several feet below the water surface.
5. Outlet devices: Outlet devices shall be designed to maintain velocities suitable for settling in the basin and to minimize short-circuiting. The use of submerged orifice is recommended.
6. Overflow rate over the outlet weir shall not exceed 20,000 gpd/ft (250 m^3/d/m). The entrance velocity through the submerged orifices shall not exceed 0.5 ft/s (0.15 m/s).
7. The velocity through settling tanks shall not exceed 0.5 ft/min (0.15 m/min).
8. Mechanical sludge collection equipment should be provided.

EXAMPLE 15.8 DESIGN OF SEDIMENTATION TANKS ACCORDING TO TEN STATE STANDARDS

Design sedimentation tanks for a 10 MGD (38 MLD = 0.44 m^3/s) water-purification plant. According to Ten State Standards the detention period should be at least 4 h.

Solution 1 (US Customary System):

Capacity of sedimentation tank required

$$V = Qt$$

$$V = 10 \times 10^6 \frac{\text{gal}}{\text{d}} \times \frac{\text{d}}{24\,\text{h}} \times 4\,\text{h} = 29{,}800\,\text{gal}$$

$$= 29{,}800\,\text{gal} \times 7.48\,\text{ft}^3/\text{gal} = 222{,}800\,\text{ft}^3.$$

Assume an effective depth of 12 ft.

Area of tank required = 222,800/12 = 18,600 ft^2.

Rectangular tank dimensions

Assume number of tanks to be provided = 4.

The width of the tank should be based on the availability of sludge collection equipment.

Assume the working width of units to be 40 ft.

$$\text{Length of each tank} = \frac{18,600}{4 \times 40} = 116\,\text{ft}.$$

Provide 4 tanks of dimensions 120 ft × 40 ft × 12 ft.

Check for overflow rate

$$\text{Area of each tank} = 120 \times 40 = 4{,}800\,\text{ft}^2$$

$$\text{Overflow rate} = \frac{2.5 \times 10^6}{4{,}800}$$

$$= 521\,\text{gpd}/\text{ft}^2 = 0.36\,\text{gpm}/\text{ft}^2 < 0.5\,\text{gpm}/\text{ft}^2, \text{ which is acceptable.}$$

Check for horizontal-flow velocity

$$\text{Velocity} = \frac{\text{length of tank}}{\text{detention time}}$$

$$\text{Velocity} = \frac{120\,\text{ft}}{4 \times 60\,\text{min.}} = 0.5\,\text{ft/min, which is acceptable.}$$

Weir Overflow rate

According to Ten State Standards the maximum weir overflow rate allowed is 20,000 gpd/ft.

So the weir length required for each tank = $\frac{2.5 \times 10^6}{20{,}000}$ = 125 ft.

Let *n* be the number of additional overflow weir troughs then,

$$(2n + 1) \times 40 = 125$$

$$n = 1.06.$$

Provide 2 additional troughs.

$$\text{Weir overflow rate} = \frac{2.5 \times 10^6}{(2 \times 2 + 1) \times 40} = 12{,}500\,\text{gpd/ft} < 20{,}000\,\text{gpd/ft, which is acceptable.}$$

Solution 2 (SI System):

Capacity of sedimentation tank required

$$\begin{aligned} V &= Qt \\ &= \left(0.44\,\text{m}^3/\text{s}\right)(4\,\text{h} \times 60\,\text{min/h} \times 60\,\text{s/min}) \\ &= 6{,}300\,\text{m}^3. \end{aligned}$$

Assume an effective depth of 3.5 m.

Area of tank required = 6,300/3.5 = 1,800 m^2.

Rectangular tank dimensions

Assume number of tanks to be provided = 4.

The width of the tank should be based on the availability of sludge collection equipment.

Assume the working width of units to be 12 m.

Length of tank = 1,800/(4 × 12) = 37.5 m.

Provide 4 tanks of dimensions 38 m × 12 m × 3.5 m.

Check for overflow rate

$$\begin{aligned} \text{Area of each tank} &= 38 \times 12 = 456\,\text{m}^2 \\ \text{Overflow rate} &= \frac{0.44 \times 60 \times 60}{4 \times 456} \\ &= 0.87\,\text{m/h} < 1.2\,\text{m/h, which is acceptable.} \end{aligned}$$

Check for horizontal-flow velocity

$$\text{Velocity} = \frac{\text{length of tank}}{\text{detention time}}$$

$$\text{Velocity} = \frac{38\,\text{m}}{4 \times 60\,\text{min.}} = 0.158\,\text{m/min, which is acceptable.}$$

Weir overflow rate

According to Ten State Standards the maximum weir overflow rate allowed is 250 m^3/d/m.

So the weir length required for each tank

$$= \frac{0.44 \times 60 \times 60 \times 24}{4 \times 250} = 38\,\text{m}.$$

Let *n* be the number of additional overflow weir troughs then,

$$\begin{aligned} (2n + 1) \times 12\,\text{m} &= 38\,\text{m} \\ n &= 1.08. \end{aligned}$$

Provide 2 additional troughs.

$$\begin{aligned} \text{Weir overflow rate} &= \frac{0.44 \times 60 \times 60 \times 24}{4 \times (2 \times 2 + 1) \times 12} \\ &= 158\,\text{m}^3/\text{d/m} < 250\,\text{m}^3/\text{d/m, which is acceptable.} \end{aligned}$$

15.12 SHALLOW DEPTH SETTLERS

It was realized early in the century by Hazen that the removal of settleable solids was independent of detention time and that it was a function of the overflow rate and the basin depth. He pointed out that the capacity of the sedimentation basin could be increased considerably by inserting horizontal trays in the basin. Camp proposed a design for a settling basin, which would have horizontal trays spaced at 6 in. (15 cm). The detention time for this basin was 10.8 minutes and its overflow rate over the trays was 667 gpd/ft^2 (27.2 m^3/d/m^2). Although shallow depth settling could have minimized the size and cost of water treatment facilities, the real interest in this concept was not aroused until recently. Two kinds of shallow depth settlers are now commercially available. These are the tube settlers and the lamella separators. Currently shallow depth settling is increasingly used in the upgrade of treatment plants, since it has the potential of reducing the size and cost of treatment facilities.

15.12.1 Theory of Shallow Depth Settling

Consider an ideal sedimentation basin for settling of discrete particles of uniform size, shape, and specific gravity. The particles will settle down with velocity v_s, as determined by Eq. (15.7). If velocity of flow through the settling basin of length L and depth d is v_1, time t_1 taken by a small parcel of water to pass through the basin is

$$t_1 = \frac{L}{v_1}$$

The time t_2 taken for the discrete suspended particles in that parcel of water to settle through depth d is

$$t_2 = \frac{d}{v_s}$$

To remove all the discrete particles before water leaves the settling basin t_1 should always be less than t_2. Maximum value of v_1 is obtained by equating t_1 to t_2, that is,

$$\frac{L}{v_1}(\text{max}) = \frac{d}{v_s}$$

or **(15.28)**

$$v_1\,(\text{max}) = \frac{Lv_s}{d}$$

Also $Q = Av_1$
where

Q = flow through the basin, m^3/s (ft^3/s)
A = the cross-sectional area of the basin, m^2 (ft^2)
v_1 = the horizontal velocity, m/s (ft/s)

or

$$Q = A\frac{Lv_s}{d} \tag{15.29}$$

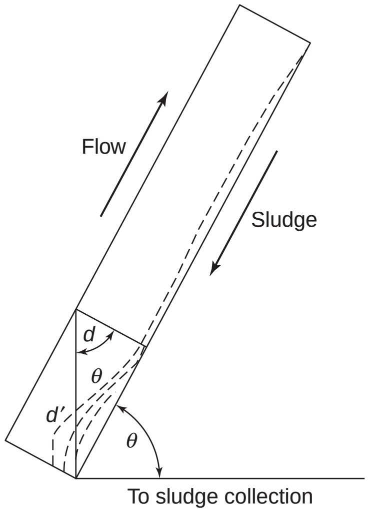


Figure 15.18 Flow pattern in an inclined tube settler.

Equation (15.29) indicates that for maximum flow through the basin of fixed area, the ratio L/d should be made as large as possible. This can be done by increasing the length of the basin and/or by decreasing the depth of the basin and making it as shallow as possible. The principle of shallow depth settling is employed in both tube settlers and Lamella separators.

In tube settlers each tube serves as an individual settling basin. For a horizontal tube of size 5 cm × 5 cm and 60 cm long, L/d ratio is 12, which is not too difficult to obtain in any conventional settling basin. The major advantage of tube settlers is that the tubes are stacked one above the other in a module providing a system with a number of small settling basins occupying the same space. Surface overflow rate through each tube calculated as flow through the tube divided by the surface area of the tube is actually less than that allowed for conventional settling tanks but since the total surface area exposed to flow in tube settlers is many times more than in the conventional settling tanks, the net result is considerable increase in the capacity of the settling basin.

When suspended particles settle in a horizontal tube the maximum vertical distance traveled by the particles is equal to d, the depth of the tubes. If the tubes are inclined at an angle θ, as shown in Fig. 15.18 the particles fall through a distance d, such that

$$v_1 = \frac{v_s L\cos\theta}{d} \tag{15.30}$$

where

d = vertical distance the particle fall through, ft (m)
θ = inclination angle of tubes

The above equation is limited to angles between 0° and $\tan^{-1} L/d$.

Laminar flow through the tubes is maintained even at high flow rates by the increased drag force due to a relatively large surface area of the tubes.

15.12.2 Tube Settlers

Tube settlers consist of numerous open-ended tubes, usually plastic about 2 in. (50 mm) in diameter and 24 in. (600 mm) long, mounted in modules and placed in a basin. The water that is carrying the suspended solids moves from an influent well, upward through the small tubes and into an outlet device. The tube cross-section may be circular, square, rectangular, hexagonal, or any other suitable shape. The tube settlers may be of either slightly inclined or steeply inclined design. In a slightly inclined tube settler system the tubes are inclined from the horizontal at an angle of about 5°. Sludge deposits form in the tubes, which need to be backwashed or drained. This system is best suited for use with filters so that the tubes can be backwashed along with the filters without any need for extra water or extra cost. This system of slightly inclined design is used primarily at small plants having a capacity of 1 MGD (3.8 MLD) or less.

In steeply inclined tube settlers the tubes are inclined at an angle greater than 45°, usually at 60°. It is observed that tube efficiency at 60° is comparable to that obtained at 5°. The sediment does not accumulate in these tubes but continues to settle downward and out of the tubes into the lower influent zone, forming an adsorptive sludge blanket. No backwashing is required in this system for sludge removal from the inside of the tubes. It is necessary, however, as in all sedimentation basins that means be provided to remove the sludge from the basin.

If tube settler modules were placed in existing basins, the sludge collection method would depend on the original basin and equipment design. At new installations, mechanical cleaning equipment should be provided. Many times the tubes become dirty and unsightly due to some of the suspended solids clinging to the walls or due to biological growth on the wall surface. This may seriously affect the efficiency of the tube settlers. Tube cleaning can be accomplished by providing a grid of diffused air headers beneath the tubes. The influent flow is stopped before turning on the air, which scrubs out attached floc, and then a 15–25 min. quiescent period is allowed before influent is again released into the tube settlers.

Table 15.3 Recommended overflow rates for tube settlers in horizontal-flow tanks

Desired effluent turbidity (JTU or NTU)	Basin overflow rate, L/min/m^2 (gpm/ft^2)	Tube settler overflow rate, L/min/m^2 (gpm/ft^2)
Water temperature 4.4°C		
1–3	———-	————
1–5	80 (2.0)	100 (2.5)
3–7	80 (2.0)	120 (3.0)
5–10	120 (3.0)	160 (4.0)
Water temperature 10°C		
1–3	80 (2.0)	100 (2.5)
1–5	80 (2.0)	120 (3.0)
3–7	80 (2.0)	160 (4.0)
5–10	———-	————

The tube settlers are designed on the basis of overflow rates calculated for the horizontal area of sedimentation basins covered by tube settlers. The overflow rates for tube settlers at water-purification plants depend on the raw water turbidity and temperature. Typically, for a horizontal-flow basin with overflow rate of 2 gpm/ft^2 (82 L/min/m^2) raw water with turbidity of less than 100 JTU and 4.4°C temperature, the recommended overflow rates for tube settlers are 2.5 gpm/ft^2 (102 L/min/m^2) for effluent turbidity of 1–5 JTU. Table 15.3 lists some manufacturers' recommended design parameters for tube settlers in a horizontal-flow basin.

According to the Recommended Standards for Water Works (2007 edition), a maximum rate of 2 gpm/ft^2 of cross-sectional area (80 L/min/m^2 = 1.2 m/h) for tube settlers, shall be used, unless higher rates are successfully shown through pilot plant or in-plant demonstration studies.

EXAMPLE 15.9 DESIGN OF A TUBE SETTLER

Design a tube settler for increasing the capacity of an existing settling tank provided for a water treatment plant (WTP) from 3 to 7.5 MGD (0.1314–0.3285 m^3/s). The existing tank dimensions are 30 ft × 130 ft × 12 ft deep (9.14 m × 39.62 m × 3.66 m). Raw water has a turbidity of 20–25 JTU and temperature of up to 40°F (4.4°C). Effluent turbidity desired is 1–5 JTU.

Solution 1 (US Customary System):

The existing tank overflow rate (OR) = $Q/A = (3 \times 10^6)/(30 \times 130) = 770\ \text{gpd/ft}^{2.}$

$$\text{The existing tank overflow rate at increased capacity (OR)} = (7.5 \times 10^6)/(30 \times 130) = 1{,}923\ \text{gpd/ft}^2$$
$$= 1.34\ \text{gpm/ft}^2.$$

Table 15.3 does not give tube settler rate for 1.34 gpm/ft^2 basin. But for a basin rate of 2 gpm/ft^2 and effluent turbidity 1–5 JTU the tube settler rate should be 2.5 gpm/ft^2.

At **2.5 gpm/ft^2 design overflow rate**, the tube settler area required is

$$\begin{aligned} A &= \frac{7.5 \times 10^6}{2.5 \times 1{,}440} \\ &= 2{,}083\ \text{ft}^2. \end{aligned}$$

Basin length, which should be covered by tube settlers, is

$$L = \frac{2{,}083}{30} = 69.4\ \text{ft}$$

Use a length of 70 ft.
This length is rounded off according to standard module dimensions to allow exact number of modules.

Solution 2 (SI System):
The existing tank overflow rate

$$\begin{aligned} \text{OR} &= Q/A = (0.1314\ \text{m}^3/\text{s})/(9.14 \times 39.62\ \text{m}^2) \\ &= 3.63 \times 10^{-4}\ \text{m/s} = 1.31\ \text{m/h} > 1.2\ \text{m/h}. \end{aligned}$$

The existing tank overflow rate at increased capacity

$$\begin{aligned} \text{OR} &= (0.3285\ \text{m}^3/\text{s})/(9.14 \times 39.62\ \text{m}^2) \\ &= 9.07 \times 10^{-4}\ \text{m/s} = 54.3\ \text{L/min/m}^2 = 3.26\ \text{m/h} \gg 1.2\ \text{m/h}. \end{aligned}$$

Table 15.3 does not show commercially available tube settler for the rate for 54.3 L/min/m^2 basin. The next available tube settler chosen is for 80 L/min/m^2 basin overflow rate. The anticipated effluent quality will be 1–5 JTU (or NTU) at water temperature of 4.4°C.

At the **100 L/min/m^2 design overflow rate** for the tube settler (Table 15.3), the tube settler area required is

$$\begin{aligned} A &= (0.3285\ \text{m}^3/\text{s})\,(60\ \text{s/min})\left(1{,}000\ \text{L/m}^3\right)/(100\ \text{L/min/m}^2) \\ &= 197.1\ \text{m}^2. \end{aligned}$$

Basin length, which should be covered by tube settlers, is

$$\begin{aligned} L &= (197.1\ \text{m}^2)/9.14\ \text{m} \\ &= 21.6\ \text{m},\ \textbf{use 22 m length}. \end{aligned}$$

This length is rounded off according to standard module dimensions to allow exact number of modules.

The modules are placed for a length of 70 ft (21.34 m) from the effluent and extending inward. In order to direct the flow through the module a baffle wall is installed at the inner edge. New effluent launders may also be required to provide uniform flow through the modules. As the basin is quite deep, 4 ft (1.22 m) long tubes may be installed.

15.12.3 Lamella Separator

The Lamella separator consists of a nest of parallel inclined plates and return tubes (Fig. 15.19). The plates are 1.5 m wide and 2.5 m long spaced 25–55 mm apart and are inclined at an angle of 25–45° from the horizontal. The main difference between the Lamella separator and tube settlers is in the way the influent travels with reference to the solids. In Lamella separators the flow of both influent and solids is concurrent whereas in tube settlers it is countercurrent. The influent in a Lamella separator is introduced at the top of the module. It travels downward with the solids, which settle to the bottom of each plate and are carried by the flow down the incline into the sludge hopper.

A return tube placed at the bottom of each plate carries the effluent back to the top of the unit and into an effluent launder. It is reported that at a flow rate of 470 gpd/ft^2 (19 m^3/d/m^2) of projected area solids removal of over 85% was obtained. It was also pointed out that a conventional sedimentation basin for the same flow at an overflow rate of 800 gpd/ft^2 (32.6 m^3/d/m^2) would require 10 times the space occupied by the Lamella separator.

15.13 GRAVITY THICKENING OF SLUDGE

Gravity thickeners are very similar in shape, design, and operation to the sedimentation clarifiers. The sludge thickening function is the major design consideration and thickening

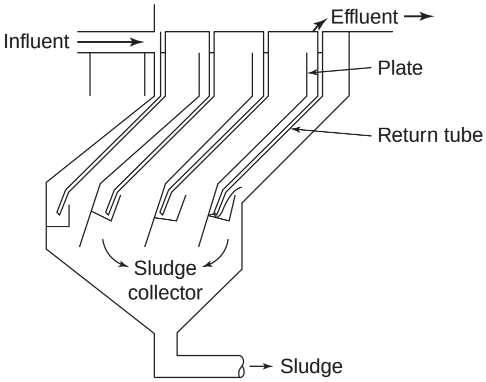


Figure 15.19 Lamella separator (After US EPA, 1959).

tanks are generally deeper than sedimentation clarifiers to provide greater sludge thickening capacity. Although gravity thickeners and sedimentation clarifiers look alike physically, the operation, theories, applications, and sizing design criteria are different. Sedimentation clarifiers are used for settling of particles by gravity from water. And the clarified effluent (i.e., supernatant) is the product. Sludge thickeners are used for concentration of the sludge by gravity and the concentrated sludge is the product. A well-designed, well-operated gravity thickeners should be able to, ideally, triple the solids content of the sludge, thereby eliminating two-thirds of its water volume. The theories and principles of sludge thickening can be found in Sections 15.4.3 and 15.4.4. A design example is illustrated below.

EXAMPLE 15.10 DESIGN OF SLUDGE THICKENING TANK BASED ON SETTLING/THICKENING COLUMN ANALYSIS

In a settling/thickening cylinder 80 cm (2.62 ft) high, the settling/thickening curve shown in Fig. 15.20 was obtained for alum sludge with an initial solids concentration C_0 of 2,500 mg/L. Determine the sludge thickener area to yield a thickened sludge concentration C_u of 12,500 mg/L with a sludge inflow of 0.1 MGD (0.00438 m^3/s). Also determine the solids loading rate lb/d/ft^2 (kg/d/m^2) and the overflow rate gpd/ft^2 (L/d/m^2 or m/h).

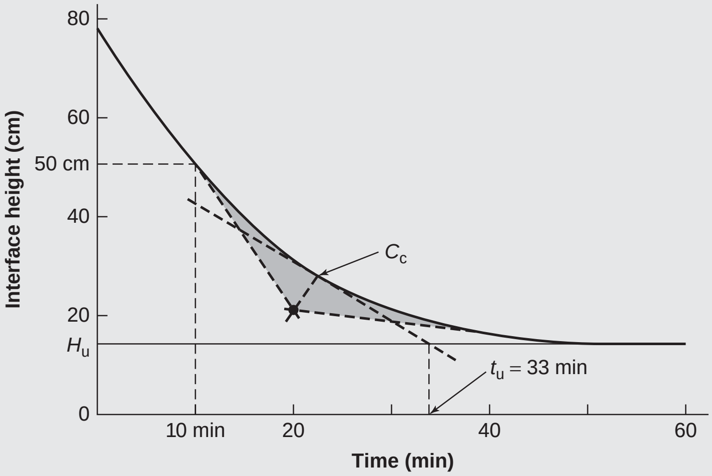


Figure 15.20 Settling curve for Example 15.11.

Solution 1 (US Customary System):

Based on the material balance, as per Eq. (15.13):

$$\begin{aligned} H_u &= c_0 H_0 / c_u \\ &= (2{,}500\,\text{mg/L})\,(80\,\text{cm})\,/12{,}500\,\text{mg/L}) \\ &= 16\,\text{cm}. \end{aligned}$$

In Fig. 15.20, a horizontal line is drawn at $H_u = 16$ cm. A tangent is then drawn to the settling/thickening curve at c_e, the midpoint of the region between hindered settling and thickening/compression. Bisecting the angle formed where the two lines meet determines point c_c. The intersection of the tangent at c_c and the line $H_u = 16$ cm determines t_u. Thus $t_u = 33$ min and the required area can be determined from Eq. (15.12):

$$\begin{aligned} A &= Qt_u / H_0 \\ &= (0.1\,\text{MGD})\left(1.547\,\text{ft}^3/\text{s/MGD}\right)(33\,\text{min} \times 60\,\text{s/min})\,/\,(2.62\,\text{ft}) \\ &= \mathbf{116.7\,ft^2\ area\ required\ for\ thickening}. \end{aligned}$$

This area is theoretically adequate for clarification to occur in the discrete-settling region; therefore, determine the subsidence velocity v_s from the hindered-settling portion of the curve. Settling at 10 min, the interface height is 50 cm; therefore,

$$\begin{aligned} v_s &= \text{distance}/\text{time} \\ &= (80 - 50)\ \text{cm}/\ (10\,\text{min} \times 60\,\text{s}/\text{min}) \\ &= 0.0\,5\ \text{cm}\ \ \text{s} \\ &= 16.4 \times 10^{-4}\ \text{ft}/\text{s subsidence velocity.} \end{aligned}$$

The overflow rate is proportional to the volume above the settled sludge zone:

$$\begin{aligned} Q &= (0.1\,\text{MGD})\,(1.547\,\text{ft}^3/\text{s}/\text{MGD})\,\left[(80 - 16)\,/80\right] \\ &= 0.124\,\text{ft}^3/\text{s}. \end{aligned}$$

The influent sludge flow is 0.1547 ft^3/s while the effluent sludge flow is 0.124 ft^3/s. The remaining portion (0.1547 – 0.124 = 0.0307 ft^3/s) is thickened and compacted. Therefore

$$\begin{aligned} A &= Q/v_s \\ &= (0.124\,\text{ft}^3/\text{s})/(16.4 \times 10^{-4}\ \text{ft}/\text{s}) \\ &= 75.6\,\text{ft}^2\ \text{arearequired for clarification.} \end{aligned}$$

Thus the controlling requirement is the thickening area (116.7 ft^2) compared to 75.6 ft^2 for clarification.

The solids loading and overflow rate are calculated as follows:

lb solids/d = (0.1 MGD)(2,500 mg/L)(8.34 lb/MG/1 mg/L) = 2,085 lb/d;

Solids loading = (2,085 lb/d)/(116.7 ft^2) = **18 lb/d/ft^2**.

Effluent sludge flow = 0.124 ft^3/s = 0.124 × 7.48 × 60 × 60 × 24 = 80,138 gpd;

Overflow rate = sludge flow/area = 80,138/116.7 = **687 gpd/ft^2**.

Solution 2 (SI System):

The following are explained in Solution 1:

H_u = 16 cm = 0.16 m (Fig. 15.20);
t_u = 33 min (Fig. 15.20);
H_0 = 80 cm = 0.80 m (Fig. 15.20);
Q = sludge flow = 0.00438 m^3/s;
c_0 = 2,500 mg/L;
c_u = 12,500 mg/L.

Area required for sludge thickening

$$\begin{aligned} \mathbf{A} &= Qt_u/H_0 \\ &= (0.00\,438\,\text{m}^3/\text{s})\,(33\,\text{min} \times 60\,\text{s}/\text{min})\,/0.80\,\text{m} \\ &= \mathbf{10.84\,m^2}. \end{aligned}$$

The discrete-settling region ends at 10 min when the interface height is 50 cm (Fig. 15.20):

$$\begin{aligned} v_s &= (80 - 50)\ \text{cm}/\ (10\,\text{min} \times 60\,\text{s}/\text{min}) \\ &= 0.0\,5\,\text{cm}/\text{s} = 5 \times 10^{-4}\ \text{m}/\text{s}. \end{aligned}$$

The overflow rate is proportional to the volume above the settled sludge volume.
Influent sludge flow = 0.00438 m^3/s.
Effluent sludge flow = 0.00438[(80 − 60)/80] = 0.0035 m^3/s.
The remaining sludge is thickened and compressed.
Area required for clarification = Q/v_s = (0.0035 m^3/s)/(5 × 10^{-4} m/s) = 7 m^2.
The controlling requirement is the thickening area 10.84 m^2, which exceeds the area required for clarification that is 7 m^2.
Influent sludge flow = 0.00438 m^3/s = 4.38 L/s.
Influent sludge concentration = c_0 = 2,500 mg/L = 2.5 × 10^{-3} kg/L.
Solids loading = Qc_0/A = (4.38 L/s)(2.5 × 10^{-3} kg/L)/(10.84 m^2) = 1.01 × 10 $^{-3}$ kg/s/m^2 = **87 kg/d/m^2**.
Overflow rate = Q/A = (0.0035 m^3/s)/(10.84 m^2) = 3.23 × 10^{-4} m^3/s/m^2 = **28 m^3/d/m^2**.

15.14 NATURAL FLOTATION

In natural flotation, oil, grease, or other substances lighter than water are allowed to rise naturally to the water surface of quiescent tanks, where they are skimmed off in ways analogous to the removal of sludge from settling tanks. In air flotation, particles heavier than water are lifted to the surface with the help of air and flotation reagents not unlike coagulation reagents in sedimentation. Flotation operations of this kind are borrowed largely from the metallurgical industry and are employed in the treatment of water, wastewater, and industrial wastes.

The natural gravitational rising of discrete particles is the obverse of natural gravitational settling. Stokes' law applies without change in formulation. As shown in Example 15.12 the Stokes velocity acquires a negative sign.

EXAMPLE 15.11 DETERMINATION OF RISING VELOCITY

Find the rising velocity in water at 20°C (68°F) of a spherical particle with a specific gravity of 0.80 and a diameter of 5×10^{-3} cm (16.4×10^{-5} ft).

Solution 1 (US Customary System):
By Eq. (15.10b),

$$v_s = (g/18)\,[(s_s - 1)/\nu]d^2.$$

From Appendix 3,

$$\nu = 1.08 \times 10^{-5}\ \text{ft}^2/\text{s}$$

$$v_s = (32.2/18)(0.80 - 1)\left(16.4 \times 10^{-5}\right)^2 / \left(1.08 \times 10^{-5}\right)$$

$$= \mathbf{-8.91 \times 10^{-4}\ ft/s}.$$

Moreover,

$$R = v_s d\rho/\mu = v_s d/\nu$$

$$= 8.91 \times 10^{-4} \times 16.4 \times 10^{-5} / \left(1.08 \times 10^{-5}\right)$$

$$= \mathbf{1.35 \times 10^{-2},\ and\ Stokes'\ law\ applies}.$$

Solution 2 (SI System):
By Eq. (15.10b),

$$v_s = (g/18)\,[(s_s - 1)/\nu]d^2.$$

From Appendix 3,

$$\nu = 1.01 \times 10^{-6}\ \text{m}^2/\text{s} = 1.01 \times 10^{-2}\ \text{cm}^2/\text{s}$$

$$v_s = (9.81/18)(0.80 - 1)\left(5 \times 10^{-5}\right)^2 / \left(1.01 \times 10^{-6}\right)$$

$$= \mathbf{-2.75 \times 10^{-4}\ m/s}.$$

Moreover,

$$R = v_s d\rho/\mu = v_s d/\nu$$

$$= 2.75 \times 10^{-4} \times 5 \times 10^{-5}/(1.01 \times 10^{-6})$$

$$= \mathbf{1.39 \times 10^{-2},\ and\ Stokes'\ law\ applies}.$$

15.15 DISSOLVED AIR FLOTATION PROCESS

15.15.1 Process Description

Dissolved air flotation (DAF) is used mainly to remove suspended and colloidal solids by flotation (rising) by decreasing their apparent density. The influent feed liquid can be raw water, wastewater, or liquid sludge. The flotation system consists of four major components: air supply, pressurizing pump, retention tank, and flotation chamber. According to *Henry's law*, the solubility of gas (such as air) in an aqueous solution increases with increasing the pressure. The influent feed stream can be saturated at several times atmospheric pressure (25–90 psig or 172–620 kPa gauge) by a pressurizing pump. The pressurized feed stream is held at this high pressure for about 0.5–3.0 min in a *retention tank* (i.e., a *pressure vessel*) designed to provide sufficient time for dissolution of air into the stream to be treated. From the retention tank, the stream is released back to atmospheric pressure in the *flotation chamber*. Most of the pressure drop occurs after a pressure-reducing valve and in the transfer line between the retention tank and flotation chamber so that the turbulent effects of the depressurization can be minimized. The sudden reduction in pressure in the flotation chamber results in the release of microscopic air bubbles (average diameter 80 μm or smaller), which attach themselves to suspended or colloidal particles in the process water in the flotation chamber. This results in agglomerations, which, owing to the entrained air, give a net combined specific gravity less than that of water, or cause the flotation phenomenon. The vertical rising rate of air bubbles ranges between 0.5 and 2.0 ft/min (0.15 and 0.61 m/min). The floated materials rise to the surface of the flotation chamber to form a floated layer. Specially designed *flight scrapers* or other *skimming devices* continuously remove the floated material. The surface sludge layer can in certain cases attain a thickness of many inches and can be relatively stable for a short period. The layer thickens with time, but undue delays in removal will cause a release of particulates back to the liquid. Clarified water (effluent) is usually drawn off from the bottom of the flotation chamber and either recovered for reuse or discharged. Figures 15.21a–15.21c illustrate three DAF systems.

The retention time in the flotation chambers is usually about 3–60 min depending on the characteristics of process water and the performance of the flotation unit. The process effectiveness depends on the attachment of air bubbles to the particles to be removed from the process water. The attraction between the air bubbles and particles is primarily a result of the particle surface charges and bubble-size distribution. The more uniform the distribution of water and microbubbles, the shallower the flotation unit can be. Generally, the depth of effective flotation units is between 1 and 9 ft (0.30 and 2.74 m).

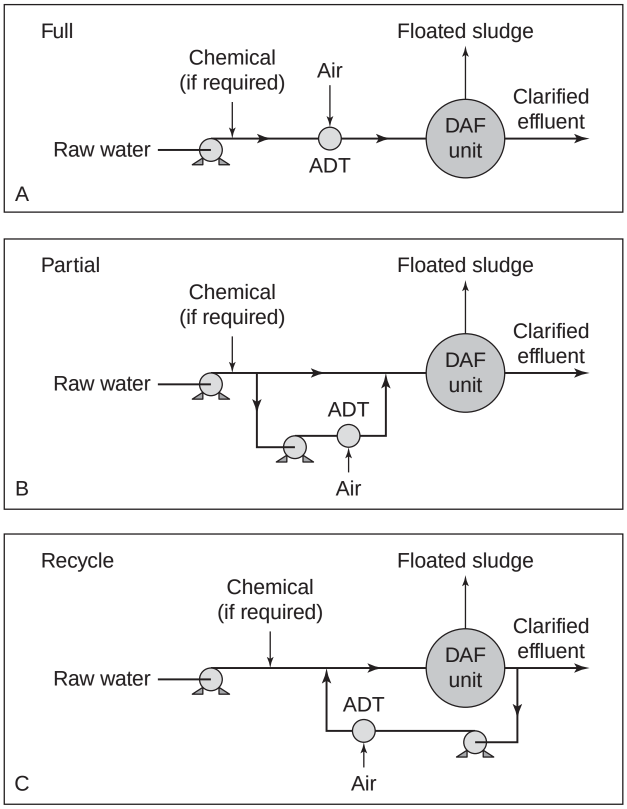


Figure 15.21 Operational modes of dissolved air flotation.

15.15.2 Process Configurations

The three common flotation system configurations are (a) full-flow pressurization, (b) partial flow pressurization without effluent recycle, and (c) recycle flow pressurization, which are graphically illustrated in Figs. 15.21a–15.21c, respectively.

In the *full-flow pressurization system* (Fig. 15.21a), the entire influent feed stream is pressurized by a pressurizing pump and held in the retention tank. The system is usually applicable to the feed stream with suspended solids exceeding 800 mg/L in concentration, and not susceptible to the shearing effects caused by the pressurizing pump and the high pressure drop at the pressure release valve. It is occasionally used for separating some discrete fibers and particles, which require a high volume of air bubbles. It is particularly feasible for solid–water separation where suspended solids will flocculate rapidly with the addition of chemical coagulants in the inlet compartment in the presence of the released air. The air bubbles may become entrapped within the floc particles resulting in a strong air to solids bond, thus in a highly efficient separation process.

In the *partial flow pressurization without effluent recycle system* (Fig. 15.21b), only about 30–50% of the influent feed stream is pressurized by a high-pressure pump and held in the retention tank. The remaining portion of influent stream is fed by gravity or a low-pressure pump to the inlet compartment of the flotation chamber where it mixes with the pressurized portion of the influent stream. Materials with low specific gravity can be removed with the partial flow pressurization system. This system is not recommended for use when the suspended solids are susceptible to the shearing effects of the pressurizing pump and the high pressure drop at the pressure release valve. It is generally employed in applications where the suspended solids concentrations are low, resulting in lower air requirement, and, in turn, lower operation and maintenance costs.

In tide *recycle flow pressurization system* (Fig. 15.21c), a portion (15–50%) of the clarified effluent from the flotation chamber is recycled, pressurized, and semisaturated with air in the retention tank. The recycled flow is mixed with the unpressurized main influent stream just before admission to the flotation chamber, with the result that the air bubbles come out of the aqueous phase in contact with suspended particulate matter at the inlet compartment of the flotation chamber. The system is usually employed in applications where preliminary chemical addition and flocculation are necessary ahead of flotation. It eliminates the problems with shearing the floc particles as only clarified effluent passes through the pressurizing pump and the pressure release valve. It should be noted, however, that the increased hydraulic flow on the flotation chamber due to the flow recirculation must be taken into account in the flotation chamber design.

While all the aforementioned three system configurations can be used for sludge (or fiber) separation, only the recycle flow pressurization system and partial flow pressurization system are recommended for water-purification or wastewater treatment.

15.15.3 Factors Affecting Dissolved Air Flotation

There are many factors affecting a DAF system:

1. *Nature of the particles:*

   The specific gravity is a characteristic of the particle or liquid to be abated or separated. It can easily be accepted that sand, for example, cannot be floated while voluminous material, such as activated sludge, or a water immiscible liquid, such as oil, can be floated. Surface-active substances can be separated by foam separation. Bubbles can strip off volatile substances.

2. *Size of particles:*

   Minutely small particles, particularly of the granular high-specific-gravity nature, cannot be floated. Generally, floatability increases with the size of the particle. In many cases, the size of particles can be increased by flocculation with various chemical coagulants.

3. *Dispersing agents:*

   Certain wastewaters and liquids contain unusual concentrations of various chemicals, resulting in specific flotation problems or advantages. Surfactants, such as detergents, tend to alter the physical properties of the sludge particle surface to be floated. The quantity and type of surfactant present in the influent may cause a variation (either positive or negative) in flotation results.

4. *Composition and nature of the influent:*

   The composition and nature of the influent is most important. Equalization of composition and flow improves the performance of the flotation unit.

5. *Liquid currents:*

   The liquid currents are governed by the physical design and hydraulics of the flotation unit. This becomes a consideration in the design of the tank and hydraulic loadings of the flotation unit.

6. *Air to solids (A/S) ratio:*

   The amount of air and the method of mixing the air with the material to be floated are functions of the design of a particular flotation unit. For a specific application, a definite amount of air is necessary for flotation. In thickening applications it has been shown that increased performance is obtained at higher A/S ratios.

7. *Float removal:*

   A float-removal mechanism must be designed to have adequate capacity to remove water carryover. Various items to be considered in this design are the depth of submergence of the scooping mechanism and the speed of scoop operation.

15.15.4 Dissolved Air Flotation Theory

15.15.4.1 Gas-to-Solids Ratio of Full-Flow Pressurization System The performance of a DAF unit will be mainly dependent on the ratio of the amount of gas to the amount of suspended solids applied to the unit. Increasing the *A*/*S* ratio beyond an optimum value results in no significant increase in the performance efficiency of a DAF unit.

The mass ratio of gas to solids can be derived from gas concentrations, solid concentrations, pressure, and flow parameters. In the case of full-flow pressurization system (Fig. 15.21a), the mass flow rate of dissolved gas entering the flotation chamber is

$$G_{\mathrm{in}} = QC_{\mathrm{r}} \qquad \textbf{(15.31)}$$

where G_{in} = mass flow rate of dissolved gas entering the flotation chamber (mg/s), Q = flow rate of influent feed stream (L/s), and C_{r} = solubility of gas in water in the pressurized retention tank (mg/L). The dissolved gas in the liquid leaving the flotation chamber is

$$G_{\text{out}} = QC_{\text{e}} \tag{15.32}$$

where G_{out} = mass flow rate of dissolved gas leaving the flotation chamber (mg/s) and C_{e} = solubility of gas in the flotation effluent (mg/L). The gas released for flotation of suspended solids (G, mg/s) is given by

$$\begin{aligned} G &= G_{\text{in}} - G_{\text{out}} \\ &= Q(C_{\text{r}} - C_{\text{e}}) \end{aligned} \tag{15.33}$$

At low dissolved gas concentrations where Henry's law is valid, the dissolved gas concentrations at saturation are proportional to gas pressures:

$$C_{\text{r}}/C_{\text{e}} = P_{\text{r}}/P_{\text{e}} = P/1 \tag{15.34}$$

where P_{r} = gas pressure in the retention tank, atmosphere, assigned to be P, and P_{e} = gas pressure in the flotation chamber's effluent compartment, atmosphere, taken to be 1 atm. The released gas can be expressed in terms of pressure as

$$G = QC_{\text{e}}\,(P - 1) \tag{15.35}$$

A correction factor, F, can be applied to the pressure term because complete gas saturation of liquid is often not achieved in a pressurized retention tank:

$$G = QC_{\text{e}}\,(FP - 1) \tag{15.36a}$$

$$G = QC_{\text{e}}f(P - 1) \tag{15.36b}$$

where f = factor of gas dissolution at pressure P (where P is any pressure above normal atmospheric pressure of 1 atm), fraction, usually 0.167–1.0, and F = fraction of gas dissolution at pressure P (where P is any pressure higher than 2 atm), fraction, usually 0.5–1.0.

Equation (15.36a) has successfully been used by design engineers since 1970s for high-pressure systems, such as DAF, in which P is higher than 2 atm. Equation (15.36a), however, is invalid for a lower pressure system in which P is below 2 atm. A new factor (f) and Eq. (15.36b) cover any pressure ranges above normal atmospheric pressure of 1 atm, but the mass flow rate of suspended solids entering the flotation system is

$$S = QX \tag{15.37}$$

where S = mass flow rate of suspended solids entering the flotation system (mg/s) and X = suspended solid concentration of influent feed stream (mg/L). The gas-to-solids ratio is thus given by

$$G/S = C_{\text{e}}\,(FP - 1)\,/X \tag{15.38a}$$

$$G/S = C_{\text{e}}\,(f)\,(P - 1)\,/X \tag{15.38b}$$

If the gas to be pressurized is air, the air-to-solids ratio for the full-flow pressurization system can be derived as

$$A/S = 1.3a\,(FP - 1)\,/X \tag{15.39a}$$

$$A/S = 1.3a\,(f)\,(P - 1)\,/X \tag{15.39b}$$

where A/S = air-to-solids ratio, A = mass flow rate of air released for flotation of suspended solids (mg/s), S = mass flow rate of suspended solids entering the flotation system (mg/s), a = air solubility in effluent at 1 atm pressure (mL/L), 1.3 = weight in milligrams of 1 mL of air at 0°C, 1 = 1 atm of air remaining in solution after depressurization, f = factor of gas dissolution at pressure P (where P is any pressure above normal atmospheric pressure of 1 atm), fraction, usually 0.167–1.0, and F = fraction of gas dissolution at pressure P (where P is any pressure higher than 2 atm), fraction, usually 0.5–1.0.

15.15.4.2 Gas-to-Solids Ratio of Partial Flow Pressurization System

In the case of partial flow pressurization system (Fig. 15.21b) the dissolved gas entering the flotation chamber is

$$G_{\text{in}} = Q_{\text{p}}C_{\text{r}} + Q_{\text{n}}C_{\text{f}} \tag{15.40}$$

where Q_{p} = portion of influent feed stream, which is pressurized (L/s), Q_{n} = portion of influent feed stream which is not pressurized (L/s), and C_{f} = dissolved gas concentration in the raw influent feed stream, mg/L.

The dissolved gas out of the flotation chamber is

$$G_{\text{out}} = (Q_{\text{p}} + Q_{\text{n}})C_{\text{e}} = QC_{\text{e}} \tag{15.41}$$

Assuming that at 1 atm

$$C_{\text{f}} = C_{\text{e}} \tag{15.42}$$

The gas released to float suspended solids is

$$\begin{aligned} G &= G_{\text{in}} - G_{\text{out}} \\ G &= Q_{\text{p}}(C_{\text{r}} - C_{\text{e}}) \end{aligned} \tag{15.43}$$

Assuming dissolved gas concentrations are low and Henry's law is valid, and then applying the correction factor, F, the mass flow rate of released gas is

$$G = Q_{\text{p}}C_{\text{e}}\,(FP - 1) \tag{15.44a}$$

$$G = Q_{\text{p}}C_{\text{e}}\,(f)\,(P - 1) \tag{15.44b}$$

The gas-to-solids ratio for the partial flow pressurization system (Fig. 15.21b) is

$$G/S = Q_{\text{p}}C_{\text{e}}\,(FP - 1)\,/QX \tag{15.45a}$$

$$G/S = Q_{\text{p}}C_{\text{e}}\,(f)\,(P - 1)\,/QX \tag{15.45b}$$

The Q_{p}/Q ratio ranges between 0.3 and 0.5.

Table 15.4 Densities and solubilities of air at various temperatures, under 1 atm (14.7 psig) pressure in the absence of water vapor

Temperature		Volume solubility		Weight solubility		Density	
°C	°F	mL/L	ft^3/1,000 gal	mg/L	lb/1,000 gal	g/L	lb/ft^3
0	32	28.8	3.86	37.2	0.311	1.293	0.0808
10	50	23.5	3.15	29.3	0.245	1.249	0.0779
20	68	20.1	2.70	24.3	0.203	1.206	0.0752
30	86	17.9	2.40	20.8	0.175	1.166	0.0727
40	104	16.4	2.20	18.5	0.155	1.130	0.0704
50	122	15.6	2.09	17.0	0.142	1.093	0.0682
60	140	15.0	2.01	15.9	0.133	1.061	0.0662
70	158	14.9	2.00	15.3	0.128	1.030	0.0643
80	176	15.0	2.01	15.0	0.125	1.000	0.0625
90	194	15.3	2.05	15.0	0.125	0.980	0.0612
100	212	15.9	2.13	15.0	0.125	0.949	0.0591

If the gas to be pressurized is air, the air-to-solids ratio for the partial flow pressurization system becomes

$$A/S = 1.3aQ_p(FP - 1)/QX \quad \textbf{(15.46a)}$$

$$A/S = 1.3aQ_p(f)(P - 1)/QX \quad \textbf{(15.46b)}$$

15.15.4.3 Gas-to-Solids Ratio of Recycle Flow Pressurization In the case of recycle flow pressurization system (Fig. 15.21c), the dissolved gas entering the flotation chamber is

$$G_{in} = QRC_r + QC_f \quad \textbf{(15.47)}$$

where R = recirculation ratio = Q_r/Q, Q_r = recycle flow (L/s), and C_f = dissolved gas concentration in influent feed stream (mg/L). The dissolved gas out of the flotation chamber is

$$G_{out} = Q(1 + R)C_e \quad \textbf{(15.48)}$$

Assuming that at 1 atm

$$C_f = C_e \quad \textbf{(15.49)}$$

The gas released to float suspended solids is

$$G = G_{in} - G_{out} \quad \textbf{(15.50)}$$

Assuming dissolved gas concentrations at saturation are proportional to pressure, or follow Henry's law, and applying the correction factor, F or f, the mass flow rate of release gas is

$$G = QRC_e(FP - 1) \quad \textbf{(15.51a)}$$

$$G = QRC_e(f)(P - 1) \quad \textbf{(15.51b)}$$

The mass flow rate of suspended solids (S) is again given by QX, the gas-to-solids ratio with effluent recycle is

$$G/S = RC_e(FP - 1)/X \quad \textbf{(15.52a)}$$

$$G/S = RC_e(f)(P - 1)/X \quad \textbf{(15.52b)}$$

If the gas to be pressurized is air, the air-to-solids ratio for the recycle flow pressurization system can then be derived as

$$A/S = 1.3aR(FP - 1)/X \quad \textbf{(15.53a)}$$

$$A/S = 1.3aR(f)(P - 1)/X \quad \textbf{(15.53b)}$$

$$A/S = 1.3a(Q_r/Q)(FP - 1)/X \quad \textbf{(15.54a)}$$

$$A/S = 1.3a(Q_r/Q)(f)(P - 1)/X \quad \textbf{(15.54b)}$$

15.15.4.4 Air Solubility and Density in Water at 1 Atm The air solubility and density are a function of water temperature, water vapor, and pressure. These air characteristics in the absence of water vapor and under 1 atm (14.7 psig) can be found from Table 15.4.

Wang and Wang (1985) reported mathematical models of dissolved oxygen, air, vapor pressure, and evaporation for practical applications in environmental engineering. The following is their model for dissolved air solubility at 1 atm that is valid in the temperature range of 0–30°C in the absence of water vapor and in the temperature range of 0–60°C in the presence of water vapor:

$$C_{air} = 38.00000 - 1.05905T + 2.19504 \times 10^{-2}T^2$$
$$-0.28865 \times 10^{-3}T^3 + 1.67723 \times 10^{-6}T^4$$

where C_{air} = air weight solubility in water, mg/L, and T = water temperature, °C. Below 30°C the effect of water vapor on air solubility is not significant.

Table 15.5 shows a comparison of the air solubility data from Table 15.4 and the calculated air solubility using the Wang and Wang model. It is clear that there is a good fit of the calculated air solubility data and the theoretical air solubility values.

It is important to note that the factor of 1.3 in Eqs. (15.39a), (15.39b), (15.46a), (15.46b), (15.53a), (15.53b), (15.54a) and (15.54b) means that each 1 mL of air weighs 1.3 mg at 0°C, which is the air density at 0°C as shown in

Table 15.5 Comparison of the air solubility data from 15.4 and the calculated air solubility using the Wang and Wang model

Temperature (°C)	Air solubility (mg/L)	Calculated solubility (mg/L)
0	37.2	38.0
5	32.7	33.2
10	29.3	29.3
15	26.9	26.2
20	24.3	23.7
25	21.7	21.4
30	20.8	19.8

Table 15.4. Actual air density values in Table 15.4 should be used instead of 1.3 if the water temperature is not 0°C.

Alternatively, an engineer may choose air weight solubility instead of air volume solubility in Table 15.4 for DAF system design using Eqs. (15.38a), (15.38b), (15.45a), (15.45b), (15.52a), (15.52b), and the Wang and Wang model.

15.15.4.5 Pressure Calculations The gauge pressure of the pressurized retention tank can be controlled and readily read. The following are two equations for converting the gauge pressure to the atmospheric pressure:

$$P = (P_g + 14.7)/14.7 \quad \text{(US customary units)} \qquad \textbf{(15.55a)}$$

$$P = (P'_g + 101.35)/101.35 \quad \text{(SI units)} \qquad \textbf{(15.55b)}$$

where P = pressure (atm), P_g = gauge pressure (psig), and P'_g = gauge pressure (kPa).

15.15.4.6 Flotation Hydraulic Loading Rate As discussed previously, DAF is a process for removing suspended matter from water, wastewater, or sludge by means of minute air bubbles that upon attachment to a discrete particle reduce the effective specific gravity of the aggregate particle to less than that of water. Reduction of the specific gravity for the aggregate particle causes separation from the carrying liquid in an upward direction. The particle to be removed may have a natural tendency either to rise or to settle. Attachment of the air bubble to the particle induces a vertical rise rate noted as V_T (ft/s or m/s).

Figure 15.22 illustrates the basic design considerations of the flotation unit. The measurement of V_T will be discussed later. Because the influent feed stream must pass through the flotation chamber, the particle to be removed will have a horizontal velocity (V_H). Certain criteria have been established for limits of the parameter V_H, which sets the width and depth of the flotation chamber:

$$V_H = Q/A_c \qquad \textbf{(15.56)}$$

where V_H = horizontal velocity (m/s), Q = influent flow rate (m^3/s), and A_c = cross-sectional area of a flotation chamber (m^2).

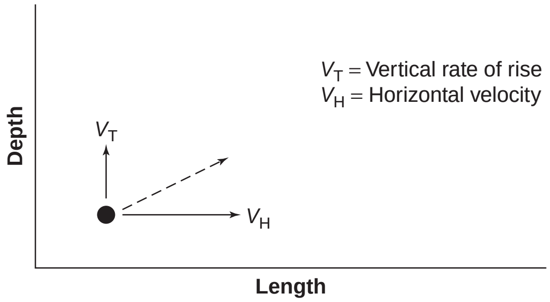


Figure 15.22 Basic design concept of flotation unit.

Figure 15.22 shows that the effective length (L) of the flotation chamber is directly proportional to the horizontal velocity and depth and inversely proportional to the vertical rate of rise of the particle to be removed. In the design of a flotation chamber, the usual procedure is to select the target suspended solids to be removed with a rise rate of V'_T and design the chamber so that all suspended solids that have a rise rate equal to or greater than V_T will be separated. The suspended solids must have sufficient rise velocity to travel the effective depth (the distance from the bottom to the water surface of the flotation chamber) within the detention time in order to be floated. That is, the rise rate V_T must be at least equal to the effective depth divided by the detention time, or equal to the flow divided by the surface area, which in effect is the *hydraulic loading*:

$$V_T = D/t = Q/A_s = \text{HL} \qquad \textbf{(15.57)}$$

where HL = hydraulic loading ($m^3/s/m^2$), V_T = vertical rise rate of suspended solids (m/s), D = effective depth of the flotation chamber (m), t = detention time (s), Q = influent flow rate (m^3/s), and A_s = surface area of the flotation chamber (m^2).

The ratio of Q/A_s (as in either a sedimentation clarifier or a flotation clarifier) is defined as the hydraulic loading rate, which is another very important design parameter. Theoretically, any particles having a rise rate equal to or greater than the hydraulic loading rate will be removed in an ideal flotation chamber.

In practical design, the rise rate (V_T) of suspended solids to be floated can be measured in the laboratory or in the field, and the influent feed rate (Q) is generally known. The minimum required surface area (A_s) of a flotation chamber can then be determined according to Eq. (15.57). The effects of short circuitry and turbulence in the flotation chamber, which interfere with the suspended solids rising through the water, should also be considered. Assuming the flotation chamber is rectangular in shape, the width (W) and the effective length (L) of the chamber can then be determined by Eqs. (15.58) and (15.59). The D/W ratio is usually between 0.3 and 0.5:

$$W = A_c/D = A_s/L \qquad \textbf{(15.58)}$$

$$L = (A_s/W)F' = (V_H/V_T)F'D \qquad \textbf{(15.59)}$$

where W = width of flotation chamber (m), L = effective length of flotation chamber (m), and F' = factor for short-circuiting and turbulence, assumed as 1.4.

15.15.4.7 Flotation Solids Loading Rate

In the flotation systems that contain low concentrations of suspended solids, only "free flotation" occurs. In free flotation, the suspended solids near the bottom of a flotation chamber rise freely toward the surface, and the floated suspended solids near the surface will not continue to compress with time. In "compression flotation," the suspended solids concentration is usually high. Initially the suspended solids also rise freely toward the surface forming a scum layer. As flotation continues, the floated suspended solids near the surface accumulate and the scum layer continues to compress with time; therefore, compression flotation permits the production of a high scum (or float) concentration for ease of handling and disposal. There are three different kinds of technologies for dissolving air into water: (a) deep-shaft air dissolving technology; (b) conventional air dissolving technology in a pressure tank; and (c) innovative air dissolving technology in an air dissolving tube using aspirated air.

The flotation phenomenon that occurs when a concentrated suspension, initially of uniform concentration throughout, is placed in a graduated cylinder may be observed as shown in Fig. 15.23.

Because of the hydraulic characteristics of flow around the particles and other interparticle forces, the particles float as a zone, maintaining the same relative position with respect to each other. As this region floats, a volume of relatively clear water is produced below the zone flotation region. Particles remaining in this region float as discrete or flocculated particles. A distinct interface exists between the discrete flotation region and the hindered-flotation region shown in Fig. 15.23. The rate of flotation in the hindered-flotation region is a function of the concentration of solids and their condition. As flotation continues, a compressed layer of particles begins to form on the top of the cylinder in the compression-flotation region. The particles in this region now apparently form a structure in which there is physical contact between particles. The forces of physical interaction between the particles are especially strong.

In design of a flotation system for sludge thickening, the overflow rate determination should be based on three factors: (a) the area needed for free flotation in the discrete flotation region; (b) the area needed on the basis of the rate of floating of the interface between water and sludge; and (c) the rate of floated sludge withdrawal from the compression region.

The area requirement for compression flotation may be calculated by the following method. Initially the flotation of a suspension is observed, and the position of the solid/liquid interface versus time is plotted as shown in Fig. 15.23. The rate at which the interface rises is then equal to the slope of a tangent to the curve at that point in time. According to the procedure, the critical area for flotation thickening is given by the following equation:

$$A_{\mathrm{s}} = Q(t_{\mathrm{u}})/H_0 \tag{15.60}$$

where Q = influent flow rate, ft^3/s (m^3/s), A_{s} = surface area required to reach desired sludge consistency, ft^2 (m^2), t_{u} = time to reach desired sludge consistency, s, and H_0 = height of floated sludge (float) in test cylinder, ft (m).

The critical concentration controlling the sludge thickening capacity of DAF is point C_{c} in Fig. 15.23. This point is determined by extending the tangents to the free and compression regions of the curve, intersecting the tangents, and bisecting the angle formed. The time t can be determined as follows: (a) construct a horizontal line at the height that corresponds to the desired float consistency; (b) construct a tangent to the curve at point C_{c}; and (c) construct a vertical line from the point of intersection of those two lines to the time axis.

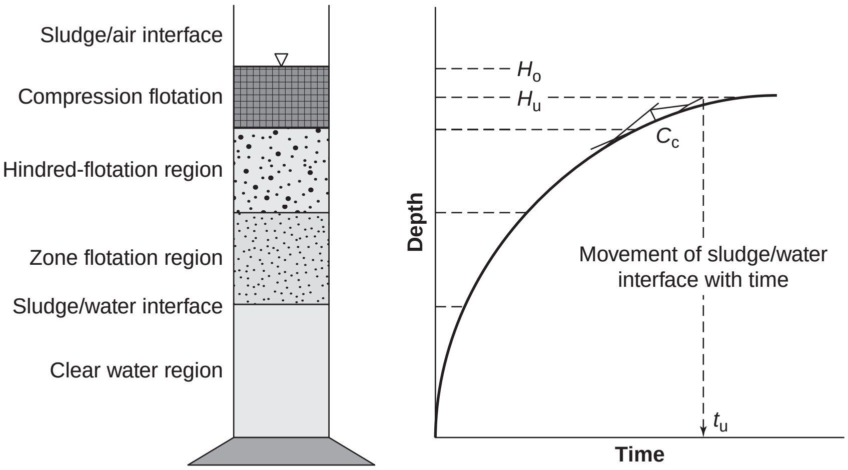


Figure 15.23 Schematic of flotation regions for solid–liquid separation and thickening curve (Wang and Wang, 1995).

Figure 15.24 DAF unit for 1.4 MGD (5.4 MLD) in operation at a paper mill (Courtesy http://en.wikipedia.org/wiki/File:REDOX_DAF_unit_225_m3-h-1000_GPM.jpg).

15.15.4.8 Volumetric Sludge Flow of Float The volumetric sludge flow of float from a DAF clarifier or thickener can be estimated by the following equation:

$$Q_s = QXE/(X_f S_s) \qquad \textbf{(15.61)}$$

where Q_s = sludge flow of float, ft^3/s (m^3/s); Q = influent flow, ft^3/s (m^3/s); X = influent suspended solids concentration, mg/L; X_f = float suspended solids concentration, mg/L; E = efficiency of suspended solids concentration, fractional number; and S_s = specific gravity of solids, dimensionless.

15.15.5 Flotation Design, Operation, and Performance

DAF units (Fig. 15.24) are typically designed based on air-to-solids ratio, hydraulic loading rate (or overflow rate), air pressure, effluent recycle, solids loading, and so on. The air-to-solids ratio is defined as the ratio of air feed to dry suspended solids feed by weight. The density of air is approx 0.08 lb/ft^3 or 1.293 g/L (see Table 15.4). The air-to-solids ratio is known when both air flow rate and influent suspended solids load are known:

$$A/S = dQ_a/W \qquad \textbf{(15.62)}$$

where d = air density, lb/ft^3 or g/L (Table 15.4); Q_a = air flow rate, ft^3/min (L/min); and W = influent suspended solids load, lb/min (g/min).

The air-to-solids ratio is important because it affects the sludge rise rate. The air-to-solids ratio needed for a particular application is a function primarily of the sludge's characteristics such as sludge volume index (SVI). The most common ratio used for design of a waste activated sludge thickener is 0.03.

Hydraulic loading rate (overflow rate) is the flow rate through the flotation chamber divided by the liquid surface area normally expressed in gpd/ft^2 (m^3/d/m^2) and must be adequately controlled. Air pressure in flotation is important because it determines air saturation or the size of the air bubbles formed. It influences the degree of solids concentration and the subnatant (separated water) quality. In general, an increase in either pressure or air flow produces greater float (solids) concentrations and a lower effluent suspended solids concentration. There is an upper limit, however, as too much air will tend to break up floc particles.

Additional recycle of clarified effluent does two things: (a) it allows a larger quantity of air to be dissolved because there is more liquid; and (b) it dilutes the feed sludge. Dilution reduces the effect of particle interference on the rate of separation. The concentration of sludge increases and the effluent suspended solids decrease as the sludge-blanket detention period increases.

The solid loading (SL) is the dry weight of suspended solids per unit time per unit of flotation surface area. This is normally expressed as lb/h/ft^2 or kg/h/m^2:

$$\mathrm{SL} = W/A_s = QX/A_s \qquad \textbf{(15.63)}$$

where A_s = the surface area in ft^2 (m^2) and W = influent suspended solids load, lb/min (kg/min).

The gas or air released at atmospheric pressure can be estimated from Eqs. (15.64a)–(15.67a) or Eqs. (15.64b)–(15.67b):

$$C_g = C_{gs}(FP - 14.7)/14.7 \quad \text{(US customary units)} \qquad \textbf{(15.64a)}$$

$$C_g = C_{gs}(f)(P - 14.7)/14.7 \quad \text{(US customary units)} \qquad \textbf{(15.65a)}$$

$$C_a = C_{as}(FP - 14.7)/14.7 \quad \text{(US customary units)} \qquad \textbf{(15.66a)}$$

$$C_a = C_{as}(f)(P - 14.7)/14.7 \quad \text{(US customary units)} \qquad \textbf{(15.67a)}$$

where C_g = gas released at one atmospheric pressure (mg/L); C_{gs} = gas saturation concentration at atmospheric conditions (mg/L), C_a = air released at 1 atm pressure (mg/L); C_{as} = air saturation concentration at atmospheric conditions (mg/L); P = absolute pressure in air dissolving tank (psig); F = system gas dissolving efficiency, fraction, usually 0.5–1.0; and f = system gas dissolving efficiency, fraction, usually 0.167–1.0.

The corresponding equations in the SI system are

$$C_g = C_{gs}(FP - 101.35)/101.35 \quad \text{(SI units)} \qquad \textbf{(15.64b)}$$

$$C_g = C_{gs}(f)(P - 101.35)/101.35 \quad \text{(SI units)} \qquad \textbf{(15.65b)}$$

$$C_a = C_{as}(FP - 101.35)/101.35 \quad \text{(SI units)} \qquad \textbf{(15.66b)}$$

$$C_a = C_{as}(f)(P - 101.35)/101.35 \quad \text{(SI units)} \qquad \textbf{(15.67b)}$$

Figure 15.25 First US DAFF 1 MGD potable water treatment plant was built in 1982 in Lenox, MA (Krofta and Wang, 1985).

where C_g = gas released at one atmospheric pressure (mg/L); C_{gs} = gas saturation concentration at atmospheric conditions (mg/L); C_a = air released at 1 atm pressure (mg/L); C_{as} = air saturation concentration at atmospheric conditions (mg/L); P = absolute gauge pressure in air dissolving tank (kPa); F = system gas dissolving efficiency, fraction, usually 0.5–1.0; and f = system gas dissolving efficiency, fraction, usually 0.167–1.0.

Table 15.4 lists the solubility of air in water with varying temperature. The values listed may be used in Eqs. (15.66a) and (15.67a) or Eqs. (15.66b) and (15.67b) for the design of a DAF system. Similar values of C_{as} for other gases are available in standard textbooks. It should be noted that an appropriate correction factor should be used when applying the data of Table 15.4 to wastewater or saline water containing a high salt concentration. For instance, less than 80% of the air soluble in pure water is soluble in sea water.

15.15.6 Municipal Potable Water Plants

DAF can be used alone as an upgrade clarifier to an existing water filtration plant or in combination with sand filtration (DAFF) built together as a package WTP. The first US potable water DAFF plant, Lenox Water Treatment Plant, was built in 1982 in Lenox, Massachusetts (Fig. 15.25). In 1986 the world's largest potable water flotation plant at the time was built in Pittsfield, Massachusetts (Fig. 15.26). Flotation (DAF and DAFF) technology has been successfully adopted in the United States in new potable WTPs. Two huge metropolitan areas such as New York City and Boston are considering possible adoption. A significant upgrade and retrofit market could become available for large- to medium-sized plants. Several large-capacity DAF and DAFF potable WTPs will come on line over the next few years. These plants will much enhance the reputation of DAF and DAFF as mainstream water clarification processes. The following is a partial list of potable DAF and DAFF built in the United States since 1982:

Figure 15.26 World's largest DAFF 37.5 MGD potable water treatment plant was built in 1986 in Pittsfield, MA (Wang, 1991).

1. Lenox WTP: 1.2 MGD (6 MLD); Town of Lenox, MA, in service since 1982.
2. Ashley WTP and Cleveland WTP: 37.5 MGD (138 MLD); City of Pittsfield, MA, in service since 1986.
3. Howell WTP: 8 MGD (30 MLD); New Jersey American Water Works, Howell, NJ, in service since 1990.
4. Nanty-Glo WTP: 2 MGD (7.6 MLD); Town of Nanty-Glo, PA, in service since 1991.
5. Mt Vernon WTP: 8 MGD (30 MLD); Mt. Vernon Water Works, Mt. Vernon, IN, in service since 1993.
6. Millwood WTP: 7.5 MGD (28 MLD); Town of New Castle, NY, in service since 1993.
7. Westmoreland WTP: 1.8 MGD (6.8 MLD); Westmoreland County Water Authority, Westmoreland, PA, in service since 1995.
8. Lakeville WTP: 0.75 MGD (2.8 MLD); Bridgeport Hydraulic Company, Lakeville, CT, in service since 1996.
9. Lake Vangum WTP: 0.75 MGD (2.8 MLD); Bridgeport Hydraulic Company, Lakeville, CT, in service since 1996.
10. Lee WTP: 2.0 MGD (7.5 MLD); Town of Lee, MA, in service since 1998.
11. Greenville Water System: 75 MGD (280 MJD); Greenville Water System, Greenville, SC, in service since 2000.
12. West Nyack WTP: 20 MGD (75 MLD); West Nyack, NY, in service since 2003.

It is encouraging to note that more new DAF and DAFF potable WTPs are being designed or under construction. The following is a partial list of potential, new DAF and DAFF WTPs to be built in the United States:

1. Boston WTP: 450 MGD (1700 MLD); Massachusetts Water Resources Authority, Boston, MA.
2. Hemlocks WTP: 50 MGD (190 MLD; Bridgeport Hydraulic Company, CT.
3. Cambridge WTP: 24 MGD (91 MLD); City of Cambridge, MA.
4. Montrose WTP: 7 MGD (26 MLD); Northern Westchester Joint Water Works, NY.
5. Danbury WTP: 5.5 MGD (21 MLD); City of Danbury, CT.
6. New York City Croton WTP, 290 MGD (1,098 MLD); New York, NY.

Internationally, flotation (DAF and DAFF) technology has been adopted for potable water treatment in both industrial and developing countries. Under an assignment of the US Department of State, Dr. Lawrence K. Wang served as the Senior Advisor of the United Nations Industrial Development Organization (UNIDO), Vienna, Austria. DAF and DAFF technology was inspected and recommended by UNIDO for adoption by developing countries that urgently need cost-effective water and wastewater treatment systems.

EXAMPLE 15.12 PRESSURE CALCULATION FOR FULL-FLOW DAF SYSTEM

Calculate the required pressure for thickening alum sludge flow using a full-flow DAF system. Assume (a) *A*/*S* (air-to-solids ratio) = 0.03 based on the manufacturer's pilot plant demonstration; (b) *a* (air solubility) at standard conditions = 28.8 mL/L; (c) *X* (suspended solids concentration) of alum sludge = 200 mg/L; and (d) *F* (factor of air dissolution) = 0.5.

Solution 1 (US Customary System):

From Eq. (15.39a),

$$A/S = 1.3a\,(FP - 1)\,/X$$

$$0.03 = 1.3\,(28.8)\,(0.5P - 1)\,/200$$

$$P = 2.3\ \text{atm.}$$

From Eq. (15.55a),

$$P = (P_g + 14.7)/14.7$$

$$2.3 = (P_g + 14.7)/14.7$$

$$P_g = 2.3 \times 14.7 - 14.7$$

$$\mathbf{P_g = 19\ psig}.$$

Solution 2 (SI System):

From Eq. (15.39a),

$$A/S = 1.3a\,(FP - 1)\,/X$$

$$0.03 = 1.3\,(28.8)\,(0.5P - 1)\,/200$$

$$P = 2.3\ \text{atm.}$$

From Eq. (15.55b),

$$P = (P'_g + 101.35)/101.35$$

$$2.3 = (P_g + 101.35)/101.35$$

$$P'_g = 2.3 \times 101.35 - 101.35$$

$$\mathbf{P_g = 132\ kPa}.$$

EXAMPLE 15.13 SIZING A FULL-FLOW DAF CLARIFIER BASED ON SOLIDS LOADING

Determine the surface area (A_s), volume (V), and detention time (t) of a full-flow DAF clarifier assuming (a) $Q = 1$ MGD $= 1.55$ ft^3/s (0.044 m^3/s); (b) $X = 200$ mg/L; (c) SL = 10 lb/ft^2/d (48.9 kg/m^2/d), and (d) depth of clarifier $D = 8$ ft (2.44 m).

Solution 1 (US Customary System):

From Eq. (15.63),

$$\begin{aligned} A_s &= QX/(\mathrm{SL}) \\ &= (1\,\mathrm{MGD})\,(200 \times 8.34\,\mathrm{lb/MG})\,/(10\,\mathrm{lb/ft^2/d}) \\ &= \mathbf{166.8\,ft^2}. \\ V &= A_s D \\ &= 166.8 \times 8 \\ &= \mathbf{1{,}334\,ft^3}. \\ t &= V/Q \\ &= (1{,}334\,\mathrm{ft^3}/1.55\,\mathrm{ft^3/s})/\,(60\,\mathrm{s/min}) \\ &= \mathbf{14.3\,min}. \end{aligned}$$

Solution 2 (SI System):

From Eq. (15.63),

$$\begin{aligned} A_s &= QX/(\mathrm{SL}) \\ &= (0.044\,\mathrm{m^3/s})\,(24 \times 60 \times 60\,\mathrm{s/d})\,\left(200 \times 10^{-3}\,\mathrm{kg/m^3}\right)/48.9\,\mathrm{kg/m^2/d} \\ &= \mathbf{15.5\,m^2}. \\ V &= A_s D \\ &= 15.5\,\mathrm{m^2} \times 2.44\,\mathrm{m} \\ &= \mathbf{37.8\,m^3}. \\ t &= V/Q \\ &= (37.8\,\mathrm{m^3}/0.044\,\mathrm{m^3/s})/\,(60\,\mathrm{s/min}) \\ &= \mathbf{14.3\,min}. \end{aligned}$$

EXAMPLE 15.14 SIZING A RECYCLE FLOW DAF CLARIFIER BASED ON HYDRAULIC LOADING

Determine the surface area (A_s), volume (V), and detention time (t) of a recycle DAF clarifier assuming (a) $Q = 1$ MGD $= 1.55$ ft^3/s (0.044 m^3/s); (b) $X = 200$ mg/L; (c) HL = 1.5 gpm/ft^2 (61 L/min/m^2); (d) depth of clarifier $D = 8$ ft (2.44 m); (e) $A/S = 0.05$; (f) $F = 0.05$; (g) $a = 28.8$ mL/L; and (h) $P = 4.4$ atm.

Solution 1 (US Customary System):

From Eq. (15.54a),

$$\begin{aligned} A/S &= 1.3a\left(Q_r/Q\right)(FP - 1)/X \\ 0.05 &= 1.3\,(28.8)\left(Q_r/1\right)(0.5 \times 4.4 - 1)/200 \\ Q_r &= 0.22\,\mathrm{MGD} \\ A_s &= (Q + Q_r)/\mathrm{HL} \\ &= (1 + 0.22) \times 10^6/\,(1.5 \times 60 \times 24) \\ &= \mathbf{565\,ft^2}. \\ V &= 565 \times 8 \\ &= \mathbf{4{,}520\,ft^3}. \end{aligned}$$

$$\begin{aligned} t &= V/\text{flow} \\ &= [4{,}520\ \text{ft}^3/(1+0.22)\ \text{MGD}(1.55\ \text{ft}^3/\text{s})(1/\text{MGD})]/(60\ \text{s/min}) \\ &= \mathbf{40\ min}. \end{aligned}$$

Note: This is a DAF design for standard temperature (°C) or cold temperature conditions. Under warm weather conditions, different air density and air solubility shall be used for DAF design. A_s and V will be larger because Q_r will be bigger.

Solution 2 (SI System):

From Eq. (15.54a)

$$\begin{aligned} A/S &= 1.3a(Q_r/Q)(FP-1)/X \\ 0.05 &= 1.3(28.8)\left(Q_r/0.044\right)(0.5\times 4.4-1)/200 \\ Q_r &= 0.01\ \text{m}^3/\text{s} \\ A_s &= (Q+Q_r)/\text{HL} \\ &= [(0.044+0.01)\ \text{m}^3/\text{s}\times 60\ \text{s/min}]/(0.061\ \text{m}^3/\text{min/m}^2) \\ &= \mathbf{53\ m^2}. \\ V &= 53\ \text{m}^2\times 2.44\ \text{m} \\ &= \mathbf{129\ m^3}. \\ t &= V/\text{flow} \\ &= [129\ \text{m}^3/(0.044+0.01)\ \text{m}^3/\text{s}]/60\ \text{s/min} \\ &= \mathbf{40\ min}. \end{aligned}$$

See the note in Solution 1.

EXAMPLE 15.15 SIZING A PRESSURE AIR DISSOLVING TANK FOR A PARTIAL FLOW DAF SYSTEM

A partial flow DAF system is used for alum sludge thickening. The total sludge flow is 1 MGD = 1.55 ft^3/s (0.044 m^3/s) of which only a portion is fed to a pressure air dissolving tank for pressurization, air addition, and bubble generation. Assume the partial flow to the DAF thickener's pressure air dissolving tank is 0.432 MGD = 0.53 ft^3/s (0.015 m^3/s), and the manufacturer's recommended detention time for the pressure air dissolving tank is 3 min. Determine the volume of the pressure air dissolving tank.

Solution 1 (US Customary System):

$$\begin{aligned} t &= V/\text{flow} \\ 3\ \text{min}\times 60\ \text{s/min} &= V/0.53\ \text{ft}^3/\text{s} \\ V &= 180\times 0.53 \\ &= \mathbf{95\ ft^3}. \end{aligned}$$

Solution 2 (SI System):

$$\begin{aligned} t &= V/\text{flow} \\ 3\ \text{min}\times 60\ \text{s/min} &= V/0.015\ \text{m}^3/\text{s} \\ V &= 180\times 0.015 \\ &= \mathbf{2.7\ m^3}. \end{aligned}$$

Note: At present new pressure air dissolving tanks are designed for just 30 s. If this criterion is adopted, the tank volume reduces to 15.8 ft^3 (0.45 m^3).

EXAMPLE 15.16 DETERMINATION OF VOLUMETRIC SLUDGE FLOW FROM A DAF CLARIFIER

Determine the volumetric sludge flow Q_f from a DAF clarifier assuming (a) influent flow Q to the clarifier = 1 MGD = 1.55 ft^3/s (0.044 m^3/s); (b) $X = 200$ mg/L; (c) solids concentration in the float $X_f = 5\%$; (d) S_s of solids = 1.05; and (e) efficiency of suspended solids removal, $E = 80\%$.

Solution 1 (U S Customary System):

$$\begin{aligned} Q_f &= QXE/X_f S_s \\ &= (1.55\,\text{ft}^3/\text{s})(200\,\text{mg/L})(0.80)/(50{,}000\,\text{mg/L})(1.05) \\ &= 4.72 \times 10^{-3}\,\text{ft}^3/\text{s} \\ &= \mathbf{3050\,gpd}. \end{aligned}$$

Solution 2 (SI System):

$$\begin{aligned} Q_f &= QXE/X_f\, S_s \\ &= (0.044\text{m}^3/\text{s})(200\,\text{mg/L})(0.8)/(50,000\,\text{mg/L})(1.05) \\ &= 1.34 \times 10^{-4}\,\text{m}^3/\text{s} \\ &= \mathbf{11.6\,m^3/d = 11,600\,L/d}. \end{aligned}$$

EXAMPLE 15.17 DESIGN OF A DAF THICKENER FOR ALUM SLUDGE

Determine the surface area required to reach desired sludge consistency of 15,000 mg/L, assuming (a) initial sludge concentration $X = 3{,}500$ mg/L; (b) alum sludge flow, $Q = 1$ MGD = 1.55 ft^3/s (0.044 m^3/s); (c) a DAF thickening curve shown in Fig. 15.23 is obtained experimentally; (d) the height of the DAF thickening column is 100 cm ($H_0 = 100$ cm); (e) the distance between H_0 and H_u is 22.25 cm; and (f) $t_u = 8$ min.

Solution 1 (US Customary System):

From Eq. (15.60),

$$\begin{aligned} A_s &= Qt_u/H_0 \\ &= (1.55\,\text{ft}^3/\text{s})(8 \times 60\,\text{s})/(100\,\text{cm}/30.48\,\text{cm/ft}) \\ &= \mathbf{227\,ft^2}. \end{aligned}$$

Solution 2 (SI System):

$$\begin{aligned} A_s &= Qt_u/H_0 \\ &= (0.044\,\text{m}^3/\text{s})(8 \times 60\,\text{s})/(100\,\text{cm}/100\,\text{cm/m}) \\ &= \mathbf{21\,m^2}. \end{aligned}$$

Note: For sizing a flotation thickener, the surface area A_s should be determined based on (a) the DAF thickening curve using Eq. (15.60); (b) the solids loading using Eq. (15.63); (c) the hydraulic loading using Eq. (15.57) and comparing the three calculated areas to choose the largest for the design.

PROBLEMS/QUESTIONS

15.1 Name the four types of sedimentation? Briefly define each type.

15.2 Name and briefly define the four settling zones in a sedimentation tank.

15.3 Differentiate between natural and air flotation.

15.4 Explain the nature of the main forces involved in settling of suspended particles in water.

15.5 What are the factors that affect the settling velocity?

15.6 A sedimentation tank is 15 ft (4.57 m) wide, 10 ft (3.05 m) deep, and 100 ft (30.48 m) long. Determine

- **(a)** The detention time in hours and
- **(b)** The overflow rate (OR) in gallons/ft^2/d (m^3/h/m^2) when the water flow rate into the tank is 1.0 MGD (3.78 MLD).

15.7 Design a sedimentation tank for a flow of $Q = 3{,}500$ m^3/d. Assume the following design criteria:

- **(a)** Overflow rate, OR = 35 m^3/d/m^2
- **(b)** Detention time, $t = 2$ h
- **(c)** Length-to-width ratio, $L{:}W = 4{:}1$
- **(d)** Horizontal velocity < 1 m/min
- **(e)** Weir overflow rate, $\text{OR}_{\text{weir}} = 250$ m^3/d/m

15.8 Determine for floc ($s_s = 1.01$), 2×10^{-1} cm in diameter, the displacement velocity at which the floc can be removed without causing resuspension and the length-to-depth ratio of the settling basin in which the removal can be effected. Assume a friction factor $f = 4.0 \times 10^{-2}$ and a temperature of 10°C.

15.9 A water treatment plant has six sedimentation basins to treat 9 MGD of water. Each tank is 70 ft (21.34 m) long, 25 ft (7.62 m) wide, and 15 ft (4.57 m) deep (working dimensions).

- **(a)** What is the detention time in the basins?
- **(b)** What is the overflow rate of the basins?
- **(c)** What % of particles having a settling velocity of 3 ft/h (0.91 m/h) would be removed in these basins?

15.10 A water treatment plant has a capacity of 10 MGD (37.8 MLD). The water is distributed among a row of five identical sedimentation tanks. Each tank is 80 ft (24.38 m) long, 18 ft (5.49 m) wide, and 9 ft (2.74 m) deep (working dimensions). Assume ν of water is 1.12×10^{-2} cm^2/s. What percent of particles having a diameter of 0.0025 cm and specific gravity of 2.50 would be removed in the sedimentation tanks?

15.11 A water treatment plant is treating 5.0 MGD (18.9 MLD) of water. The raw water has a suspended solids concentration of 300 mg/L, composed of particles having a diameter of 0.0015 cm and specific gravity of 2.20. The plant has four sedimentation tanks. Each tank is 80 ft (24.38 m) long, 25 ft (7.62 m) wide, and 8 ft (2.44 m) deep (all are water working dimensions). Assume ν of water is 1.12×10^{-2} cm^2/s

- **(a)** What will be the detention time for sedimentation? (h)
- **(b)** What will be the overflow rate of the tanks? (gal/ft^2/d or m^3/d/m^2)
- **(c)** What is the % solids removal or settling efficiency of the plant? (%)
- **(d)** What is the actual velocity of a solid particle in the tanks? (cm/s)

15.12 A rectangular water sedimentation tank is to treat 40,000 m^3/d of water with a detention period of 1.5 h. The raw water entering the basin has the following distribution of particulate matter:

Diameter of particle (cm)	% by weight of total
0.0005	10
0.001	12
0.005	13
0.010	20
0.015	15
0.017	10
0.020	5
0.030	7
0.040	3
0.050	5

$$s_s = 1.20$$

$$\nu = 1.20 \times 10^{-2}\ \text{cm}^2/\text{s}$$

- **(a)** If the tank is to remove 100% of 0.017 cm size particles, determine the required overflow rate (m^3/m^2/d).
- **(b)** If the length of the tank is six times its width, determine its working dimensions (m).
- **(c)** Calculate the tank efficiency.

15.13 A rectangular sedimentation tank is to treat 8,500 m^3/day, with a detention period of 2 h. The raw water entering the tank has the following distribution of discrete particulate matter:

Particle diameter (cm)	weight (%)
0.001	20
0.005	20
0.010	15
0.015	10
0.020	25
0.025	10
	Total = 100%

Properties at 20°C are

$\rho_s = 1.10$ g/cm^3

$\rho_w = 1.00$ g/cm^3

$\mu_w = 1.01 \times 10^{-2}$ poises

$g = 981$ cm/s^2

- **(a)** If the tank is to remove 100% of 0.010 cm size particles, determine the required overflow rate, m^3/m^2/day.
- **(b)** If the length of the tank is five times its width, find its dimensions (length, width, and depth).
- **(c)** Compute the total percentage of solids that are expected to be removed.

15.14 A laboratory settling analysis of a water or wastewater settling tank effluent (see figure) produced the data shown in the table. Design a settling tank 8 ft (2.44 m) deep to remove 68% of the suspended solids for a flow, Q equals to 1 MGD (3.78 MLD). Use a factor of safety equals 2.

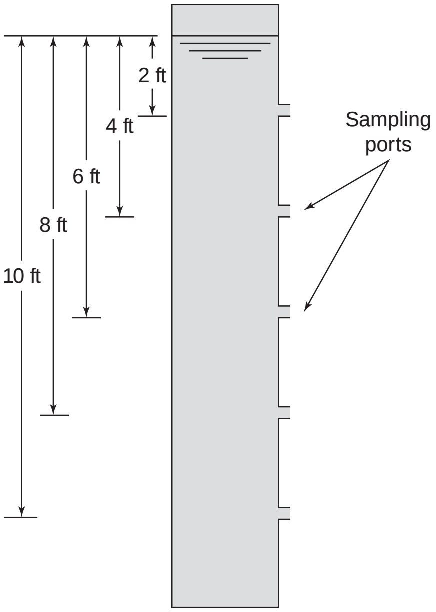


Figure Settling column for Problem 15.14 (conversion factor: 1 ft = 0.3048 m).

Table Settling data for Problem 15.14

Time (min) / Depth (fit)	% suspended solids removed at indicated depth and time					
	15	30	45	60	90	120
2	50	65	69	72	73	74
4	31	56	64	67	72	73
6	20	49	61	64	69	72
8	16	41	57	62	67	71
10	15	37	52	61	65	69

15.15 The inlet farthest from the point of supply of a settling tank is to discharge 97% of the flow delivered by the nearest inlet. Find, in terms of the friction head h_f,

(a) The required head loss through the nearest inlet and

(b) The associated head loss through the farthest inlet.

15.16 Design a tube settler for increasing the capacity of an existing settling tank provided for a water treatment plant from 2 to 4 MGD (7.56–15.12 MLD). The existing tank dimensions are 20 ft × 120 ft × 10 ft deep (6.10 m × 36.58 m × 3.05 m). Raw water has a turbidity of 25–30 JTU and temperature of up to 10°C. Effluent turbidity desired is 1–3 JTU.

15.17 Find the rising velocity in water at 20°C of a spherical particle with a specific gravity of 0.90 and a diameter of 3×10^{-3} cm.

15.18 Determine the weight solubility of air in water under 1 atm (102 kPa) pressure if the water temperature is 18°C.

15.19 A pilot plant investigation was conducted in a northern country on dissolved air flotation (DAF) for treating an industrial process water. The results showed that the optimum air to solids ratio (A/S) was near 0.03. The design rise rate of the flow was established at 2.4 in./min (6.1 cm/min) as the optimum A/S ratio. Determine

(a) The surface hydraulic loading rate

(b) DAF surface area

(c) DAF detention time

(d) Solids loading rate

(e) Operating pressure

(f) Weight of air required

(g) Volume of air required

Assume the following conditions:

(a) Full-flow DAF (without recycle) system with a depth of 5 ft (1.52 m).

(b) Treatment plant flow = 0.3 MGD (0.01314 m^3/s).

(c) Influent water temperature = 16°C.

(d) Influent water TSS = 1,200 mg/L.

(e) Fraction factor F of air dissolved in a 1-min air dissolving tank at normal operating pressure is 0.5.

(f) Atomic weights of oxygen and nitrogen are 16 and 14, respectively.

(g) Air volume at standard conditions = 359 ft^3/lb mole (22.4 m^3/kg mole).

15.20 Conduct an Internet search to find the commercially available sedimentation clarifiers, sedimentation thickeners (i.e., gravity thickeners), flotation clarifiers, and flotation thickeners. Write an engineering report to introduce and discuss their brand/model names, process principles, advantages, and disadvantages.

15.21 Search the Internet to find any natural flotation process that may involve the use of lamella separator, parallel plates, or tube floater for enhancing flotation. Introduce the process name and draw one or more flow diagrams. Discuss the process theory, advantages, and disadvantages if any.

15.22 Search the Internet to find any dissolved air flotation process that may involve the use of lamella separator, parallel plates, or tube floater for enhancing flotation. Introduce the process name and draw one or more flow diagrams. Discuss the process theory, advantages, and disadvantages if any.

15.23 Define the process of dissolved air flotation. Discuss the process terminologies of gas flotation, dissolved nitrogen flotation, dissolved ozone flotation, dissolved carbon dioxide flotation, dissolved oxygen flotation, and dissolved air-ozone flotation in comparison to dissolved air flotation clarification.

15.24 It has been known for several decades that heavy fine particles can be used as the weighing agents or sedimentation aids to increase the settling rate in sedimentation clarifiers. A high-rate sand-ballasted sedimentation process has been developed to utilize the sedimentation aids. The process involves five steps: coagulation, microsand and polymer injection, floc maturation, settling, and sand recirculation. Conduct an Internet research to search for the developer's process names, flow diagrams, case histories, advantages, and disadvantages for water and wastewater clarification.

15.25 What is the process flow diagram of a potable water flotation-filtration plant in comparison with a conventional water treatment plant consisting of chemical feeding rapid mixing, coagulation-flocculation, filtration, and disinfection?

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Chapter 16

Conventional Filtration

The seepage of rainfall and runoff into porous soils and rocks and the storage and movement of groundwater in open-textured geological formations are important elements in the resource and quality management of water and wastewater. Although fine-textured granular materials remove pollutants, the water drawn from them is acceptable only when *natural filtration* together with *time lapse* between pollution and use bars the rapid transport of pollutants to springs, wells, and infiltration galleries.

Natural and managed recharge of aquifers with surface or spent waters may serve one or more of the following purposes: (a) water conservation by underground rather than surface storage; (b) water quality improvement by storage and filtration; (c) replenishment of overdrawn aquifers; and (d) freshwater protection by the erection of groundwater barriers against saltwater intrusion.

Early water filters were constructed as small sand beds in 1804 in Paisley, Scotland, and as large sand beds in 1828 for the clarification of Thames River water in London, England. Filters of this kind were not introduced into the United States until 1872, when *English* filter was constructed in Poughkeepsie, NY, on the banks of the Hudson River.

Early wastewater filters evolved as natural sand beds from New England *sewage farms*. These as well as more sophisticated granular filters were pressed into service with increasing frequency as higher quality standards for wastewater effluents became more general in the protection of the water resource.

16.1 GRANULAR WATER FILTERS

Water filters constructed for cities and towns during most of the nineteenth century were *slow sand filters* that consisted of beds of *run-of-bank* sands provided with tile underdrains and with inlet and outlet wells. The beds were placed in operation by being filled from below to displace air from their pores, thereafter being kept flooded when in operation, even while the beds were being cleaned. Depending on the substances to be removed and their concentration, rates of filtration were seldom less than 1 MG/acre/d or more than 10 MG/acre/d (seldom less than 9.355 ML/ha/d or more than 93.55 ML/ha/d). In warm climates filter beds were usually left open to the sun and air. In cold climates they were generally covered to keep them from freezing.

The essential components of early slow sand filter beds are illustrated in Fig. 16.1a. Their general features of construction and operation are summarized in Table 16.1. Also summarized in this table are the features of *rapid sand filters* introduced toward the end of the nineteenth century (Fig. 16.1b). These could be operated about 30 times faster than the older filters. Although rapid filters soon began to be constructed in large numbers for the treatment of turbid river waters by combining alum or iron coagulation and sedimentation with rapid filtration, a number of places kept their slow sand filters going, and they remain of interest where raw-water quality and economics of construction and operation favor their use. Aside from their modest hydraulic loading, the distinguishing features of slow sand filters are their nonvarying sand mixtures at all bed depths, the small effective size and large coefficient of nonuniformity of their grains, and the associated selective removal and accumulation of raw-water impurities at the surface of the bed and within its top inch or two. Because of this, slow sand filters can be kept going for long periods of time, (a) if the surface of the partially dewatered bed is raked after about 2 weeks and again a week or so later to break through the surface accumulations and (b) if the top inch or two of the bed and the surface accumulations are scraped off at the end of a month and transported out of the filter for winter storage either before or after washing, or for immediate washing and return to the bed.

The incorporation of rapid filter units into municipal water works was encouraged by the successful studies of coagulation and rapid filtration in Louisville, KY, in 1895. The first sizable, municipal, rapid filter plant was constructed in Little Falls, NJ, in 1909.

Because rapid sand filters operate at many times the rate of slow sand filters, they need to be cleaned many times more often. As shown in Table 16.1, filter runs are normally no more than 1 day long. Distinguishing features of rapid filters, therefore, are their relatively small size and the ways in which they must be cleaned because they accumulate impurities

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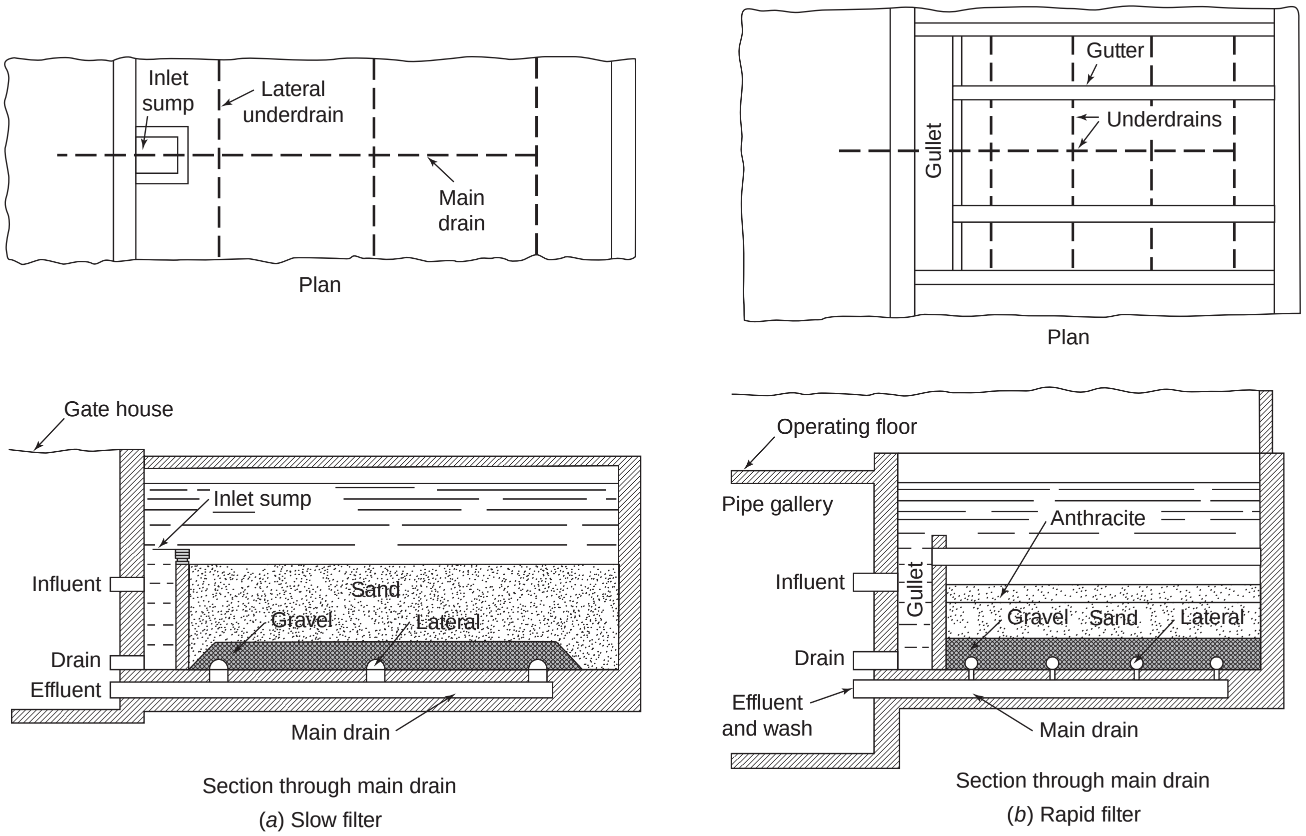
Water Engineering: Hydraulics, Distribution and Treatment, First Edition. Nazih K. Shammas and Lawrence K. Wang.



Section through main drain
(*a*) Slow filter

Section through main drain
(*b*) Rapid filter

Figure 16.1 Diagrammatic sections through and simplified plans of (a) a slow and (b) a rapid sand filter (After Fair et al., 1971).

rapidly and at nearly all depths. As suggested in Fig. 16.1b, the filter units are washed from below with previously filtered water retained in storage for that purpose. The underdrainage system then doubles as a washwater distribution system. The sand cleans itself when the bed has been *fluidized* by the rising water. Scour is intensified in one of the following ways:

1. By stepping up the velocity or rate of backwash per unit area sufficiently (*high-velocity wash*)
2. By directing jets of water into the fluidized bed (*surface scour*)
3. By blowing air upward through the bed before or during fluidization (*air scour*)
4. By stirring the fluidized bed mechanically (*mechanical scour*)

After the bed has been washed clean, the washwater is turned off and the sand settles back into place. The coarsest or heaviest grains reach the bottom first; the finest or lightest arrive last. This stratifies the bed and imposes a size limit on its finest or lightest particles.

Rapid filters have almost wholly displaced slow filters in North American practice because of their convenience of size, ability to remove turbidity in conjunction with coagulation and sedimentation or flotation, adaptability to changing raw-water quality, and overall economy of construction and operation under North American conditions.

The obtainable purification becomes very high when water is submitted, in succession, to rapid and slow filtration. However, this has been done principally where the treatment works began as slow filters and rapid units were added later either to compensate for deteriorating raw-water quality or to increase plant output. Recently, slow sand filtration has gained popularity for its use in the removal of trace organics from water.

Rapid filters enclosed in pressure tanks and operated under line pressures have been installed principally in industrial water systems and in the recirculation systems of swimming pools. Some of their operating parts are accessible only with difficulty, and their space requirements are often unfavorable.

In conventional downflow sand filters, the entering water comes into contact with fine grains before coarse grains or with clogged portions of the bed before open ones. The design of granular water filters is being modified to permit the influent water to pass through coarse grains before it is finally polished by passing through fine grains. This is accomplished (a) by using several media of varying specific gravity, with large lighter particles on top or (b) by upward flow in a conventionally stratified filter. Process design, too,

Table 16.1 General features of construction and operation of conventional slow and rapid sand filters

	Slow sand filters	Rapid sand filters
Rate of filtration	1–**3** to 10 **MG/acre/d**	100–**125** to 300 **MG/acre/d**
Size of bed	Large, $\frac{1}{2}$ acre	Small, $\frac{1}{100}$ to $\frac{1}{10}$ acre
Depth of bed	12 in. of gravel; 42 in. of sand, usually reduced to no less than 24 in. by scraping	18 in. of gravel; 30 in, of sand, or less; not reduced by washing
Size of sand	Effective size 0.25–**0.3** to 0.35 mm; coefficient of nonuniformity 2–**2.5** to 3	0.45 mm and higher; coefficient of nonuniformity 1.5 and lower, depending on underdrainagc system
Grain size distribution of sand in filter	Unstratified	Stratified with smallest or lightest grains at top and coarsest or heaviest at bottom
Underdrainage system	Split tile laterals laid in coarse stone and discharging into tile or concrete main drains	(1) Perforated pipe laterals discharging into pipe mains; (2) porous plates above inlet box; (3) porous blocks with included channels
Loss of head	0.2 ft initial to 4 ft final	1 ft initial to 8 or 9 ft final
Length of run between cleanings	20–**30** to 60 days	12–**24** to 72 h
Penetration of suspended matter	Superficial	Deep
Method of cleaning	(1) Scraping off surface layer of sand and washing and storing cleaned sand for periodic resanding of bed; (2) washing surface sand in place by washer traveling over sand bed	Dislodging and removing suspended matter by upward flow or backwashing, which fluidizes the bed. Possible use of water or air jets, or mechanical rakes to improve scour
Amount of washwater used in cleaning sand	0.2–0.6% of water filtered	1–**4**% to 6% of water filtered
Preparatory treatment of water	Generally none	Coagulation, flocculation, and sedimentation
Supplementary treatment of water	Chlorination	Chlorination
Cost of construction, U.S.A.	Relatively high	Relatively low
Cost of operation	Relatively low where sand is cleaned in place	Relatively high
Depreciation cost	Relatively low	Relatively high

*The most common values are shown in boldface type.

Source: After Fair et al. (1971).

Conversion factors: 1 MG/acre/d = 9.355 ML/ha/d; 1 acre = 4047 m^2; 1 ft = 0.3048 m; 1 in. = 25.4 mm.

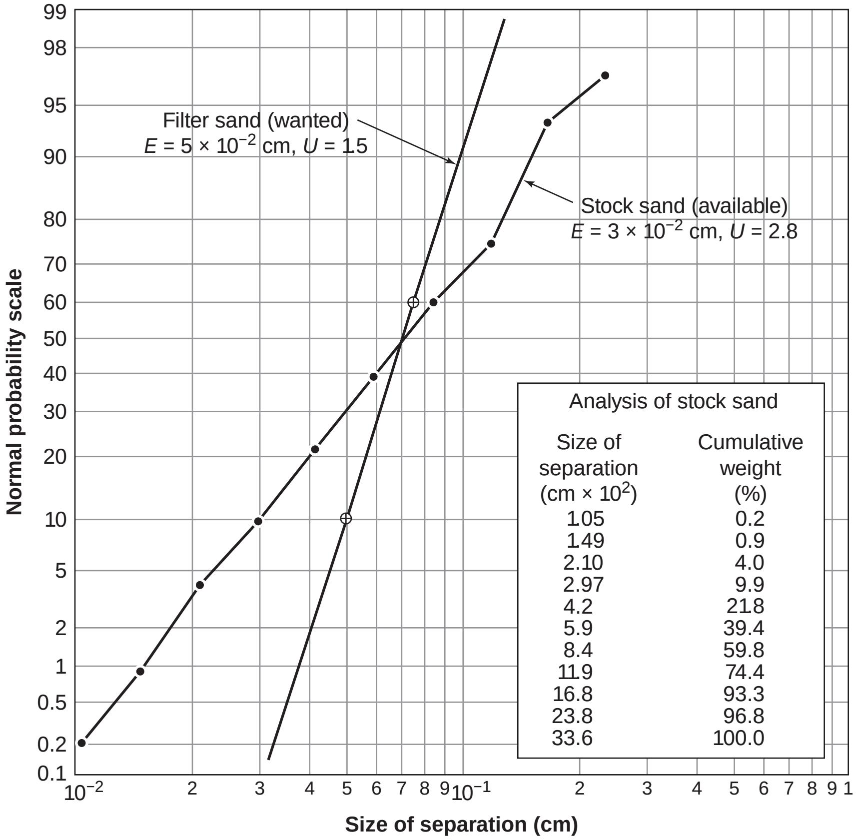
has been modified in a number of ways. Interesting is the optimization of the coagulation, flocculation, and filtration sequence, which may result in a heavier than conventional loading of the filter for economic reasons or other functional uses of the sequent components in ways somewhat different from the conventional.

16.2 GRANULAR WASTEWATER FILTERS

Because they are putrescible and clog filters rapidly, *raw* municipal wastewaters are not amenable to treatment in conventional slow or rapid filtration plants. However, effluents from biological or chemical wastewater treatment works can be filtered as successfully as heavily polluted raw waters. Sand filtration then becomes a *tertiary* wastewater treatment operation. As such it can provide improved protection to receiving bodies of water or prepare effluents for reuse in industry, agriculture, and recreation, and for recharge to the ground. Rapid filters serving these ends are patterned after rapid water filters but generally contain considerably coarser grains and operate at lower rates. Filter runs are often short, and beds may have to be heavily chlorinated or purged regularly with caustic or other chemicals to keep them open and sweet.

Under favorable geological and topographical conditions, natural sand deposits have been converted into *intermittent sand filters* to good purpose. Settled urban wastewaters are run onto them at rates averaging 40,000–120,000 gpd/acre (374,000–1,123,000 L/d/ha), and biologically treated effluents at 400,000–800,000 gpd/acre (3,744,000–7,487,000 L/d/ha). At such rates these filters produce excellent effluents.

An intermittent sand filter contains drainage pipes that are laid with open joints at depths of 3–4 ft (0.91–1.22 m) and surrounded with layers of coarse stone and gravel graded from coarse to fine to keep sand out. In deep sand deposits the percolating waters may reach the groundwater table, and no effluent may come to view. Influent wastewater is piped to the beds for discharge onto a protective stone or concrete



Analysis of stock sand

Size of separation (cm × 10^2)	Cumulative weight (%)
1.05	0.2
1.49	0.9
2.10	4.0
2.97	9.9
4.2	21.8
5.9	39.4
8.4	59.8
11.9	74.4
16.8	93.3
23.8	96.8
33.6	100.0

Figure 16.2 Grain size distribution of stock sand and required sizing of filter sand (After Fair et al., 1971).

apron or into a concrete flume distributor. Flooding the bed to a depth of 1–4 in. (25.4–101.6 mm) delivers 25,000–100,000 gal/acre (233,975–935,900 L/ha) of surface. One or more doses are applied each day for 7–20 min. Surface accumulations of solids are scraped off from time to time. Some sand is included in the scrapings, which are ordinarily buried or disposed of as fill. Filters are resanded when they become too shallow. In cold weather the beds are furrowed to keep them from freezing and opening up cracks through which the applied wastewater can escape with little treatment. To form protective sheets of ice spanning the furrows and keep the beds warm, furrowed beds are dosed deeply on cold nights.

16.3 GRANULAR FILTERING MATERIALS

Rapid filters may be composed of natural silica sand, crushed anthracite (hard) coal, crushed magnetite (ore), and garnet sands. Plastic spheres and granules manufactured to wanted specifications have been used experimentally.

The natural granular sands and crushed minerals normally employed differ in size and size distribution, in shape and shape variation, and in density and chemical composition. Mean values and variances provide acceptable parameters (a) for describing the geometry of the materials, (b) for the rational prediction of their hydraulic behavior, and (c) for assessing the removal of impurities by filtration.

16.3.1 Grain Size and Size Distribution

The parameters of mean grain size and variance in grain size are commonly determined by sieving a representative sample of filter grains through a set of calibrated sieves.

Starting with the weight of sievings shaken downward through a vertical stack of successively finer sieves to the pan at the bottom, the portions of sand held between adjacent sieves are added in sequence, and the cumulative weights are recorded. After conversion into percentages by weight equal to or less than the rated size of the overlying coarser sieve, a summation curve can be plotted for purposes of generalization (Fig. 16.2).

The American (US) standard sieve series (Appendix 24) is referred to a sieve opening of 1 mm (generated by approximately 18 meshes to the inch). Thence, sieve openings stand successively in the ratio of one to another, the largest being 5.66 mm (produced by approximately 3% meshes to the inch) and the smallest 0.037 mm (generated by approximately 400 meshes to the inch). For less precise measurements, every second sieve in the series is removed, possibly even every second sieve in the reduced stack. Resulting ratios are $\sqrt[4]{2} : 1, \sqrt[2]{2} : 1$, and $2 : 1$, but intermediate sieves within important size ranges may be left in place at the discretion of the analyst. US standard sieves are normally calibrated directly by measuring the clear dimensions of a representative

EXAMPLE 16.1 DETERMINATION OF GRAIN SIZE AND SIZE DISTRIBUTION OF FILTERING MATERIAL

For the size frequencies by weight of the sample of sand listed in Fig. 16.2, find (a) the effective size E and coefficient of nonuniformity U and (b) the geometric mean size μ_g and geometric standard deviation σ_g.

Solution:

a. From Fig. 16.2,

$$E = P_{10} = \mathbf{3.0 \times 10^{-2}\ cm}.$$
$$U = P_{60}/P_{10} = 8.5 \times 10^{-2}/(3.0 \times 10^{-2}) = \mathbf{2.8}.$$

b. From Fig. 16.2,

$$\mu_g = P_{50} = \mathbf{7.1 \times 10^{-2}\ cm}.$$
$$\sigma_g = P_{50}/P_{16} = 7.10 \times 10^{-2}/(3.7 \times 10^{-2}) = \mathbf{1.9\ cm}.$$

number of screen openings. This is known as the *manufacturer's rating*.

For many natural filtering materials, the summation curve of weights below or equal to given sieve sizes approaches geometric normality. Analyses plotted on logarithmic probability paper then trace an almost straight line, in which interpolation is straightforward (Fig. 16.2), and from which geometric mean size μ_g and geometric standard deviation in size σ_g can be read or calculated from the 50% and 15.84% values. In practice, the size–frequency parameters still in common use are *Hazen's effective size E*, or 10 percentile P_{10}, and *coefficient of nonuniformity U*, or ratio of the 60 percentile to the 10 percentile, P_{60}/P_{10}. Hazen (1892) chose the 10 percentile as the (hydraulically) effective size, because he had observed that the hydraulic resistance of unstratified sand beds was left relatively unaffected by size variation (up to a nonuniformity coefficient of about 5.0) so long as the 10 percentile remained unchanged.

It should be noted that (a) sizes of separation for sieves calibrated by Hazen's method are about 10% larger than corresponding sizes based on the manufacturer's rating; and (b) Hazen called $U = P_{60}/P_{10}$ the uniformity coefficient, but this ratio is more logically a coefficient of nonuniformity, because it increases numerically with nonuniformity rather than uniformity.

16.3.2 Grain Shape and Shape Variation

The surface area of filter sand or other granular materials in contact with water passing through a filter is both an operational and a hydraulic (frictional) determinant. Unfortunately there is as yet no satisfactory way to find this surface area by direct measurement. However, it is possible to establish the pore volume and complementary sand volume, V, of a filter without difficulty. Because the surface area A of grains relative to the volume of water in the pore space is a function of A/V, it is this size characteristic that is normally employed in formulations of filter performance and referred to in identifying shape. Among useful measures of shape are the *sphericity*, ψ, and the *shape factor*, S. Defined as the ratio of the surface area of the equivalent-volume sphere to the actual or true surface area, ψ assumes values such as those listed in Table 16.2. Representative shapes of particles are outlined in Fig. 16.3. For a single truly spherical grain of diameter d, ψ equals unity and, by definition, $A/V = \pi d^2/(1/6\ \pi d^3) = 6/d$. The shape symbols of true spheres are therefore a sphericity $\psi = 1.0$ and a shape factor $S = 6.0/\psi = 6.0$. These are, respectively, the largest possible value of ψ and smallest possible value of S. The magnitudes of ψ and S in Table 16.2 include measurements by Carman (1937).

The magnitude of ψ for a given sample of sieved particulates can be estimated from Fig. 16.3 or derived from observed settling velocities of representative grains when the temperature of the test fluid and the specific weight of the particulates are known. If the settling velocity is v_s and the grain diameter is d_s

$$\psi = d_o/d_s = \sqrt{v_s/v_o} \qquad \textbf{(16.1)}$$

Here d_o is the diameter of the equivalent-volume sphere read from Fig. 16.2 or Fig. 16.3, and v_o is its settling velocity.

The hydraulic properties of a bed or a layer of granular material do not depend on the ratio of surface area to volume

Table 16.2 Values of sphericity, shape factor, and porosity of granular material

Description	Sphericity, ψ	Shape factor, S	Typical porosity, f
Spherical	1.00	6.0	0.38
Rounded	0.98	6.1	0.38
Worn	0.94	6.4	0.39
Sharp	0.81	7.4	0.40
Angular	0.78	7.7	0.43
Crushed	0.70	8.5	0.48

Source: After Fair et al. (1971).

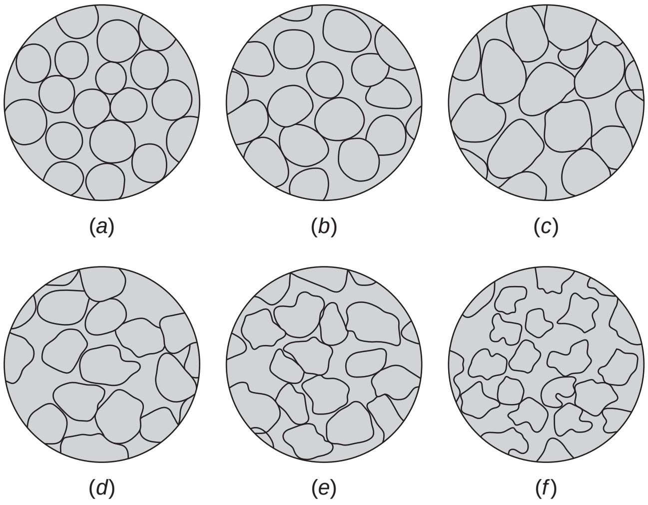


Figure 16.3 Sphericity and shape factors of granular (not flake-like) materials and typical porosities associated with them in stratified rapid sand-filter beds (After Fair et al. 1971).

of solid material, but on the ratio of surface area to volume of void space, A/V_v. This ratio is the reciprocal of the familiar hydraulic radius. For a bed or layer of perfect spheres of diameter d, the area–voids volume ratio, A/V_v, of equal spheres within a given cube of porosity f can be calculated from the number of spheres, n. The following simple relationships hold true: n = (total volume of spheres)/(volume of a single sphere); $A = n$ (surface area of a single sphere) = $n\pi d^2$; V (total volume of spheres) = $(1-f)$ (volume of given cube) = 1/6 $n\pi d^3$; volume of given cube = $V/(1-f)$; and V_v = volume of voids = f (volume of given cube) = $[f/(1-f)]$ V, or

$$\frac{A}{V_v} = \frac{n\pi d^2}{1/6[f/(1-f)]n\pi d^2} = \frac{1-f}{f}\left(\frac{6}{d}\right) \quad \textbf{(16.2)}$$

16.4 PREPARATION OF FILTER SAND

Natural *run-of-bank* sand may be too coarse, too fine, or too nonuniform for a projected filter. Within economical limits, specified sizing and uniformity can be obtained by *screening out* coarse grains and washing out fines.

If a filter sand is specified in terms of effective size and nonuniformity coefficient and a sieve analysis of the stock sand has been made (Fig. 16.2), the coarse and fine portions of stock sand to be removed are functions of P_{10} and P_{60}, namely, the percentages of stock sand smaller than the desired effective size and the 60 percentile size, respectively. This can be explained as follows:

1. Because the sand lying between the P_{60} and P_{10} sizes constitutes half the specified sand, the percentage of usable stock sand is

$$P_{\text{usable}} = 2(P_{60} - P_{10}) \quad \textbf{(16.3)}$$

2. Because the specified sand can contain but one-tenth of the usable sand below the P_{10} size, the percentage below which the stock sand is too fine for use is

$$P_{\text{too fine}} = P_{10} - 0.1P_{\text{usable}} = P_{10} - 0.2(P_{60} - P_{10}) \quad \textbf{(16.4)}$$

provided that the grain size associated with the sand that is too fine is equal to or greater than the smallest size of sand to be included in the filter.

3. Because a percentage of stock sand equal to P_{usable} + $P_{\text{too fine}}$ has been accounted for, the percentage above which the stock sand is too coarse is

$$P_{\text{too coarse}} = P_{\text{usable}} + P_{\text{too fine}} = P_{10} + 1.8(P_{60} - P_{10}) \quad \textbf{(16.5)}$$

Before sand is placed in a filter, fines can be washed from it in a sand or grit washer. Washers of the same kind will also cleanse sand removed from a filter—a slow sand filter, for example—and relieve grit of organic matter.

EXAMPLE 16.2 CONVERSION OF STOCK SAND INTO FILTER SAND

What must be done to the stock sand of Fig. 16.2 to convert it into a filter sand of effective size 5×10^{-2} cm and nonuniformity coefficient 1.5?

Solution:

From Fig. 16.2 the proportion of sand, P_{10}, less than the desired effective size of 5×10^{-2} is 30% and the proportion less than the desired P_{60} size of $5 \times 10^{-2} \times 1.5$ cm is 53.5%. Hence by Eq. (16.3) the proportion of usable sand equals

$$\begin{aligned} P_{\text{usable}} &= 2(P_{60} - P_{10}) \\ &= 2(53.5 - 30.0) = 47.0\%. \end{aligned}$$

By Eq. (16.4) the percentage below which the stock sand is too fine is

$$\begin{aligned} P_{\text{too fine}} &= P_{10} - 0.1P_{\text{usable}} \\ &= 30.0 - 0.1 \times 47 \\ &= 25.3\% \end{aligned}$$

and the diameter of this sand from Fig. 16.2 is **4.4 × 10^{-2}** cm.

The percentage above which the stock sand is too coarse is by Eq. (16.5)

$$\begin{aligned} P_{\text{too coarse}} &= P_{\text{usable}} + P_{\text{too fine}} \\ &= 47.0 + 25.3 \\ &= 72.3 \end{aligned}$$

and the diameter of this sand is **1.10 × 10^{-1} cm** from Fig. 16.2.

It follows that all stock sand finer than 4.4 × 10^{-2} cm at one end of the size range and coarser than 1.10 × 10^{-1} cm at the other end must be removed.

16.5 HYDRAULICS OF FILTRATION

At the velocities commonly employed in granular water filters, flow is normally laminar and obeys Darcy's law, $v = Ks$, where v is the face or approach velocity of the water above the sand bed, $s = h/L$ is the loss of head, h, in a depth of filter bed, L, and K is Darcy's coefficient of permeability (Fig. 16.4). Identifiable components of Darcy's K are the density, ρ, and viscosity, μ, of the water, the porosity, f, of the bed, and the size and shape of the component sand grains that establish the surface area A of the grains within the bed relative to their volume, V.

Specifically, the resistance of a clean bed of sand to the filtration of clean water is given by the Blake–Kozeny equation:

$$h/L = (k/g)(\mu/\rho)\, v\, \frac{(1-f)^2}{f^3}(A/V)^2 \qquad \textbf{(16.6)}$$

Here, after all recognizable factors have been introduced into Darcy's coefficient of permeability, k (Kozeny) becomes a residual dimensionless coefficient that assumes a magnitude close to 5.0 under most conditions of water filtration. The porosity factor, $(1-f)^2/f^3$, derives in part from the conversion of the approach velocity term, v, into an interstitial velocity term v/f, and in part from identification of the hydraulic radius, r, of the interstitial channels as

r = cross-sectional area of flow
× length of channel/wetted perimeter of channel
× length of channel
= volume of water in interstices/surface area of sand

Because the volume of water in the interstices is f (pore volume of the bed) and the bed volume is $V/(1-f)$,

$$r = \frac{f}{1-f}\,\frac{V}{A} = \frac{f}{1-f}\,\frac{\psi d}{6} \qquad \textbf{(16.7)}$$

In terms of the size and shape of its constituent sand grains, therefore, the rate of head loss becomes

$$h/L = (5/g)\nu v\frac{(1-f)^2}{f^3}(6/\psi d)^2 \qquad \textbf{(16.8)}$$

or

$$h/L = (5/g)\nu v\frac{(1-f)^2}{f^3}(S/d)^2 \qquad \textbf{(16.9)}$$

Here the kinematic viscosity ν has replaced the ratio μ/ρ.

Figure 16.4 Head loss in filtration and expansion in backwashing (After Fair et al., 1971).

16.5.1 Hydraulics of Stratified Beds

In a clean filter, stratified by backwashing, the head loss is the sum of the losses in successive sand layers. Because the thickness of each of n layers is closely proportional to the fractional weight, p_i, of sieved size d_i, Eq. (16.8), as proposed by Fair and Hatch (1933), becomes

$$h/L = (5/g)\nu v\frac{(1-f)^2}{f^3}(6/\psi)^2 \sum_{i=1}^{n} (p_i/d_i^2) \quad \textbf{(16.10)}$$

Here the diameter, d_i, is an average diameter such as the geometric mean diameter or square root of the product of the upper and lower sieve sizes representing a fraction p_i of the analyzed sand. Normally the values of pertinent sizes and fractions are, respectively, those of the adjacent sieves employed and the proportion, p_i, of the sand trapped between them. However, it is possible to read usable values also from the plotted analysis (Fig. 16.2).

EXAMPLE 16.3 HEAD LOSS IN A STRATIFIED SAND BED

Assume that the straight line drawn in Fig. 16.2 describes the size distribution by manufacturer's rating of filter sand in a 24 in. (610 mm) bed operated at a rate of 3 gpm/ft^2 (0.2 cm/s); that the bed after backwashing settles back uniformly to a porosity of 0.4 (porosity is calculated from measurements of depths, L, weight of sand per unit area of filter, W, and sand-specific weight, γ_s, as $f = 1 - W/(L\gamma_s)$; and that the sand has a sphericity of 0.8. Assume a water temperature of 20°C. Find

- **a.** The head lost in the clean bed
- **b.** The head lost if the particles in the bed are all of the geometric mean size
- **c.** The Darcy coefficient of bed permeability

Solution 1: (US Customary System):

a. By Eq. (16.10):
From Appendix 3, ν at 20°C = 1.01 centistokes = 1.01×10^{-2} cm^2/s
and from Table 16.3, $\sum_{i=1}^{n} (p_i/d_i^2 = 233)$.

$$h/L = (5/g)\nu v\frac{(1-f)^2}{f^3}(6/\psi)^2 \sum_{i=1}^{n} (p_i/d_i^2)$$

$= (5/981)\ 0.2 \times 1.01 \times 10^{-2}[(1-0.4)^2/0.4^3](6/0.8)^2 \times 233$
$= 0.76$, and
$h = 2 \times 0.76 =$ **1.52 ft.**

Table 16.3 Computation of $\sum_{i=1}^{n} (p_i/d_i^2)$for Example 16.3

Size of sand, cm × 10^2	Sand larger than stated size,[a] %	Sand fraction within adjacent sieve sizes, p_i, %	Geometric mean diameter, d_i, cm × 10^2	$\frac{p_i}{d_i^2}$
3	0.0			
		2	3.5	16
4	2.0			
		8	4.5	40
5	10.0			
		17	5.5	56
6	27.0			
		23	6.5	52
7	50.0			
		20	7.5	36
8	70.0			
		20	8.9	25
10	90.0			
		10	11.8	8
14	100.0	—		—
Sum	—	100	…	233

Source: After Fair et al. (1971).

[a] It is assumed that the filter does not contain sand smaller than 0.03 cm or larger than 0.14 cm.

b. For a geometric mean size of 7×10^{-2} cm, find
$1/d_i^2 = (7 \times 10^{-2})^{-2} = 204$, and
$h/L = 0.76 \times 204/233 = 0.67$, making
$h = 2 \times 0.67 =$ **1.33 ft** compared with 1.52 ft.

c. Calculate the Darcy coefficient of permeability as
$K = v/(h/L)$
$= 0.2/0.76 =$ **0.26 cm/s**
$= 0.10$ in/s.

Solution 2 (SI System):

a. From Appendix 3, ν at 20°C = 1.01 centistokes = 1.01×10^{-2} cm^2/s and from Table 16.3, $\sum_{i=1}^{n} (p_i/d_i^2 = 233)$

$$h/L = (5/g)\nu v\frac{(1-f)^2}{f^3}(6/\psi)^2 \sum_{i=1}^{n} (p_i/d_i^2)$$

$= (5/981)\ 0.2 \times 1.01 \times 10^{-2}[(1-0.4)^2/0.4^3](6/0.8)^2 \times 233$
$= 0.76$, and
$h = 610 \text{ mm} \times 0.76 = 463.6$ mm.

b. For a geometric mean size of 7×10^{-2} cm, find
$1/d_i^2 = (7 \times 10^{-2})^{-2} = 204$, and
$h/L = 0.76 \times 204/233 = 0.67$, making
$h = 610 \times 0.67 = 408.7$ mm compared with 463.6 mm.

c. Calculate the Darcy coefficient of permeability as
$K = v/(h/L) = 0.2/0.76 =$ **0.26 cm/s.**

16.5.2 Hydraulics of Unstratified Beds

In an unstratified bed of homogeneously packed sand each component fraction, p_i, of size d_i contributes its share to the total area, the individual area–volume ratio being $6/(\psi d_i)$. For uniform sphericity, therefore,

$$A/V = 6/\psi \sum_{i=1}^{n} \frac{p_i}{d_i} \tag{16.11}$$

and

$$h/L = (5/g)\nu v\frac{(1-f)^2}{f^3}[(6/\psi) \sum_{i=1}^{n} p_i/d_i]^2 \tag{16.12}$$

EXAMPLE 16.4 HEAD LOSS IN AN UNSTRATIFIED SAND BED

If the natural sand of Fig. 16.2 is used in an unstratified slow sand filter 42 in. (1,067 mm) deep operated at 3 MG/acre/d = 3.24×10^{-3} cm/s, the water temperature is 68°F (20°C) or $\nu = 1.01$ centistokes (Appendix 3), the porosity is 0.35, and the sphericity is 0.8, find

a. The initial head loss following cleaning and replacement of all the sand

b. The single diameter, d_e, that would produce the same hydraulic result as the mixed bed and where this size lies in the cumulative weight scale of Fig. 16.2

c. The Darcy coefficient of permeability

Solution 1 (US Customary System):

a. Using the sizes and percentages shown in Fig. 16.2, find $\sum_{i=1}^{n} p_i/d_i$ (see table below).
From the table, $\sum_{i=1}^{n} p_i/d_i = 17.84$.
By Eq. (16.12), find

$$h/L = (5/g)\nu v\frac{(1-f)^2}{f^3}\left[(6/\psi) \sum_{i=1}^{n} p_i/d_i\right]^2$$

$= (5/981)\ (3.24 \times 10^{-3} \times 1.01 \times 10^{-2})[(1-0.35)^2/0.35^3]\ [(6/0.8) \times 17.84]^2$
$= 2.9 \times 10^{-2}$ and
$h = 2.9 \times 10^{-2} \times 42/12 =$ **0.10 ft.**

Size of sand (cm × 10^2)	Sand smaller than stated size (%)	Sand fraction within adjacent sieve sizes, p_i (%)	Geometric mean diameter, d_i (cm × 10^2)	p_i/d_i
0.5	0			
1.05	0.2	0.2	0.72	0.28
1.49	0.9	0.7	1.25	0.56
2.10	4.0	3.1	1.77	1.75
2.97	9.9	5.9	2.50	2.36
4.2	21.8	11.9	3.53	3.37
5.9	39.4	17.6	4.98	3.53
8.4	59.8	20.4	7.04	2.90
11.9	74.4	14.6	10.0	1.46
16.8	93.3	18.9	14.1	1.34
23.8	96.8	3.5	20.0	0.18
33.6	100	3.2	28.3	0.11
Sum		100		17.84

b. Determine the single equivalent size by equating $1/d_e$ to $\sum_{i=1}^{n} p_i/d_i$
or $d_e = 1/\sum_{i=1}^{n} p_i/d_i$
$= 1/17.84$
$= \mathbf{5.6 \times 10^{-2}}$ **cm.**
From Fig. 16.2, the amount of sand smaller than 5.6×10^{-2} cm is **37%.**

c. Find the Darcy coefficient of permeability as
$K = v/(h/L)$
$= 3.24 \times 10^{-3}/(2.9 \times 10^{-2})$
$=$ **0.11 cm/s**
$= 0.043$ in./s.

Solution (SI System):

a. Using the sizes and percentages shown in Fig. 16.2, find $\sum_{i=1}^{n} p_i/d_i$ (see table).
From the table, $\sum_{i=1}^{n} p_i/d_i = 17.84$.
By Eq. (16.12), find

$$h/L = (5/g)\nu v\frac{(1-f)^2}{f^3}[(6/\psi)\sum_{i=1}^{n} p_i/d_i]^2$$

$= (5/981)\,(3.24 \times 10^{-3} \times 1.01 \times 10^{-2})[(1-0.35)^2/0.35^3]\,[(6/0.8) \times 17.84]^2$
$= 2.9 \times 10^{-2}$ and
$h = 2.9 \times 10^{-2} \times (1067 \text{ mm}) = 31$ mm.
The remaining SI solutions of (b) and (c) are identical to the US customary solutions of (b) and (c).

16.6 HYDRAULICS OF FLUIDIZED BEDS—FILTER BACKWASHING

The operation of granular water filters is discontinuous. As described earlier, output is stopped before an excessive loss of head is built up in the clogging bed or the quality of the product water—its clarity, for instance deteriorates. Output is not resumed until the bed has been backwashed thoroughly. The rate of backwash is made high enough (a) to fluidize the active portion of the bed and (b) to open the passageways between adjacent grains wide enough to allow floc and other residues deposited in the bed during filtration to escape with the overflowing washwater.

Conceptually, the hydraulics of fluidized beds differs from the hydraulics of filtration principally by the increase in pore volume of the expanded (lifted) sand. With the materials and degrees of expansion ordinarily used, however, the flow in a fluidized filter bed is laminar even though the suspended grains are in motion.

If a grain is to be kept in suspension, the frictional drag of the water rising past it must equal the pull of gravity on it; that is, the difference or head loss, h, between the bottom and top of any layer of thickness, L_e, must equal the weight in water of the suspended material. For each unit area of filter,

$$h\rho g = L_e(\rho_s - \rho)g(1 - f_e) \qquad \textbf{(16.13)}$$

Table 16.4 Values of $1/(1-f_e)$ corresponding to values of $f_e{}^3/(1-f_e)$ ranging from 0.1 to 9.9

$\frac{f_e^3}{1-f_e}$	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.00	1.62	1.89	2.10	2.28	2.44	2.59	2.74	2.88	3.01
1	3.14	3.27	3.40	3.52	3.65	3.78	3.89	4.01	4.13	4.24
2	4.35	4.47	4.58	4.70	4.81	4.93	5.05	5.16	5.27	5.38
3	5.49	5.60	5.71	5.82	5.92	6.03	6.14	6.24	6.35	6.46
4	6.57	6.68	6.78	6.88	6.99	7.10	7.20	7.31	7.41	7.52
5	7.62	7.73	7.83	7.94	8.04	8.15	8.25	8.35	8.46	8.56
6	8.67	8.77	8.88	8.98	9.08	9.18	9.29	9.39	9.49	9.60
7	9.70	9.81	9.91	10.01	10.11	10.21	10.32	10.42	10.52	10.62
8	10.72	10.83	10.93	11.03	11.14	11.24	11.35	11.45	11.56	11.66
9	11.76	11.86	11.96	12.06	12.16	12.27	12.37	12.47	12.58	12.68

Source: After Fair et al. (1971).

where f_e is the porosity ratio of the expanded layer of thickness, L_e. Hence

$$(h/L_e) = [(\rho_s - \rho)/\rho](1 - f_e) \tag{16.14}$$

In terms of Eq. (16.6), therefore,

$$h/L_e = \frac{\rho_s - \rho}{\rho}(1 - f_e) = \frac{k_e}{g}\nu v\frac{(1-f_e)^2}{f_e^3}[(6/\psi)(1/d)]^2 \tag{16.15}$$

Here k_e is a function of f_e. By experiment it assumes a value of about 4 when the bed is just fluidized. If the ith layer consists of grains of size d_i, Eq. (16.15) reduces to

$$\frac{f_{ei}^3}{1-f_{ei}} = \frac{k}{g}\frac{\mu}{\rho_s - \rho}v(6/\psi d_i)^2 \tag{16.16}$$

For such a layer, the ratio of the expanded depth, L_{ei}, to the unexpanded depth, L_i, is

$$\frac{L_{ei}}{L_i} = \frac{(1-f)}{(1-f_{ei})} \tag{16.17}$$

and the total expansion is

$$L_e = \sum L_{ei} = \sum \frac{L_i(1-f)}{1-f_{ei}} \tag{16.18}$$

If the porosity ratio, f, of the unexpanded bed is constant for all layers, the unexpanded thickness of any layer, L_i, equals the full depth, L, times the percentage, p_i, of diameter d_i, and L_e becomes

$$L_e = L(1-f)\sum_{i=1}^{n}\left[\frac{p_i}{(1-f_{ei})}\right] \tag{16.19}$$

The reciprocal of $(1-f_{ei})$ can be evaluated from the ratio $f_{ei}{}^3/(1-f_{ei})$ in Eq. (16.16) with the assistance of a table such as Table 16.4, or of a plot of $1/(1-f_{ei})$ against given magnitudes of $f_{ei}{}^3/(1-f_{ei})$.

Alternatively, it has been shown by experiment that the ratio of the measured settling velocity of a grain, v_s, to the face or hypothetical, viscous, settling velocity, v, equals about $(1/f_e)^5$, and hence that

$$v_s \approx v/f_e^{5.0}, v \approx v_s/f_e^{5.0}\ f_e \approx (v/v_s)^{0.20} \text{ and } f_e^{5.0} \approx (v/v_s) \tag{16.20}$$

The settling velocity, v_s, must be either measured by test or calculated by first obtaining a value v from Eq. (15.9) (Chapter 15) and an appropriate sphericity from Fig. 16.3 for introduction into Eq. (15.7) (Chapter 15) before computation of v_s. Associated values of f_e are given by Eq. (16.20).

EXAMPLE 16.5 FILTER BED EXPANSION

Given the sand sizes and size frequencies shown in Cols. 3 and 5 of Table 16.5, calculate for a face velocity of 25 in./min, an unexpanded filter depth of 28 in., a porosity of 0.45, and a grain sphericity 0.78, the bed expansion at water temperature of (a) 25°C and (b) 4°C, respectively, and (c) check the results obtained by applying the approximate method of calculation.

Table 16.5 Data and computations for Example 16.5

Sand characteristics					Expansion at 25°C			Expansion at 4°C			Approximation at 25°C		
Sieve mesh, per in.	Mfgr's rating, cm × 10^2	d_i cm × 10^2	$d_i^{-2} \times 10^{-2}$	$p_i \times 10^2$	$\frac{f_{ei}^3}{1-f_{ei}}$	$\frac{1}{1-f_{ei}}$	$\frac{p_i}{1-f_{ei}}$	$\frac{f_{ei}^3}{1-f_{ei}}$	$\frac{1}{1-f_{ei}}$	$\frac{p_i}{1-f_{ei}}$	f_{ei}	$\frac{1}{1-f_{ei}}$	$\frac{p_i}{1-f_{ei}}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
80	1.75												
		1.91	26.4	0.1	3.64	6.18	0.006	6.40	9.08	0.009	0.864	7.35	0.007
65	2.08												
		2.48	16.2	2.9	2.23	4.62	0.134	3.90	7.46	0.187	0.781	4.57	0.133
48	2.95												
		3.22	9.65	8.5	1.38	3.50	0.298	2.34	4.74	0.402	0.707	3.41	0.290
42	3.51												
		3.82	6.85	11.0	0.93	3.05	0.335	1.66	3.96	0.435	0.660	2.94	0.326
35	4.17												
		4.54	4.85	21.5	0.67	2.69	0.580	1.18	3.37	0.725	0.613	2.58	0.555
32	4.95												
		5.40	3.42	31.0	0.47	2.39	0.742	0.83	2.92	0.907	0.574	2.35	0.729
28	5.89												
		6.42	2.42	13.5	0.33	2.15	0.290	0.59	2.57	0.347	0.536	2.16	0.294
24	7.01												
		7.64	1.71	9.0	0.236	1.97	0.177	0.415	2.30	0.207	0.497	1.99	0.179
20	8.83												
		9.09	1.21	1 .8	0.167	1.82[a]	0.033	0.294	2.09	0.037	0.487	1.95	0.035
16	9.91												
		10.76	0.86	0.7	0.120	1.82[a]	0.013	0.209	1.91	0.013	0.487	1.95	0.014
14	11.68						2.608			3.269			2.562

Source: After Fair et al. (1971).

[a] $1/(1 - f_e)$ of unexpanded bed is $1/(1 - 0.45) = 1.82$. This is the smallest possible value. Sand larger than the 20 mesh sieve size is not lifted at the washwater rate and the temperature (25°C) obtaining in the example.

Solution:

a. By Eq. (16.16),

$$\frac{f_{ei}^3}{1-f_{ei}} = \frac{k}{g}\frac{\mu}{\rho_s-\rho}v(6/\psi d_i)^2$$
$$= \frac{4}{981}\times\frac{0.895\times10^{-2}}{2.65-0.997}\times\frac{25\times2.54}{60}(6/0.78d_i)^2$$
$$= 1.38\times10^{-3}/d_i^2$$

Hence Column 6, Table 16.5, is 1.38×10^{-3} × Column 4. Column 7 is taken from Table 16.4. Column 8 is Column 7 × Column 5. By Eq. (16.19), therefore,

$$L_e = L(1-f)\sum_{i=1}^{n}\left[\frac{p_i}{(1-f_{ei})}\right]$$
$$= 28(1-0.45)(2.608) = 40 \text{ in.}$$

and the percentage expansion is

$$100L_e/L = 100\times40/28 = \mathbf{143}\%.$$

b. In proportion to μ in Eq. (16.16),

$$\frac{f_{ei}^3}{1-f_{ei}} = 1.38\times10^{-3}/d_i^2$$
$$= (1.38\times10^{-3}/d_i^2)(1.57\times10^{-2})/(0.895\times10^{-2})$$
$$= 2.43\times10^{-3}/d_i^2.$$

Hence, as in the first instance,

$$L_e = L(1-f)\sum_{i=1}^{n}\left[\frac{p_i}{(1-f_{ei})}\right]$$
$$L_e = 28(1-0.45)(3.269) = 50 \text{ in.}$$

and the percentage expansion is

$$100\,L_e/L = 100\times50/28 = \mathbf{180\%.}$$

c. For each value of d_i, Eq. (15.10a) (Chapter 15) gives the hypothetical viscous (hindered) settling velocity, v_{oi}', as

$$v_{oi}' = (g/18)[(\rho_s - \rho)/\mu]d_i^2$$
$$= (981/18)[(2.65–0.997)/0.895 \times 10^{-2})]d_i^2$$
$$= 10^4\ d_i^2.$$

From Eq. (16.1),

$$\psi = d_o/d_s = \sqrt{v_s/v_0}$$
$$v_{si} = v_{oi}'\ \psi^2$$
$$v_{si} = 10^4\ d_i^2\ \psi^2 = 6.1 \times 10^3\ d_i^2.$$

By Eq. (16.20),

$$f_e \approx (v/v_s)^{0.20}$$
$$f_{ei} = (v/v_{si})^{0.20}$$
$$= [(25 \times 2.54/60)/(6.1 \times 10^3 d_{si}^{\ 2})]^{0.20}$$

Columns 12, 13, and 14 show the results of calculations.

By Eq. (16.19),

$$L_e = L(1-f)\sum_{i=1}^{n}\left[\frac{p_i}{(1-f_{ei})}\right]$$
$$= 28(1 - 0.45)(2.56)$$
$$= 39.4\text{in.}$$

and the percentage expansion is

$100\ L_e/L = 100 \times 39.4/\ 28 =$ **141%**, which is not significantly less than the value of 143% computed in the first instance.

Example 16.5 illustrates how strongly water temperature affects sand expansion during backwashing. Accordingly, it is necessary to vary the flow rate in line with the temperature or to measure and control expansion during washing. When the bed is fully fluidized, there is no further increase in head loss, and the rate of upflow can be kept constant thenceforward. A washwater rate controller can regulate flow automatically.

16.7 REMOVAL OF IMPURITIES

The overall removal of solid impurities is brought about in various ways. Which one of them exerts the controlling influence depends on circumstances. At the beginning of a filter run, fine-grained, unstratified filters act primarily as strainers. Slow sand filters are examples. They normally accumulate most of the impurities at the sand surface and in the top 2 in. or so of sand. Large microbic populations, derived in the first instance from the applied water, multiply and flourish in the accumulating organic matter, nutrients, or *Schmutzdecke* (from the German Schmutz, dirt or impurity, and Decke, cover or layer). To clean these simple structures in simple ways, the top sand and its Schmutzdecke are scraped off and removed or washed in place.

By contrast, well-designed and well-operated rapid filters, especially multilayered filters, are intended to remove impurities, principally flocculated impurities, down to the last layer of active grains that is cleared of accumulated floc when the bed is washed.

Most granular water filters remove particulates smaller than the passages between adjacent grains. This can be explained by the concept of a filter as possessing, in the aggregate, a relatively huge surface area or interface in contact with the water and its impurities that pass by. Surface forces are brought into play among them *van der Waals* forces that bind particles to the surfaces even though they may bear the same electrical charge as the filter grains. Other attachment mechanisms may involve electrostatic interaction, chemical bridging, and adsorption.

The substances removed during filtration are distributed irregularly, that is, by chance, over the grain surfaces and generally are not dislodged by the passing water. Interstices, however, are narrowed down by accumulating deposits, and some of them are undoubtedly closed. Particulates entering pores still open are then transported deeper into the bed, until they reach grain sites still able to accept them. Only if they fail to find sites of this kind do they escape into the effluent. In fluidized beds the grain surfaces are scoured clear of deposits by repeated abrasive contact between moving grains. At the water velocities engendered during filtration even in partially clogged interstices there is no scour as such and no consistent downward displacement of filter deposits because of scour.

16.8 KINETICS OF FILTRATION

As a rate process, filtration shares the common elements of purification kinetics. This was shown by Iwasaki in 1937. Since then the basic purification equation, $-dy/dt = k\ (y_0 - y)$ has been expanded by Ives (1960, 1965) to include important factors in filter behavior. Among them is the identification of filtration kinetics as the kinetics of a transient operation responsible for progressive changes in the rate constant k during the course of a filter run. Ives (1960) has expressed the changes as follows:

$$k = k_o + c\sigma - \varphi\sigma^2/(f_o - \sigma) \qquad \textbf{(16.21)}$$

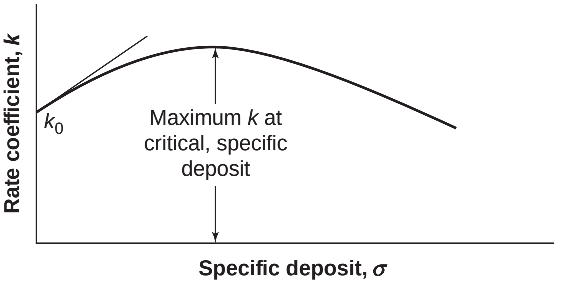


Figure 16.5 Rise and fall in rate coefficient k with specific deposit in a filter run (After Fair et al., 1971).

where k_o is the initial rate constant, which, along with the coefficients c and φ, describes a specific exposure; f_o is the porosity of the clean bed; and σ is the specific deposit or volume of deposited matter per unit filter volume. Equation (16.21) is not entirely empirical. The second term depends on the increase in specific surface by initial deposits. The third term depends on the increase in interstitial velocity caused by the accumulation of deposits and the accompanying straightening of flow paths in pores and diminution of specific surface. As sketched in Fig. 16.5, k rises to a maximum value during a breaking-in or maturation period. After that it falls consistently. During the rise the second term of Eq. (16.21) is dominant; after that the third term, which derives in essence from a materials balance in which input minus output must equal the volume of deposit stored, that is,

$$\partial\sigma/\partial t = -v(\partial c/\partial L) \qquad \textbf{(16.22)}$$

Here t is the time from the start of filtration, c is the concentration of suspended matter passing a given filter depth L, and v is the rate of filtration per unit cross-sectional area of filter.

The resulting equations describe the distribution of suspended matter in the water and the specific deposit σ through a depth L, with time t, as a function of k_o, c, ϕ, f_o, and v as determined in pilot tests that involve sampling at successive depths and times.

16.9 FILTER DESIGN

The dimensioning of filters and their appurtenances depends on entrant water quality; filter type; process and hydraulic loading; method and intensity of cleaning; nature, size, and depth of filtering materials; and prescribed quality of product water. Unless pilot-plant tests provide information on (a) the observed responses of a specific water to treatment in a plant being designed for this purpose and (b) the associated responses of the filter or filters examined, it is common to start with design values that have stood the test of experience and to modify these values in the light of new information and wanted objectives.

Some filters are expected to be no more than polishing units that separate residual floc from well-coagulated, well-settled water. Such filters can be relatively shallow, coarse-grained, high-rate units producing a clear effluent that is readily and reliably amenable to disinfection. Roughing filters—so called because they only prepare water for further filtration—may also be of this design. Placed in advance of slow sand filters, they have been operated with or without coagulation. By contrast, filters are normally designed as relatively deep, fine-grained, low-rate units if they are intended to (1) offer an effective barrier to waterborne pathogens, (2) treat water containing much floc, or (3) serve as reacting units for floc formation and removal after dosage of the applied water with a coagulant. Characteristic design values are shown in Table 16.1 for water filters and discussed in Section 16.2 for intermittent sand filters.

Even for waters of fairly constant composition, it is not yet possible to prescribe unique or optimizing combinations of filter depth, layered arrangements of filtering media (size, shape, and density), and rates of filtration. It is doubtful, too, whether designers would wish to dimension units narrowly. Raw-water quality is bound to change in time as well as season; preparatory processes are certain to be improved if necessary; and demands on plant productivity are sure to be raised concurrently. Thus, *built-in flexibility* becomes an essential design requirement.

Performance objectives to be reconciled are terminal head loss, standards of effluent quality (for instance, turbidity), and length of filter run. As grain size is increased, head loss drops markedly at upper levels, but effluent turbidity increases relatively rapidly. As rates of filtration are upped, head loss and turbidity rise relatively fast at all filter levels. In coarse-grained units, filter runs ended at a given head loss are relatively longer than in fine-grained units, but filter runs ended at a given turbidity are relatively shorter.

16.9.1 Bed Depth

The design depth of rapid filters is generally related to grain size, terminal head loss, and terminal turbidity or a suitable surrogate measure of terminal effluent quality. In an analysis of bed performance, Hudson (1959) concluded that inadequately filtered water breaks through rapid filters when

$$Qd^3h/L = B \quad \text{(US customary system)} \qquad \textbf{(16.23a)}$$

Here Q is the rate of filtration in gpm/ft^2 of filter, d is the sand size in centimeters, h is the terminal loss of head in ft, L is the depth of bed in inches, and B is a breakthrough index that assumes the magnitudes shown in Table 16.6 for different influent waters at 50°F (10°C). The equivalent equation using the SI units is presented below:

$$2.05Qd^3h/L = B \quad \text{(SI system)} \qquad \textbf{(16.23b)}$$

Here Q is the rate of filtration in L/min/m^2 of filter, d is the sand size in centimeters, h is the terminal loss of head in m, L

Table 16.6 Magnitude of B the breakthrough index in Eq. (16.23a) for different influent waters at 50°F (10°C)

Response to coagulation	Degree of pretreatment	Value of B
Poor	Average	4×10^{-4}
Average	Average	1×10^{-3}
Average	High	2×10^{-3}
Average	Excellent	6×10^{-3}

Source: After Fair et al. (1971).

is the depth of bed in mm, and B is a breakthrough index that assumes the magnitudes shown in Table 16.6 for different influent waters at 10°C.

For temperatures other than 50°F (10°C), viscosity effects can be allowed for by multiplying the breakthrough index by $60/(T + 10)$ when using the US customary units, or by $60/(1.8T + 42)$ when using the SI units.

Multilayered filters, referred to as *mixed-media* filters, simulate countercurrent operation by being composed of successive layers of coarse but light filter grains on top of increasingly finer but heavier particles. Such filters must preserve their layered structure during backwashing and resettling. To this purpose, the light grains of largest size in an upper layer must rise higher and settle more slowly than the heavy grains of smallest size in the layer next below.

Equal expansion during backwashing is identified by Eq. (16.16), and equal rate of settling by calculations involving Eq. (16.20), because settling is hindered. In accordance with these equations,

$$d_u/d_L = (\psi_L/\psi_u)[(\rho_L - \rho)/(\rho_u - \rho)]^{1/2} \qquad \textbf{(16.24)}$$

where the subscripts u and L, respectively, denote the largest grains within the upper layer and the smallest grains within the lower layer.

It follows that mixing during settling as well as during expansion determines the maximum allowable ratio of the grain sizes in the two layers. Because the conditions of flow and the specific shape of filter grains are normally uncertain, a value of d_u/d_L smaller than that from Eq. (16.24) would be employed.

EXAMPLE 16.6 REQUIRED MINIMAL DEPTH OF A SAND BED THAT WILL PREVENT BREAKTHROUGH

Water receiving an average degree of pretreatment and responding in average fashion to coagulation is to be filtered at a rate of 3 gpm/ft^2 (122 L/min/m^2) through a layer of sand grains 0.1 cm in diameter. Find the requisite minimal depth of sand that will prevent breakthrough of turbidity at a terminal loss of head of 8 ft (2.44 m).

Solution 1 (US Customary System):

From Table 16.6 the expected value of the breakthrough index is 1×10^{-3}. From Eq. (16.23a)

$Qd^3h/L = B$

$L = Qd^3h/B$

$L = (3)(0.1)^3(8)/(1 \times 10^{-3})$

$=$ **24 in.**

Solution 2 (SI System):

From Table 16.6 the expected value of the breakthrough index is 1×10^{-3}. From Eq. (16.23b)

$2.05\ Qd^3h/L = B$

$L = 2.05\ Qd^3h/B$

$L = 2.05\ (122)(0.1)^3(2.44)/(1 \times 10^{-3})$

$=$ **610 mm.**

EXAMPLE 16.7 MAXIMUM ALLOWABLE RATIO OF THE GRAIN SIZES IN MIXED-BED FILTERS

A layer of crushed anthracite with a sphericity of 0.70 and a density of 1.50 is to rest on a layer of sand with a sphericity of 0.80 and a density of 2.65. Find the maximum ratio of the diameter of the coarsest anthracite to the finest sand that will ensure both equal expansion and equal settling of the two materials at the common boundary. Assume that the density of the water is 1.00.

Solution:

By Eq. (16.24)

$d_u/d_L = (\psi_L/\psi_u)[(\rho_L - \rho)/(\rho_u - \rho)]^{1/2}$

$= (0.80/0.70)[(2.65 - 1.00)/(1.50 - 1.00)]^{1/2}$

$= 2.1$ for equal expansion and settling.

Therefore the grain–diameter ratio in the two layers must be less than 2.1.

16.9.2 Underdrainage Systems

Underdrainage systems of rapid filters perform two primary functions:

1. They collect the filtrate and send it on its way to a clearwell or pure water reservoir.
2. They distribute washwater to the bed during scouring operations.

Because the rate of wash is many times that of filtration, the hydraulics of underdrains is governed by upflow requirements. If these are met, downflow distribution should be satisfactory. A secondary responsibility of underdrainage systems is to withdraw to waste chemical solutions added to filter beds from time to time, (a) to break up, loosen, and remove incrustants accumulated on filter grains and (b) to break up mudballs formed and built up near the cleavage plane between supporting grains and filtering and expanding grains.

Two types of underdrainage systems are in use in rapid filters: (a) pipe grids and (b) filter floors or false bottoms.

16.9.2.1 Pipe Grids and Filter Gravel In their simplest form, pipe grids comprise a main, called a *manifold*, and perforated laterals (Fig. 16.1). Perforations are normally drilled into the laterals in a single row of orifices directed vertically downward or in two rows as pairs of orifices directed downward at angles of 45° on either side of the vertical (see Fig. 16.6). To protect them against corrosion, pipe grids are normally lined with cement or bitumastics and well coated on the outside. Their walls are generally so thick that water issuing from the orifices expands within the perforations, which then have a coefficient of discharge nearer 0.75 than 0.6.

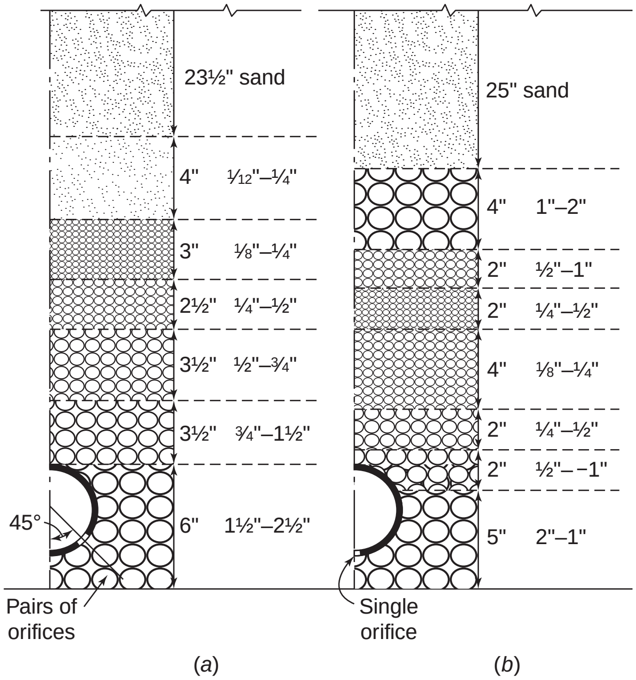


Figure 16.6 (a) Asymmetrical and (b) symmetrical sequences of gravel below a sand bed (After Fair et al., 1971). Conversion factors: 1′ = 1 ft = 0.3048 m; 1″ = 1 in. = 25.4 mm.

Because the jets are broken up when they strike the filter bottom or the gravel surrounding the grid, the head loss equals the full driving head. This is important. Made large enough in reference to the system's head loss as a whole, the orifice loss can overshadow the remaining losses in magnitude. The hydraulic performance of the system then becomes much the same above each orifice and relatively uniform over the entire bed. In practice, the *controlling head loss* or driving head is set between 3 and 15 ft (0.91–4.57 m). This makes for a ratio of orifice area to bed area of $(3/60)\,(0.75\sqrt{2gh}) = 0.5$–$0.2\%$ when wash rate rise is 36 in./min (91.44 cm/min).

Other useful calculations for hydraulic equilibrium are (a) that the allowable friction loss, h_{f}, in a lateral with n orifices must equal $(1-m^2)$ times the driving head, h_{d}, on the first orifice reached by the washwater if the head on the last or nth orifice is to be no more than m^2 times the driving head (Eqs. 15.26 and 15.27),

$$h_n = m^2 h_1$$
$$h_1 = h_{\mathrm{f}}/(1-m^2)$$

and (b) that, in terms of lost head, and in the absence of side effects, the friction in a lateral of unvarying size is numerically equal to the friction exerted by the full incoming flow in passing through one-third the length of the lateral (Section 13.8). For $m = 0.9$, for example, $h_{\mathrm{f}} = (1-0.81)\,h_{\mathrm{d}} = 0.57$–$2.85$ ft (0.17–0.87 m). Hydraulic studies of pipe grids must also cover entrance losses and losses in fittings. Reductions in flow by takeoff at orifices, moreover, produce hydraulic transients similar to the transients associated with sudden enlargement of flow channels. In accordance with Borda's loss function, the reduction in velocity and recovery of velocity head produces a net head recovery at the ith orifice equal to

$$[(v_i^2/2g) - (v_{i+1})^2/2g)] - (v_i - v_{i+1})^2/2g = 2v_i v_{i+1}/2g \tag{16.25}$$

where v_i and v_{i+1} are the pipe velocities on approach to the ith and $(i+1)$th orifices, respectively. It follows that there may be a net gain, not a net loss, in head as the washwater travels down the lateral if $(1/g)\Sigma v_i v_{i+1} > 1/3$ Ls,that is, the pipe friction.

As a rule, hydraulic systems are optimized when 25% of the overall energy loss is incurred in delivering required flows to the control points. In the case of pipe grids, the orifices are these points. Therefore the unused 75% of the total energy remains to be expended in driving the washwater in succession through the orifices, gravel, and fluidized bed.

Rules of thumb that make it possible, on first trial, to lay out a reasonably well-balanced underdrainage system for filters washed at rates of 6–36 in./min follow:

1. Ratio of area of orifice to area of bed served: [(1.5 to 5) $\times 10^{-3}$]:1.
2. Ratio of area of lateral to area of orifices served: (2 to 4):1.
3. Ratio of area of main to area of laterals served: (1.5 to 3):1.
4. Diameter of orifices: 1/4–3/4 in (6–19 mm).
5. Spacing of orifices: 3–12 in. on centers (75–300 mm).
6. Spacing of laterals: closely approximating spacing of orifices.

Twinned units halve washwater rate requirements of individual filters by washing the two component units in succession.

Perforated pipe grids are only one part of an underdrainage system. The other part comprises the stone and gravel that surround the grid and support the sand bed. This system of collecting and distributing waterways is seldom less than 10 (250 mm) or more than 24 in. (600 mm) thick. For particles sized by screening, the depth, L, in inches, of a component gravel layer of size d in., where $d > 3/64$ in., may be estimated from the following equation:

$$L = k(\log d + 1.40) \quad \text{(US customary system)} \quad \textbf{(16.26a)}$$

Here k varies numerically from 10 to 14. An equivalent equation using the SI system is presented below:

$$L = 25.4k[\log(d/25.4) + 1.40] \quad \text{(SI system)} \quad \textbf{(16.26b)}$$

where L is the depth, in mm, of a component gravel layer of size d mm, assuming $d > 1.19$ mm, and k varies numerically from 10 to 14.

Stones as large as 3 in. (75 mm), but generally no larger than 2 in. (50 mm), are placed near the pipes. To keep the screened gravel in place during backwashing, it should be carefully packed. Indeed, the larger gravel sizes should be packed into the filter by hand (Fig. 16.6).

Because the behavior of fine gravel and coarse sand in a stratified bed is much the same, this arrangement of supporting gravel is by no means as stable as the designer may want it to be. For this reason, some works have introduced the arrangement illustrated in Fig. 16.6. Here the progress is symmetrically from coarse to fine to coarse. At the sand–gravel interface, coarse sand slips into the interstices of the upper half, but there is no harm in this. The supporting gravel layer maintains itself no matter what the rate of washwater rise is, within reason. Effective depths of gravel are given by Eq. (16.26a) with the following modifications: (a) all but the increments in depth of the largest gravel are halved; (b) the bottom halves together with the unhalved depth of the largest gravel increased by 1 in. (25 mm) are put in place; and (c) the other half of the gravel, together with the unhalved depth of the largest gravel, is superimposed on the bottom half (Fig. 16.6b).

If overall filter depths must be kept small, pipe perforations and gravel can be replaced by nozzles with slots or holes small enough to exclude filter grains but large enough in the aggregate to deliver and accept required flows. There is danger of galvanic corrosion when dissimilar metals are used for the nozzles and the piping.

16.9.2.2 *Filter Floors*

Filter floors, also called *false bottoms* or *false floors*, are intended to replace pipe grids and serve two functions:

1. To support the filter bed, possibly without stone and gravel in transitional layers below the filter bed proper
2. To create a single, boxlike waterway beneath the filter as the dispenser of washwater and the collector of filtered water (Fig. 16.7)

The floor, depending on its thickness, is perforated by short tubes or orifices of such dimension as to introduce the *controlling loss of head* that will make for even distribution of washwater. The openings must be relatively small and closely spaced, and their jets must be broken up by discharging from an effective nozzle or into a suitable depth of gravel. In some designs inverted-square pyramids are cast

EXAMPLE 16.8 DETERMINATION OF GRAVEL LAYERS IN A FILTER BED

A sand bed is to be supported on gravel 1/10–$2\frac{1}{2}$ in. in size. Find the requisite depths of component gravel layers.

Solution:

Equation (16.26a) and a value of $k = 12$ establish the following sequences for sieve ratios of 2:1:

Size (in.)	5/64	5/32	5/16	5/8	$1\frac{1}{4}$	$2\frac{1}{2}$
Depth (in.)	3.5	7.1	10.7	14.4	17.0	21.6
Increment (in.)	3.5	3.6	3.6	3.7	3.6	3.6
Chosen depth (in.)	$3\frac{1}{2}$	$3\frac{1}{2}$	$3\frac{1}{2}$	$3\frac{1}{2}$	$3\frac{1}{2}$	$4 + 2 = 6$

Accordingly, the **total depth is $23\frac{1}{2}$ in**. (Compare with Fig. 16.6a.)

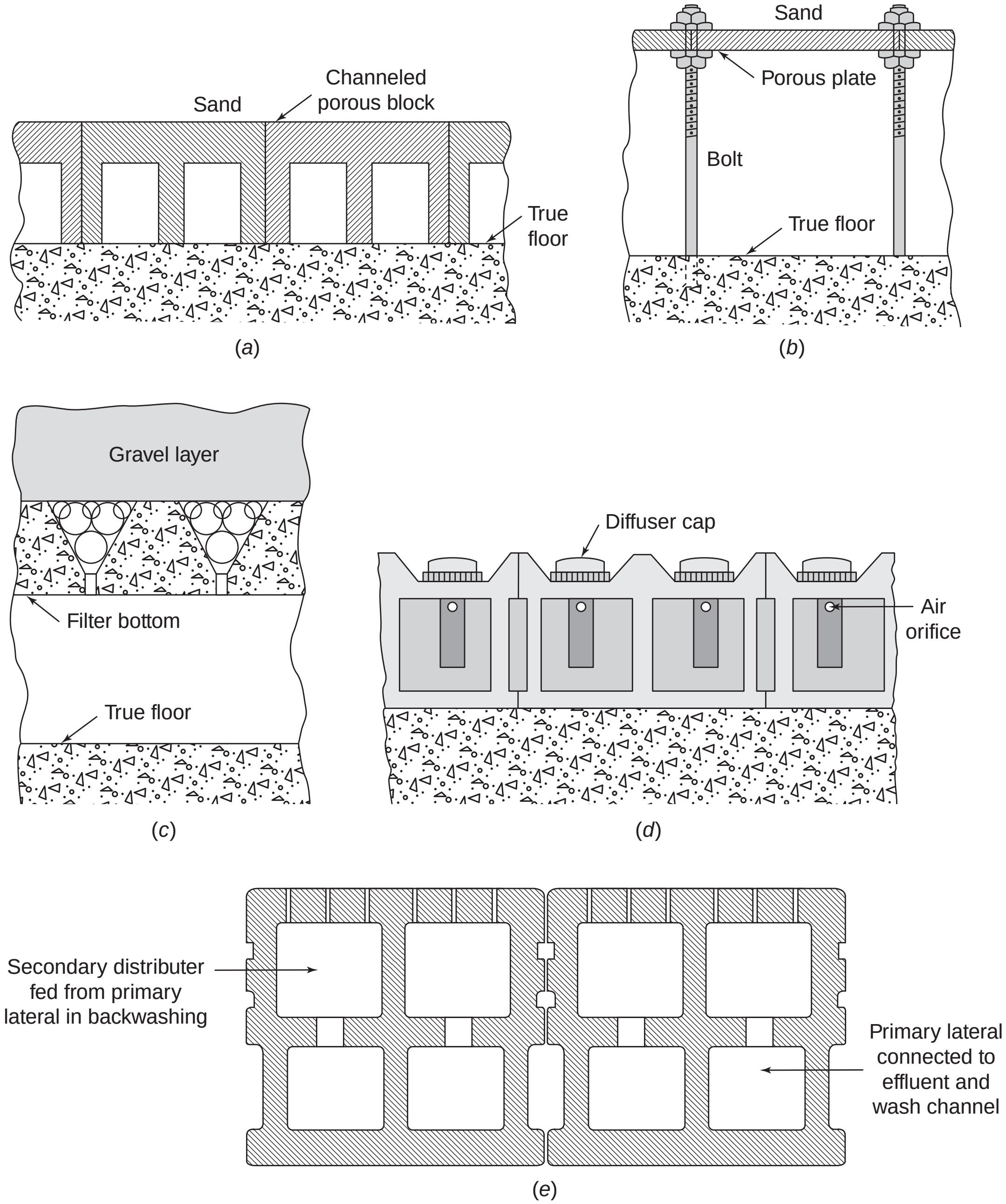
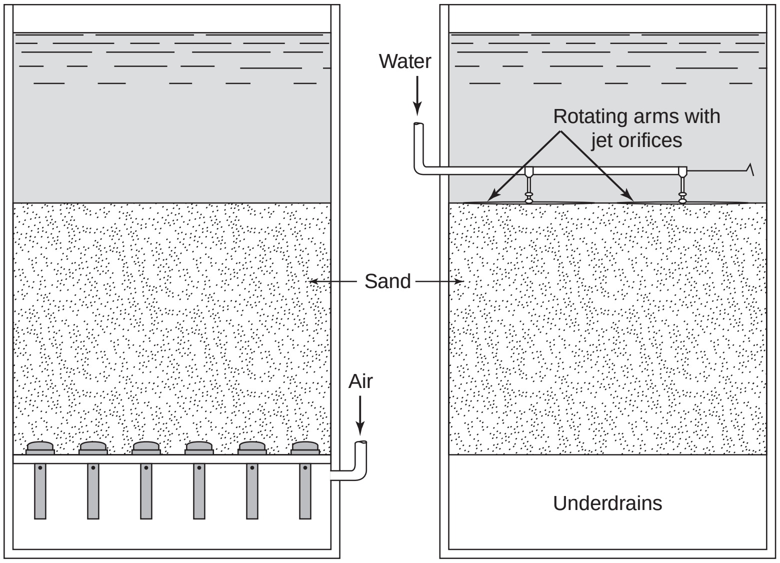


Figure 16.7 Filter floors. (a) channeled porous block; (b) porous plate floor; (c) wheeler bottom; (d) camp filter underdrain (combination air/water backwash—Walker process); (e) Leopold duplex filter bottom (After Fair et al., 1971).

into the false floor. Large and small spheres within these geometric depressions force the rising jets to spread out. In other designs a checkerboard of porous plates or perforated plates with nozzle strainers is supported on bolts or beams that are anchored to the true floor or bottom of the filter box. Similar porous plates are also employed to diffuse air into water and wastewater. Channel blocks that create continuous waterways and are set directly on the true bottom may take the place of plates. Graded gravel is not required if porous plates or nozzles with fine openings are used. Many proprietary underdrain systems are available.

Iron and lime are common clogging precipitates. Others are the result of after-precipitation of coagulants and suspended matter. These reach the false floor from poorly cleaned or otherwise poorly operated beds. Slime growths are troublesome on occasion. However, it is generally possible to restore the porosity of clogged plates and blocks with acid or alkali washes.



Air scour or combination air/water

Surface wash

Figure 16.8 Auxiliary scour intensification in rapid filters (After Fair et al., 1971).

16.9.3 Scour Intensification

How fluidized beds can be agitated in order to intensify scour of their filter grains is sketched in Fig. 16.8. Only one of the methods there illustrated, namely, auxiliary *surface wash*, has been incorporated into North American plants for some time. By contrast, European engineers continue to favor auxiliary air scour as such, because it saves water. Mechanical scour has found little use since the early days of rapid sand filtration, then called *mechanical filtration*. However, other designs are made possible by individualized power sources. Unlike air and water, mechanical rakes and related devices do not seek out paths of least resistance. Instead, they destroy points of resistance.

Surface jets operate under pressures of 50–75 psig (347–520.5 kPa gauge). They add an equivalent of 3–5 in./min (76–127 mm/min) to the rate of wash, which is otherwise only large enough to fluidize the bed. The jets should strike more or less horizontally into the boiling sand just beneath the topmost reach of the expanded maximum rise. Necessary nozzles are attached to fixed pipe grids or to moving pipe arms.

The rate of air scour is generally 3–5 $\text{ft}^3/\text{min}/\text{ft}^2$ (0.91–1.52 $\text{m}^3/\text{min}/\text{m}^2$) of filter. Normally this air flow is maintained for several minutes before the bed is ready for hydraulic expansion and coincidental removal of deposits loosened from the filter grains and freed from the pore space during air scour. The air is usually supplied from a pipe grid laid just above the gravel line or from air channels or domes incorporated in a general-purpose, perforated pipe underdrainage system or filter floor. Elements of air-grid design are much the same as for water grids. In both instances a controlling loss of head equal to about 75% of the total head supplied to the washing system will normally make for economy of design.

16.9.4 Washwater Troughs

Troughs spanning filters and receiving the spent washwater are intended to keep the upwelling water from establishing a significant slope to its surface and impressing a head differential on the washwater distribution system. The aim is an even washwater rate over the whole bed surface. Troughs are not common in European practice.

Hydraulically the troughs are not unlike those serving the effluent weirs of settling tanks and the side spillways of dams. A general relationship for the water–surface curve within washwater troughs and similar structures can be based on the momentum theorem by making the following simplifying assumptions: (a) the kinetic energy of the water falling into the trough does not contribute to the longitudinal (displacement) velocity; (b) channel friction can be neglected in all but very long channels; (c) flow is essentially horizontal; and (d) the water–surface curve approximates a parabola. The forces changing the momentum then derive solely from the unbalanced static pressure forces P_1, P_2, and P_3 shown in Fig. 16.9. They have the following magnitudes:

$$P_1 = \tfrac{1}{2}\gamma b h_0{}^2$$
$$P_2 = -\tfrac{1}{2}\,\gamma b h_L{}^2$$
$$P_3 = \gamma bLi\,(2/3h_0 + 1/6iL + 1/3h_L)$$

because, volume of water in the trough $= bL[h_0 + iL - 1/2iL - 1/3(h_0 + iL - h_L)]$.

Here γ is the unit weight of water; b, L, and i are, respectively, the width, length, and invert slope of the trough; and h_0 and h_L are, respectively, the initial and terminal depths of the water. The change in momentum, $Qv_L\gamma/g$, equals $wbh_c{}^3/h_L$, because $Q = bh_c v_c$, $v_L = h_c v_c/h_L$, and $v_c = \sqrt{gh_c}$. Here Q is the rate of discharge, and v_L and v_c are the velocities of flow at the submerged depth h_L and the critical depth h_c,

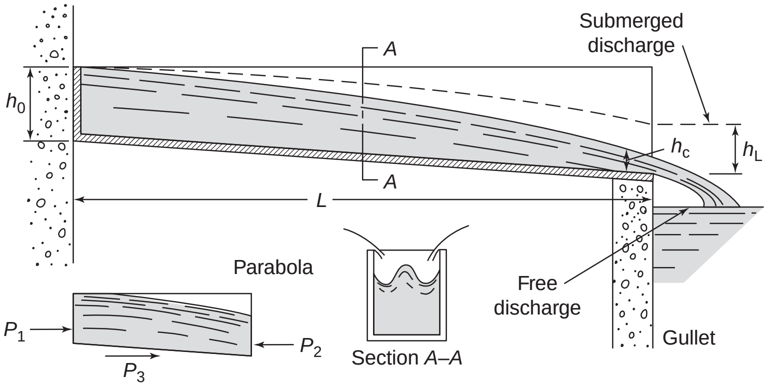


Figure 16.9 Water-surface curves in a washwater gutter (After Fair et al., 1971).

respectively. Equating the sum of the forces to the change in momentum and solving for h_0,

$$h_0 = \sqrt{\left(\frac{2h_c^3}{h_L}\right) + \left(h_L - \frac{1}{3}iL\right)^2} - \frac{2}{3}\,iL \quad \textbf{(16.27)}$$

For level inverts ($i = 0$) and for the critical depth $h_c{}^3 = Q^2/gb^2$, where Q is the total rate of discharge and b is the width of a rectangular trough,

$$h_0 = \sqrt{h^2 + \frac{2Q^2}{g\,b^2\,h_t}} = \sqrt{h^2 + 2\frac{h_0^3}{h_L}} \quad \textbf{(16.28)}$$

When discharge is free, h_L closely equals h_c and

$$h_0 = h_c\sqrt{3} = 1.73h_c \quad \textbf{(16.29)}$$

or

$$Q = 2.49bh_0^{3/2\,2} \quad \text{(US customary system)} \quad \textbf{(16.30a)}$$

where the units are Q (ft^3/s), b (ft), and h_0 (ft). The equivalent equation using the SI system is presented in below:

$$Q = 1.37bh_0^{3/2} \quad \text{(SI system)} \quad \textbf{(16.30b)}$$

where the units are Q (m^3/s), b (m), and h_0 (m). Equations (16.28) and (16.29) hold also for troughs that are not rectangular in cross-section. In long troughs the frictional drawdown of the water surface can be approximated by assuming that turbulence increases the roughness factor about twofold.

In American practice lateral travel of the water overflowing into the troughs is commonly limited to between 2.5 and 3.5 ft (0.76 and 1.07 m); that is, the clear distance between gutters is held to between 5 and 7 ft (1.52 and 2.13 m) in order to keep the head of water on the underdrains, and, with it, the rate of wash, nearly uniform. The height of troughs above the sand bed is determined by the degree of sand expansion. The trough must lie above the surface of the expanded sand if it is not to reduce the waterway open to the sand–water mixture, increase its upward velocity, and cause sand loss.

Troughs run the length or width of the filter. They discharge into a gullet, the hydraulics of which is like that of the tributary troughs except that flow increases stepwise instead of uniformly. Choice of trough cross-section depends somewhat on the material of construction. Rectangular, semicircular, semihexagonal, and semioctagonal shapes are common. Externally the semicircle interferes least with the upward streaming of washwater. Internally it possesses the best hydraulic properties as well.

EXAMPLE 16.9 FILTER BED TROUGH DESIGN

Troughs 24 ft long, 18 in. wide, and 7 ft on centers (7.32 m long, 0.46 m wide, and 2.13 m on centers) are to serve a filter that is washed at a rate of 26 in./min (0.66 m/min). Find (a) the depth of the troughs if their invert is to be kept level and they are to discharge freely into the gullet and (b) the height of the lip of the trough if the bed below it contains a 30 in. (762 mm) bed of sand and is expanded 50%.

Solution 1 (US Customary System):

a. Find the flow, Q

$Q = 24 \text{ ft} \times 7 \text{ ft} \times [(26 \text{ in/min})/(12 \times 60)] = 6.1 \text{ ft}^3/\text{s}.$

From Eq. (16.30a),

$Q = 2.49bh_0{}^{3/2}$

$h_0 = [6.1/(2.49 \times 18/12)]^{2/3}$

$=$ **1.39 ft or, say, 17 in.**

b. It follows that the troughs must be placed above the sand by a distance of (0.50 × 30 in. + 17 in.) = 32 in. = **2.67 ft, plus the depth of freeboard in the trough and its thickness.**

Solution 2 (SI System):

a. Find the flow, Q

$$Q = 7.32 \text{ m} \times 2.13 \text{ m} \times [(0.66 \text{ m/min})/60] = 0.172 \text{ m}^3\text{/s}.$$

From Eq. (16.30b),

$$Q = 1.37bh_0^{3/2}$$

$$h_0 = [Q/(1.37 \times b)]^{2/3}$$

$$= [0.172/(1.37 \times 0.46)]^{2/3}$$

$$= \mathbf{0.42 \text{ m or } 420 \text{ mm.}}$$

b. It follows that the troughs must be placed above the sand by a distance of (0.50 × 762 mm + 420 mm) = 800 mm = 0.8 m**, plus the depth of freeboard in the trough and its thickness.**

16.9.5 Filter and Conduit Dimensions

The dimensioning of individual filter units in a plant of given size becomes a matter of economics once their number has been made large enough to cover cleaning operations and allow for occasional repairs. Essential factors are the cost of the filter proper, its walls, and appurtenances. Final sizings must be confirmed by comparative designs. As stated before, twinned units can be introduced to keep the size of the washwater system within useful working limits.

Filters must be large enough to allow for time out of service for cleaning and repair. Rapid filters are thrown out of operation for about 10 min during each cycle—normally 1 day (the time required to clean slow filters may be as much as 3 days in every 30-day period). Further allowances must be made for the amount of filtered water consumed in washing the beds, on the order of 3–5% of the plant output, and for occasional repairs. Including a freeboard of 1 ft (0.3 m), the depth of both slow and rapid filter units is commonly 10 ft (3 m).

Pipes and other conduits, including valves and gates, are ordinarily designed to carry water at velocities close to the following:

	ft/s	m/s
Influent conduits carrying raw water	3–6	0.91–1.83
Influent conduits carrying flocculated water	1–3	0.30–0.91
Effluent conduits carrying filtered water	3–6	0.91–1.83
Drainage conduits carrying spent washwater	4–8	1.22–2.44
Washwater conduits carrying clean washwater	8–12	2.44–3.66
Filter-to-waste connections	8–12	2.44–3.66

Where washwater tanks are used, they are made large enough to wash two filters in sequence. They must refill between washes. In large plants it is usually economical to pump washwater directly to the filters. In small plants washwater can be taken from distribution systems through pressure-reducing valves.

16.10 FILTER APPURTENANCES AND RATE CONTROL

Filtration is carried out in three different ways: (a) constant pressure filtration, (b) constant rate filtration, and (c) declining rate filtration.

Constant pressure filtration: In this method the total available pressure drop is applied throughout the filter run. At the beginning of the filter run the sand is clean, the permeability is high, and the filtration rate is high. As filtration runs its course, the filter begins to clog with solids and the filtration rate decreases with time. This method requires large volume of water storage and consequently it is not in common use.

Constant rate filtration: In this operation, the filter rate is held constant by means of a flow control valve that holds a constant pressure drop across the filter. As filtration proceeds, the filter clogs with solids, and the flow controller opens the valve slowly to maintain a constant flow rate. The disadvantages of this type of filtration are high initial and maintenance costs of the rate controller and effluent water quality that is inferior to that obtained using the declining rate method.

Declining rate filtration: This alternative filtration method is intermediate between constant pressure and constant rate operations. The influent to the filters enters below the low water level of the bank of filters through a large influent header and a large influent valve to each individual filter. Consequently, the head losses in the header and influent valves are small and do not restrict the flow, so the water level is essentially the same at all times in all filters. As the filters start to get clogged with solids the flow through the dirtiest filters decreases, automatically causing the cleaner filters to pick up the lost capacity. The water level in all filters rises slightly providing the additional head needed by the cleaner filters as they pick up the diverted flow. This method of operation causes a gradually declining rate near the end of

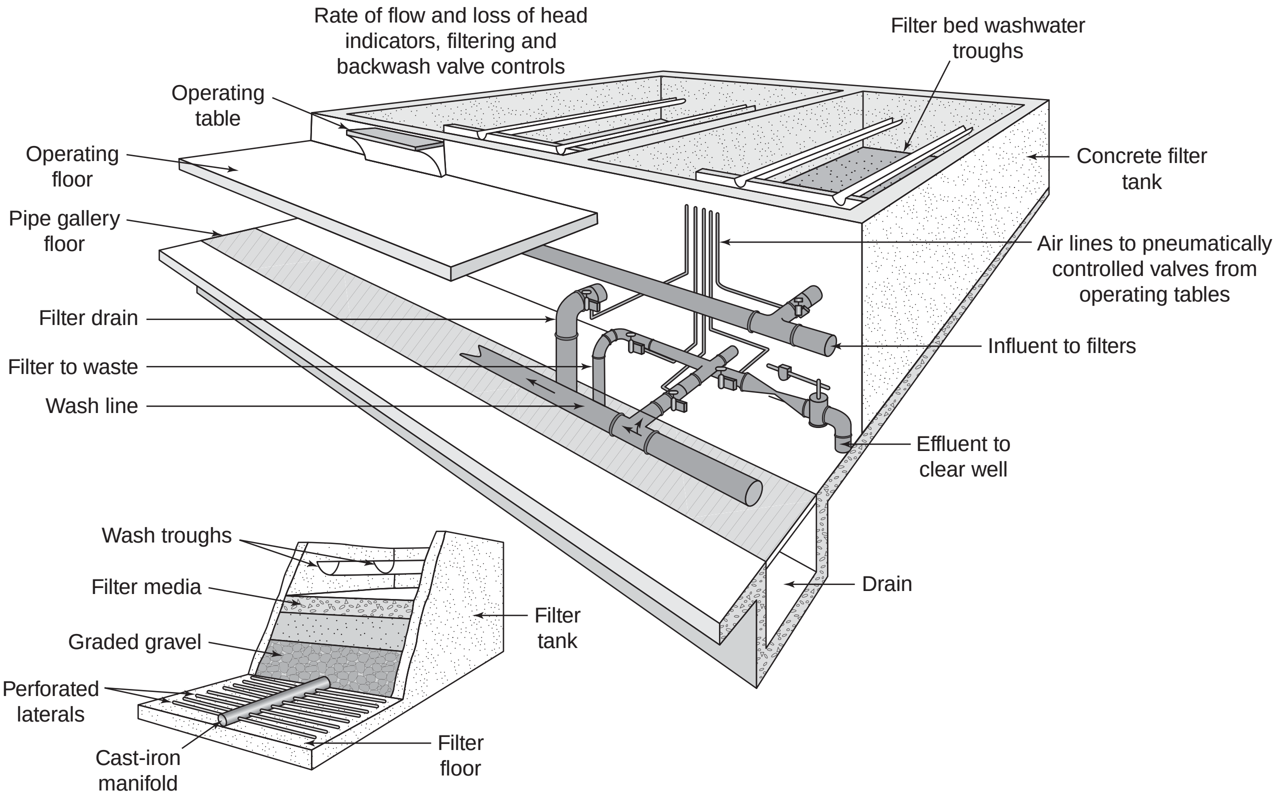


Figure 16.10 Rapid filters and accessory equipment (After Fair et al., 1971).

a filter run. As the dirty filters lose capacity, the load is redistributed among all of the filters. The resulting rate increase in each filter is very small and the water level on all filters rises very slowly until it approaches the maximum permissible water level. The filter with the longest time in service is backwashed, and when put back in service, that filter operates at the highest flow rate, so the water level on all filters declines to a new equilibrium level. The advantages of the declining rate operation over constant rate operation include significantly better filter effluent quality and less head loss.

Filter appurtenances include manually, hydraulically, pneumatically, or electrically operated sluice gates and valves on the influent, effluent, drain, and washwater lines (where valves are inserted into pipe lines, butterfly valves are economical in cost and space); measuring devices such as venturi meters; rate controllers activated by a measuring device; loss-of-head and rate-of-flow gauges; sand expansion indicators; washwater controllers and indicators; operating tables and water sampling devices; water quality monitoring devices; sand ejectors and washers; and washwater pumps and tanks. The larger the plant and the higher the rate of filtration, the more is the inclusion of mechanical and automatic operating aids justified. Flow regulators automatically open or close a valve mechanism to keep the discharge rate constant. Pressure differentials between the venturi throat and outlet are translated into valve movements by a balancing diaphragm or a piston.

Much of the equipment and piping of modern gravity filters are shown in Fig. 16.10. The *filter-to-waste* connection serves the purpose of wasting the water held in the filter after washing. It was called the *rewash connection* when filters were washed regularly with raw water. Today it is operated only when beds must be cleaned with deterging chemicals. The chemicals are usually drawn into the bed after distribution onto or over the surface, allowed to remain in contact with the grains until they are clean, and displaced from the bed through the filter-to-waste connection. The connection branches from the filter effluent in advance of the rate controller. Generally it is a valved pipe stub proportioned to discharge at about the maximum rate of filtration when the bed is clean. A waste connection of this kind should not endanger the normal filtrate by allowing polluted water to be drawn into the system.

16.11 LENGTH OF FILTER RUN

The impurities transferred from the applied water to the filter together with their coagulating and precipitating agents clog its pores and increase the hydraulic loss of head. The time rate at which head loss rises depends on sand size, porosity, filtration rate, and amount and character of the suspended matter in the applied water. Relationships between these factors are best determined in pilot tests. Filter runs are terminated either when the head loss exceeds a reasonable

Figure 16.11 Control room and schematics of the water purification plant to Lac de Bret, Switzerland (http://en.wikipedia.org/wiki/File:Usine_Bret_MG_1638.jpg).

value or when the filtered water shows an increase in turbidity, or *floc breakthrough*. For common terminal head losses and terminal turbidities, the length of filter runs is normally determined by buildup of head in fine-grained units and by buildup of turbidity in coarse-grained units. When head loss is the governing factor, length of filter run normally varies inversely as the product of the initial loss of head of the clean sand bed and the square root of the rate of filtration. Filter runs have been found to vary between 8 and 96 h, depending on influent quality, rate of flow, and media. Figure 16.11 shows the control room for a water filtration plant.

The terminal head loss is usually established at a value at which the bed and its underdrainage system come under partial vacuum or *negative head* (Figs. 16.4 and 16.12). Within the bed, the head becomes negative when the loss of head through the overlying bed depth exceeds the static head. Variations in suspended matter removal and filter grain size may cause negative heads to appear first at intermediate depths. Toward the end of their runs, when any part of the bed goes under a partial vacuum, dissolved air begins to be released. At atmospheric pressure and normal temperatures, water can hold in solution about 3% air by volume (principally oxygen and nitrogen). Because the amount of air precipitated from solution is about 100/34 = 3% per ft (9.65% per m) of negative head, it does not take long to fill the pore space of the filter or the volume of the underdrainage system. The filter then becomes air bound, head losses rise sharply, and filter output capacity drops rapidly.

16.12 FILTER TROUBLES

Rapid filters are subject to a variety of ailments:

1. Cracking of the bed
2. Formation of mudballs and plugging of portions of the bed
3. Jet action at the gravel–sand separation plane and sand boils
4. Sand leakage into the underdrainage system

Associated operating difficulties may become so great that the filter media and supporting gravel may have to be removed every 2 or 3 years, cleaned, reclassified, and replaced in proper order.

Troubles of this kind usually result from poor plant design and operation, particularly inadequate or disruptive backwashing. Possible yardsticks of relative dirtiness are the concentrations of alumina, iron, color, or turbidity freed from a known weight or volume of sand by shaking it vigorously

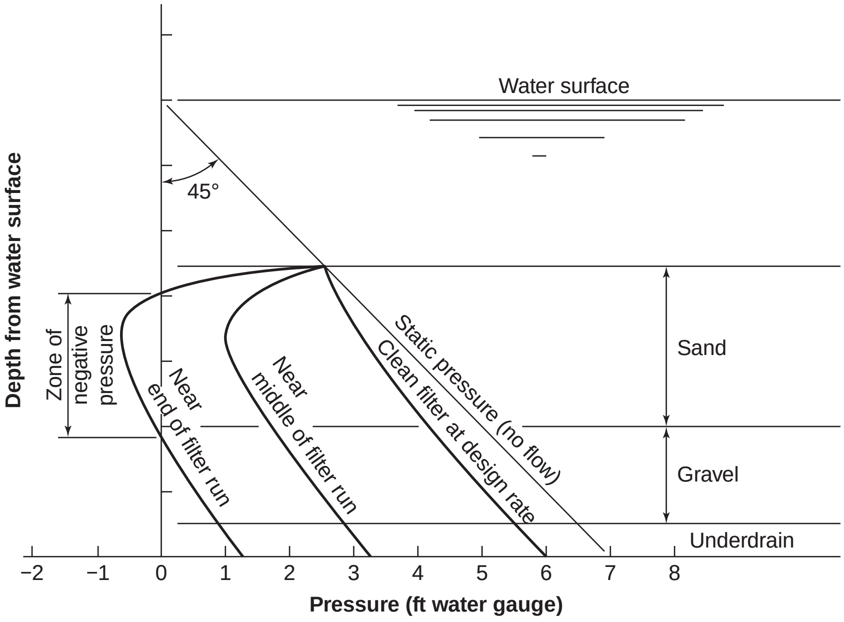


Figure 16.12 Pressure variation with depth of a rapid sand filter (*Source*: US EPA).

with a known volume of water to which a detergent or dissolving chemical has been added.

Badly clogged filters can be restored to usefulness by

1. Ejecting the sand and cleaning it in a sand washer
2. Agitating the expanded bed by hand with the help of long-tined rakes
3. Directing hose streams into the expanded bed
4. Adding a detergent such as a 2–5% solution of caustic soda ($\frac{1}{4}$–2 lb/ft^2 or 1.22–9.76 kg/m^2 of filter covered with 2–3 in. or 50–76 mm of water), draining it off through the filter-to-waste connection, and washing the bed clean.

Filter cracks: Because resistance to flow is least along the walls of a filter, less head is lost there, and resulting pressure differentials within the bed establish inward as well as downward flows; filter grains are pushed away from the walls; water short-circuits through resulting shrinkage cracks and fills them with caking suspended matter; and pressure differentials may open cracks also in the body of the bed. In coarse, well-compacted, clean sand, shrinkage is less than 1% at maximum head loss.

Mudballs: Mudballs are conglomerations of coagulated turbidity, floc, sand, and other binders. Mudball concentration can be measured by washing a known volume of sand through a 10 mesh sieve, transferring the retained conglomerations to a graduated cylinder partly filled with water, and noting the volume of displaced water. In well-designed and well-operated filters, mudballs should not occupy more than about 0.1% of the volume of the top 6 in. (152 mm) of sand.

Newly formed mudballs are small and light. They collect on the surface of the bed after it has been backwashed and can be removed by suction. Otherwise they should be broken up by air scour or destroyed by one of the methods listed in connection with filter clogging. As they grow in size, they increase in density by accretion. Eventually they become heavy enough to migrate downward as far as the gravel. There they accumulate and clog portions of the bed. Backwashing then becomes uneven, and gravel is lifted and mixed into the sand along the margin of the clogged volume. Associated deterioration in both filter output and effluent quality can be rectified only by rebuilding the filter.

Jetting and sand boils. Even small differences in porosity and permeability of sand and gravel cause the first flush of backwash water to follow paths of least resistance and break through to the surface at scattered points. Within the zone of flow, the clogged and compressed sand is fluidized, back pressure is reduced, flow is increased, and water is jetted from the gravel into the sand. If jetting becomes severe, the sand boils up like quicksand, and gravel as well as sand is lifted to the surface. If, in defense against these happenings, backwash valves are opened slowly, the bed is given an opportunity to disintegrate from the surface downward. Surface wash or air scour in advance of final bed fluidization can help to loosen a clogged bed, yet leave the gravel where it belongs.

Sand leakage. If both gravel layers and sand are properly sized and placed, sand cannot leak into the underdrainage system unless the layers of smallest gravel are displaced during backwashing.

16.13 PLANT PERFORMANCE

In practice, plant performance as a whole rather than the performance of specific portions of filter plants is normally identified. This is understandable, because it is the overall performance that establishes the quality of the product water. The factors affecting filtration include

1. Bed characteristics
    (a) Sand size
    (b) Sand shape
    (c) Surface properties of sand particles
    (d) Packing arrangement
    (e) Bed depth
    (f) Bed porosity
2. Suspension characteristics
    (a) Particle size
    (b) Particle shape
    (c) Particle density
    (d) Particle concentration
    (e) Surface properties
    (f) Water temperature
    (g) Chemical composition of water
3. Operational procedures
    (a) Flow rate
    (b) Backwash properties

The methods used in evaluating the above variables include

1. Time rate of clogging (head loss increase with time)
2. Bed penetration
3. Turbidity removal
4. Microorganisms removal
5. Length of filter run

16.13.1 Bacterial Efficiency

Modern, well-operated water treatment plants, in which effluent residuals of free chlorine are maintained, deliver a product water consistently free of coliform organisms, no matter how polluted the raw water may have been. At one time this was not so, even in large plants. Today it may still not be so in small plants in which supervision of chlorination and other operations is intermittent and deficient. Accordingly, the experience of earlier times remains of interest.

16.13.2 Removal of Color, Turbidity, and Iron

On an average only 30% of the natural color in water is removed by slow sand filters themselves. However, colorless water can be produced by both slow and rapid filters after suitable coagulation and settling and by rapid filters after the addition of polyelectrolytes immediately in advance of filtration. Turbidity responds well to slow sand filtration without the aid of coagulation. However, all but very small amounts of turbidity clog slow filters so quickly that waters with turbidities above about 40 units should not be applied to them.

Both slow and rapid sand filters remove oxidized or oxidizing iron and manganese. Manganese precipitates slowly and responds better to slow than to rapid filtration, unless it has been suitably coagulated. If the sand is coated with manganese, precipitation is hastened by catalysis.

16.13.3 Removal of Large Organisms

The large microorganisms, including the algae and diatoms, are readily removed by filtration, but the odors and tastes associated with them may pass through unchanged in intensity unless treatment processes adapted to their removal or destruction are included in the works. In the absence of turbidity, chlorophyllaceous organisms flourish on the surface of open filters. Mats of appreciable thickness may, indeed, build up on open slow filters. On sunny days, algal photosynthesis may release enough oxygen to lift sections of the mats and cause a rush of water through denuded spots. This makes for poor performance. The cells of diatoms interlock and form tenacious, often almost impenetrable, mats. This shortens filter runs.

The eggs and adults of the common intestinal parasitic worms, as well as the cysts of the pathogenic amebae, are relatively so large that they do not normally pass through beds of sand. By contrast, the cercariae of the blood flukes, although larger than amebic cysts, are sufficiently motile to wriggle through beds of sand of normal depth. Upward and downward currents created during scouring operations, moreover, may carry even large microorganisms deep into and through filter beds.

16.13.4 Oxidation of Organic Matter

Intermittent sand filters treating raw or settled wastewaters can remove bacteria and other particulates in acceptable degree. They can also oxidize organic matter quite completely. However, the rates of filtration at which good results are obtainable are woefully low. In particular circumstances biological oxidation of organic matter is a recognizable element in the purification of municipal water supplies.

PROBLEMS/QUESTIONS

16.1 Explain what is meant by effective size E and coefficient of nonuniformity U.

16.2 Explain how granular water filters can remove particulates smaller than the passages between adjacent grains.

16.3 Built-in flexibility is an essential design requirement for rapid sand filters. Why?

16.4 What are the functions of the underdrainage systems of rapid filters?

16.5 When do you terminate filter runs and have the filters backwashed?

16.6 How is the terminal head loss established in a rapid granular filter?

16.7 State the operational problems that could be encountered in rapid sand filtration and discuss the following issues:

(a) Nature of the problem

(b) Causes

(c) Prevention and control measures

16.8 How are badly clogged filters restored to usefulness?

16.9 What is the rate of filtration in MGD/acre (MLD/ha) for a plant operated as shown below?

Total water filtered in 1-day operation = 24 MG (91 ML)
Average hours of service per filter = 20
Number of filters used per day = 10
Area of each filter = 1,100 ft^2 (102 m^2)

16.10 A water treatment plant has a flow of 3 MGD (11.4 MLD). The plant has two identical rapid sand filters operated at a filtration rate of 2 gpm/ft^2 (81.4 L/min/m^2) and a backwash rate of 24 in./min (610 mm/min). Determine the required capacity of an elevated washwater tank needed to backwash one filter unit at a time. Assume a washing time of 6 min.

16.11 A water treatment plant has a flow of 6 MGD (22.7 MLD). The plant has 12 identical rapid granular filters operating at a filtration rate of 2 gpm/ft^2 (81.4 L/min/m^2) and a backwash rate of 30 in./min (762 mm/min). Compute the required capacity of an elevated water tank needed to backwash two filter units at a time. Assume a backwash time of 5 min per filter.

16.12 Determine the number of filter units needed to treat 5 MGD (19 MLD) of water given that the filtration rate is 5 gpm/ft^2 (203.5 L/min/m^2) and that each filter is to be 10 ft (3 m) wide and 20 ft (6 m) long.

16.13 For the size frequencies by weight of the sample of sand shown Table 16.7, determine

(a) The effective size E and coefficient of nonuniformity U

(b) The geometric mean size μ_g and geometric standard deviation σ_g

Table 16.7 Analysis of stock sand

Size of sand (mm)	Sieve analysis % retained by weight
0.15	0.04
0.28	0.60
0.36	1.10
0.42	4.25
0.50	14.80
0.60	27.15
0.70	21.20
0.83	12.65
1.02	6.02
1.23	4.05
1.59	3.50
1.75	2.75
2.50	1.28
3.75	0.65

16.14 What must be done to the stock sand of Table 16.7 to convert it into a filter sand of effective size 4×10^{-2} cm and nonuniformity coefficient 1.6?

16.15 Assume that the straight line drawn for the filter sand of Problem 16.14 (effective size 4×10^{-2} cm and nonuniformity coefficient 1.6) describes the size distribution by manufacturer's rating of filter sand in a 30 in. (762 mm) bed operated at a rate of 4 gpm/ft^2 (0.27 cm/s); that the bed after backwashing settles back uniformly to a porosity of 0.5; and that the sand has a sphericity of 0.7. Assume a water temperature of 10°C. Find

(a) The head lost in the clean bed

(b) The head lost if the particles in the bed are all of the geometric mean size

(c) The Darcy coefficient of bed permeability

16.16 If the natural sand of Problem 16.13 is used in an unstratified slow sand filter 48 in. deep operated at 2 MG/acre/d (2.16 cm/s $\times$ 10^{-3}), the water temperature is 68°F (20°C) or $\nu = 1.01$ centistokes $= 1.01 \times 10^{-2}$ cm^2/s (Appendix 3), the porosity is 0.50, and the sphericity is 0.7, determine:

(a) The initial head loss following cleaning and replacement of all the sand

(b) The single diameter, d_e, that would produce the same hydraulic result as the mixed bed and where this size lies in the cumulative weight scale of the sand in Problem 16.13

(c) The Darcy coefficient of permeability

16.17 Water receiving an average degree of pretreatment and responding in high fashion to coagulation is to be filtered at a rate of 4 gpm/ft^2 (162.8 L/min/m^2) through a layer of sand grains 0.15 cm in diameter. Find the requisite minimal depth of sand that will prevent breakthrough of turbidity at a terminal loss of head of 6 ft (1.83 m).

16.18 A layer of crushed anthracite with a sphericity of 0.60 and a density of 1.40 is to rest on a layer of sand with a sphericity of 0.85 and a density of 2.50. Find the maximum ratio of the diameter of the coarsest anthracite to the finest sand that will ensure both equal expansion and equal settling of the two materials at the common boundary. Assume that the density of the water is 1.00.

16.19 Troughs 30 ft long, 20 in. wide, and 8 ft on centers (9.14 m long, 0.51 m wide, and 2.44 m on centers) are to serve a filter that is washed at a rate of 30 in./min (762 mm/min). Determine

(a) The depth of the troughs if their invert is to be kept level and they are to discharge freely into the gullet

(b) The height of the lip of the trough if the bed below it contains a 24 in. (610 mm) bed of sand and is expanded 50%

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Chapter 17

Alternative and Membrane Filtration Technologies

17.1 INTRODUCTION OF FILTRATION TECHNOLOGIES

17.1.1 Filtration Overview

This chapter describes several available filtration technologies ranging from commonly used conventional water purification systems to new and emerging technologies. Many water supply systems had to add filtration to comply with the 1986 US Safe Drinking Water Act (SDWA) Amendments and the resultant Surface Water Treatment Rule (SWTR). This section provides overviews of the following filtration technologies:

1. Conventional treatment and direct filtration
2. Direct filtration (gravity and pressure filters)
3. Slow sand filtration
4. Package plants
5. Diatomaceous earth filtration
6. Cartridge filters
7. Membrane filters

Conventional treatment and direct filtration are the two most widely used filtration systems. Slow sand filters, package filtration, and diatomaceous earth filtration have broad applicability. While slow sand filters have been used for many decades around the world, they do not have an established record of performance with a large number of water systems. Membrane and cartridge filtration systems have lately become popular technologies because they have generally been used very successfully in treatment of drinking water. Package plants and slow sand filters are considered best suited for small systems with less than 0.44 m^3/s (10 MGD or 38 MLD) of flow.

17.1.2 Filtration Applications

Filtration is usually the final step in conventional treatment trains, although disinfection frequently follows filtration. Filtration systems are regarded as effective for removal of turbidity, suspended solids, and microbial contaminants. Microbial contaminants of special concern include coliform bacteria, *Cryptosporidium, Giardia lamblia*, enterovirus, and *Legionella*.

The Surface Water Treatment Rule (SWTR) requires that filters achieve turbidities of less than 0.5 NTU in 95% of the finished water samples and never exceed 5 NTU. Turbidity is a measure of suspended particles, which can include organic solids, viruses, bacteria, and other substances. Turbidity particles range in size from less than 1 to 100 microns.

The effectiveness of filtration systems is determined by their ability to remove microorganisms, turbidity, and color. Color is imparted to water supplies by organic material and can be removed by chemical coagulation. Once the color is coagulated and combined with the floc particles, color removal can be related to turbidity removal. The filtration systems discussed in this chapter are appropriate for raw water with varying characteristics. Table 17.1 contains the recommended upper limits for several influent variables, including coliform bacteria, turbidity, and color, for four common filtration technologies. Conventional treatment is clearly the most acceptable type of the filtration systems in the table because it includes flocculation and sedimentation clarification (or flotation clarification), which reduce turbidity before the water is actually processed by the filter. Diatomaceous earth filtration systems, which include little pretreatment, require high-quality influent.

Giardia lamblia and *Cryptosporidium* are of particular concern in drinking water supplies because they form cysts that cannot be effectively killed by traditional chlorine disinfection. Their effective removal can be attained only by filtration. Conversely, viral and bacterial pathogens are relatively easy to destroy with disinfection.

Figure 17.1 is a graph of the relationship between filter effluent turbidity and cyst removal efficiency after filtration. The graph clearly shows that filtered water with very low levels of turbidity, ranging below 0.1 NTU, contained almost

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Water Engineering: Hydraulics, Distribution and Treatment, First Edition. Nazih K. Shammas and Lawrence K. Wang.

Table 17.1 Generalized capability of filtration systems to accommodate raw water quality conditions

	Recommended upper limits for influent		
Treatment technology	Total coliforms (#/100 mL)	Turbidity (NTU)	Color (CU)
Conventional treatment	< 20,000	No restrictions	75
– With no predisinfection	< 5,000	No restrictions	75
Direct filtration	<500	< 7–14	< 40
Slow sand filtration	<800	< 10	< 5
Diatomaceous earth filtration	<50	< 5	< 5

Source: US EPA.

no cysts. The four filtration systems shown in Table 17.2 are very effective in removing *Giardia*; the only exception is direct filtration without coagulation, which does not effectively reduce turbidity. If water supplies containing *Cryptosporidium and Giardia* are not effectively coagulated, the cysts will pass through the entire treatment process, including the filters.

The viral removal efficiencies of four filter technologies found to be very effective are shown in Table 17.3. Of these processes, direct filtration provides the poorest removal of viruses, ranging from 90% to 99%. Table 17.4 summarizes all US Surface Water Treatment Rule (SWTR) compliance technologies for filtration.

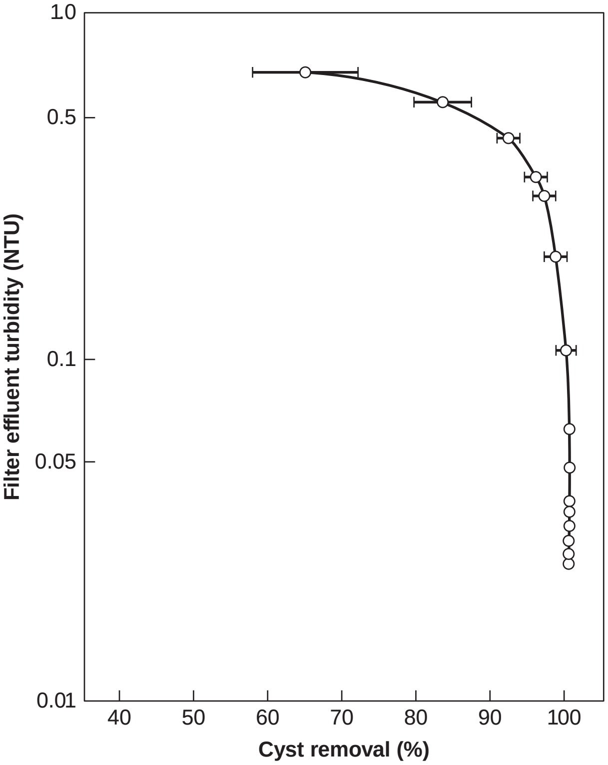


Figure 17.1 Relationship between cyst removal and filtered water quality.

Table 17.2 Removal efficiencies of *Giardia Lamblia* by water treatment processes

Unit process	Percent removal
Rapid filtration with coagulation, clarification[a]	96.9–99.9
Direct filtration	
– With coagulation	95.9–99.9
– Without coagulation	10–70
– With flocculation	95–99
Diatomaceous earth filtration	99–99.99[b]
Slow sand filtration	98–100[c]

Source: US EPA.

[a] Sedimentation or flotation clarification.

[b] Aided by coagulation.

[c] Fully ripened filter.

Table 17.3 Removal efficiencies of viruses by water treatment processes

Unit process	Percent removal
Slow sand filtration	99.9999
Diatomaceous earth filtration (with filter aid)	> 99.95[a]
Direct filtration	90–99
Conventional treatment	99

Source: US EPA.

[a] No viruses recovered.

17.2 DIRECT FILTRATION

Direct filtration systems are similar to conventional systems, but omit sedimentation. This section describes direct filtration technology.

17.2.1 Process Description

Direct filtration is an established technology that was developed because dual- and mixed-media filters are able to effectively process higher influent turbidities without the use of sedimentation. The direct filtration process is expected to be

Table 17.4 Surface Water Treatment Rule compliance technology for filtration

SWTR compliance technology table			Filtration
Unit technologies	Limitations (see footnotes)	Raw water quality range	Operator skill level required
Conventional filtration (includes dual-stage and dissolved air flotation)	[a]	Wide range	Advanced
Direct filtration (includes in-line filtration)	[a]	High quality	Advanced
Diatomaceous earth filtration	[b]	Very high quality or pretreatment	Intermediate
Slow sand filtration	[c]	Very high quality or pretreatment	Basic
Reverse osmosis filtration	N/A	Requires pre-filtration for surface waters	Advanced
Nanofiltration	N/A	Very high quality or pretreatment	Basic
Ultrafiltration	N/A	Very high quality or pretreatment	Basic
Microfiltration	N/A	High quality or pretreatment	Basic
Bag filtration	[d]	Very high quality or pretreatment	Basic
Cartridge filtration	[d]	Very high quality or pretreatment	Basic

Source: US EPA.
Note: New technologies in bold.
[a] Involves coagulation. Coagulation chemistry requires advanced operator skill and extensive monitoring. A system needs to have direct full-time access or full-time remote access to a skilled operator to use this technology properly.
[b] Filter cake should be discarded if filtration is interrupted. For this reason, intermittent use is not practical. Recycling the filtered water can remove this potential problem.
[c] Water service interruptions can occur during the periodic filter-to-waste cycle, which can last from 6 hours to 2 weeks.
[d] Site-specific pilot testing prior to installation of a bag or cartridge filter likely to be needed to ensure adequate performance.

more widely used on water supplies that, up until now, only performed chlorination.

Direct filtration is only applicable for systems with high-quality and seasonally consistent influent supplies. The influent generally should have turbidity of less than 5–10 NTU and color of less than 20–30 units.

Direct filtration consists of several combinations of treatment processes. It always includes coagulation and filtration, and sometimes includes flocculation or a contact basin after coagulation addition. Typical coagulant dosages range from less than 1 to 30 mg/L. Cationic polymers often successfully coagulate water supplies and assist direct filtration. Non-ionic polymers sometimes are added to the filtration step to increase filter efficiency. A flow diagram of a typical direct filtration system is shown in Fig. 17.2.

In-line direct filtration, the simplest version of direct filtration, is commonly used in pressure filtration systems (see Fig. 17.3). In this process, chemical coagulant application and mixing are followed by addition of a nonionic polymer aid and filtration. There is no separate flocculation step. This treatment is effective and commonly used in the removal of iron and manganese from groundwater when accompanied by appropriate coagulant doses. In-line direct filtration, however, is only applicable for surface waters with seasonal average turbidities of less than 2–3 NTU and free of contamination by wastewater sources (when permitted by regulatory agencies).

Another version of a direct filtration system is the "modified" system, which substitutes a contact basin for the flocculation basin found in the typical direct filtration process. Figure 17.4 is a flow diagram of modified direct filtration. The 1-h detention contact basin serves primarily to condition the floc prior to filtration. Contact basins also provide pretreatment by decreasing turbidity peaks in the influent, providing silt and sand removal, and allowing for more equal distribution of coagulant.

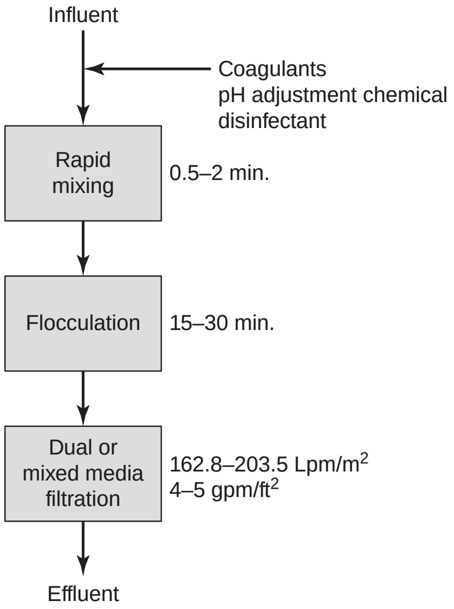


Figure 17.2 Flow diagram of typical direct filtration system.

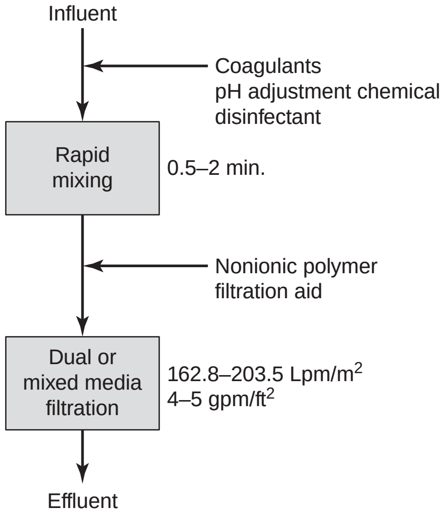


Figure 17.3 Flow diagram of in-line direct filtration system.

17.2.2 System Performance

Direct filtration units can perform effectively within the following general influent parameters: (a) less than 500 total coliforms per 100 mL, (b) less than 14 NTU of turbidity, and (c) less than 40 color units.

While the direct filtration process is able to operate satisfactorily with influent turbidities as high as 14 NTU, optimally, influent turbidity should be less than 5 NTU. Effective direct filtration performance ranges from 90% to 99% for virus removal and from 10% to 99.99% for *Giardia* and *Cryptosporidium* removal. The wide variation in direct filtration's *Giardia* and *Cryptosporidium* removal efficiencies is due to the wide range of available system configurations. The most effective direct filtration configurations for *Giardia* and *Cryptosporidium* removal must include coagulation and flocculation.

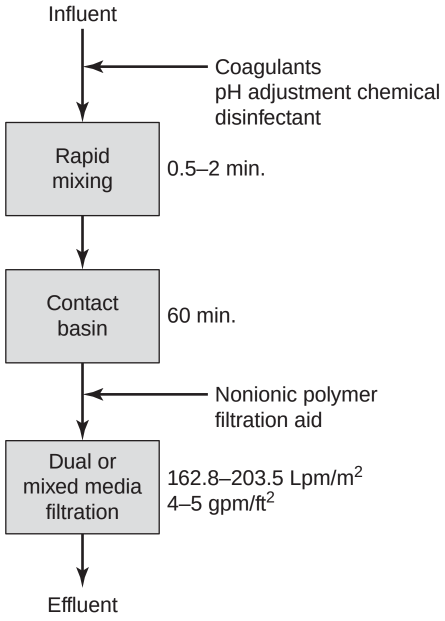


Figure 17.4 Flow diagram of modified direct filtration system.

17.3 SLOW SAND FILTRATION

17.3.1 Process Description

Slow sand filtration systems have a long history, having been used without disinfection at least since the 1850s in London, England. Slow sand filters were also commonly used in the United States in smaller facilities. Rapid sand filters have replaced many of them to accommodate the increase in water demand; consequently, slow sand filters are now not very common. According to the Surface Water Treatment Rule under the SDWA, many small systems could meet their regulatory filtration obligations with the simple slow sand filters. With the requirement for the multiple barrier disinfection approach in water treatment, slow sand filters are almost always accompanied by disinfection. Because slow sand filters with disinfection have not been used extensively, they are classified as "new" technology in the current literature.

Slow sand filters are similar to single media rapid rate filters (conventional systems) except that they

1. Are 50–100 times slower than normal filtration rates
2. Use biological processes, which may enhance chemical/physical removal processes
3. Require a ripening period before operation
4. Use smaller pores between sand (i.e., smaller sand particles)
5. Do not require backwashing
6. Have longer run times between cleanings

17.3.2 System Performance

Slow sand filters are most attractive for smaller systems with high-quality raw water. Specifically, water which comes from a protected surface water supply, has previously received only chlorination as a treatment, contains less than 10 NTU, and has no color problems is the most suitable for slow sand filtration. While their operational simplicity makes them very suitable for small plants, slow sand filters are also applicable for medium to large plants.

The advantages of slow sand filtration include its simplicity and reliability, low cost, and ability to achieve virus, *Giardia*, and *Cryptosporidium* removal from 91% to 99.9999% and 100%, respectively. In addition, these systems do not require continuous turbidity monitoring since they are applied to water supplies with relatively low turbidity.

Slow sand filters have several limitations, however. Due to a low filtration rate, these filters require relatively extensive land area. Without pretreatment (particularly coagulation), limitations exist on the quality of raw water that is suitable for treatment using slow sand filtration. Also because pretreatment is minimal or nonexistent at slow sand filter plants, other contaminants such as synthetic organic chemicals, disinfection by-product precursors, or inorganic chemicals are not readily removed. Systems with raw water containing high color or algae content are probably not appropriate for slow sand filtration, because these contaminants are not removed by slow sand filtration and the raw water will likely contain precursors for chlorination by-products.

17.3.3 System Design Considerations

Slow sand filters require influent that does not exceed the following parameters:

1. Turbidity of less than 10 or up to 20 NTU depending on other operating characteristics
2. Coliforms of less than 800 total/100 mL
3. Color of less than 30 units as prescribed by the Ten State Standards
4. Algae of less than 5 mg/m^3 of chlorophyll A

These are maximum limits, not typical operational parameters. Design flow rates range from 0.94 to 9.35 m^3/m^2 (1–10 MG/acre) per day, with the usual range from 2.8 to 5.6 m^3/m^2 (3–6 MG/acre) per day. Slow sand filters require sand with particle sizes ranging in effective diameters from 0.25 to 0.35 mm, with a uniformity coefficient of 2–3. A higher uniformity coefficient is acceptable for filters with pilot test verifications of the performance.

Typical underdrains for slow sand filters are made from split tile with laterals placed in coarse stone. These drains routinely discharge to a tile or concrete main drain. Recently constructed slow sand systems are equipped with perforated polyvinyl chloride pipe as laterals. Manually adjusted weirs, outlets, and valves are adequate for these systems. Inlet structures may be located at the end or side of the filter.

Slow sand filters perform poorly during the first 1–2 days of operation, called the "*ripening period*." The ripening period is the time required by the filter after a cleaning cycle to become a functioning biological filter. Although *Giardia* and *Cryptosporidium* removal is not usually affected by the ripening period, the overall poor water quality during this period requires provision of a filter-to-waste cycle.

These filters require continuous operation under submerged conditions, ranging in depth from 0.9 to 1.5 m (3–5 ft). Hydraulic filter outlet controls keep the filters submerged at all times. The difference between the level of water in the collection gallery and the level of water above the filter media is called the *head loss* through the media. The initial head loss is about 0.06 m (0.2 ft). Maximum head loss should be less than the submerged depth of 1.2–1.5 m (4–5 ft) to avoid air binding and the uneven flow of water through the filter medium. The buildup of the maximum head loss is slow, taking up to 6 months.

Redundant, or stand-by, systems are required to accommodate the extended cleaning periods associated with slow sand systems.

In climates subject to below freezing temperatures, slow sand filters usually must be housed. Unhoused filters in harsh climates develop an ice layer that prevents cleaning during the winter months. Uncovered slow sand filters will operate effectively, however, if influent turbidities are low enough to permit the filter to operate through the winter months without cleaning. An illustration of an unhoused system is provided in Fig. 17.5. Figure 17.6 shows an example of a housed system.

Because of filter housing costs, slow sand filters are most appropriate for small systems. Due to this expense, large systems can usually only utilize slow sand filters when they are located in moderate climates and therefore do not require housing. The slow sand filter in Salem, OR, is an example of a large system that is able to use unhoused slow sand filters because of climate.

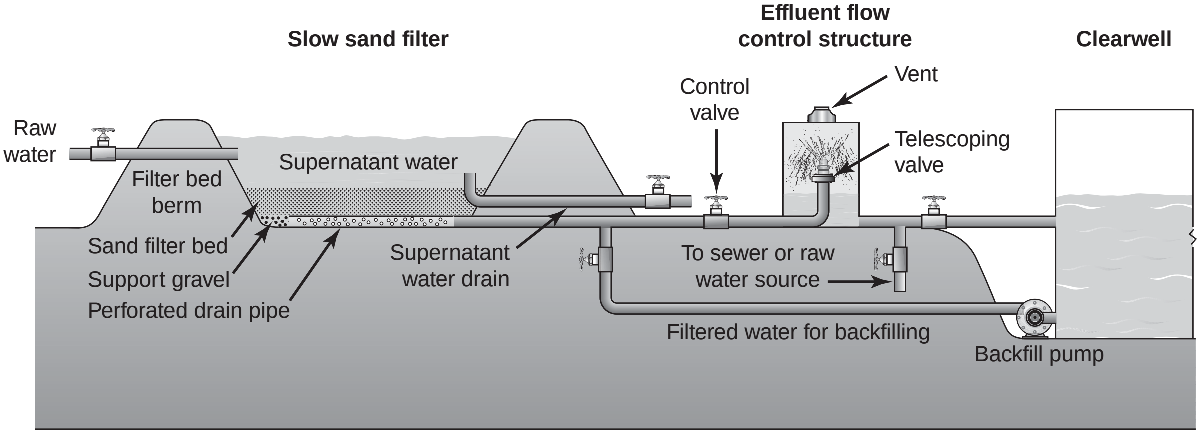


Figure 17.5 Typical un-housed slow sand filter installation (*Source*: U.S. EPA, 1992).

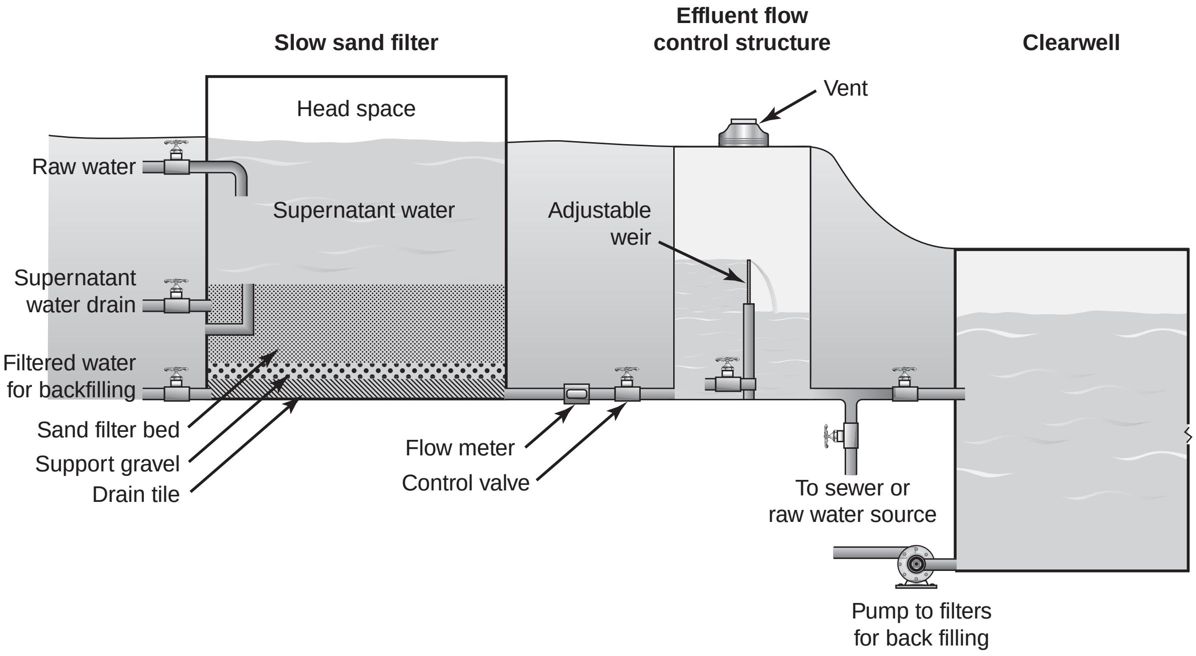


Figure 17.6 Typical housed slow sand filter installation (*Source*: UNIDO, 1995b).

17.3.4 Operation and Maintenance

The primary operational consideration for a slow sand filtration system is maintaining a clean filter. Cleaning becomes necessary when head loss reaches about 1.2 m (4 ft). The normal length of time between cleanings ranges from 20 to 90 days, but will vary depending on raw water quality, sand size, and filtration rate. Filters should be cleaned every 1–2 months.

Cleaning involves manually scraping off the top 2.5–5 cm (1–2 in.) of the filter media. The removed sand is either discarded or treated separately. Most small systems use manual cleaning techniques, but very large systems may use mechanical scrapers.

A number of precautions must be observed in regard to cleaning operations. Cleaning filters necessitates removing the filter from service, after which a ripening period is required to bring the filter back into operation. Operators must minimize intermittent operation by coordinating maintenance tasks. While cleaning, operators should take care to avoid disturbing the filter bed during backwashing, shock chlorination (if practiced), and raking the surface of the filter bed. Scouring and subsequent erosion of the filter media from refilling after cleaning must also be avoided. Gradual filling from the bottom of the filter during the startup phase partially addresses this problem. Excessive drying of the filter beds also should be avoided during cleanings. During the post-cleaning ripening period, filtered water should be directed to waste. Operators should periodically remove algae from the filter and, to accommodate seasonal changes, reduce filtration rates during the winter months.

If sand depth drops below 61 cm (24 in.), new sand needs to be added. Bed depths of only 30.5–50.8 cm (12–20 in.) have had poor performance records. Replacing sand, however, is not considered a normal operational or maintenance task because with careful cleaning, resanding may be necessary only once every 10 years.

In addition to the above maintenance considerations, slow sand filters require daily inspection, control valve adjustment, and turbidity monitoring. The filter housing structure, if present, also requires routine maintenance.

17.4 PACKAGE PLANT FILTRATION

17.4.1 General Process Description

Filtration package plants are built in a factory, skid mounted, and transported virtually assembled to the operation site.

They are designed to effectively remove turbidity and bacteria from water with generally consistent low-to-moderate turbidity levels.

There are over 1,000 filtration package plants operating in the United States with capacities ranging from 27.3 m^3/day to 1.6 m^3/s (7,200 gpd to 37.5 MGD). Many are built to conventional design standards. Others, using tube settlers or flotation cell, have reduced size and larger capacities.

The four major advantages of package plants are their compact size, cost effectiveness, relative ease of operation, and design for unattended operation. Typically, these types of filtration plants are used to treat small community water supplies and for a variety of special applications, including (a) emergency supplies, (b) recreational areas, (c) state parks,

(d) construction sites, (e) ski areas, (f) military installations, and (g) other areas not served by municipal supplies.

Filtration package plants can differ widely with regard to design criteria and operating and maintenance considerations. The four basic types of package plant filter systems are *conventional package plants, tube-type clarification package plants, adsorption clarifier package plants and dissolved air flotation-filtration package plants*.

17.4.2 Conventional Filtration Package Plants

Conventional filtration package plants are manufactured by several firms to a variety of specifications. As their name indicates, they contain the conventional processes of coagulation, flocculation, sedimentation, and filtration. Typical design standards for these units are

1. 20–30 min flocculation detention time
2. 2 h sedimentation detention time
3. Rapid sand filters rated at 1.34 L/s/m^2 (2 gpm/ft^2)

17.4.3 Tube-Type Clarifier Package Plants

A flow diagram of a tube-type clarifier package plant is illustrated in Fig. 17.7. This type of filtration plant has two versions with different capacity ranges; one version can treat from 0.63 to 6.3 L/s (10–100 gpm) and the other, equipped with dual units, can treat from 12.6 to 88.3 L/s (200–1400 gpm).

In these filtration package systems, the disinfectant, primary coagulant, and polyelectrolyte coagulant aid are added before the influent enters the flash mixer. After the flash mixer, the water enters the flocculation chamber where mechanical mixers gently agitate the water for 10–20 min depending on the flow.

The flocculated water then enters the tube settlers, which consist of many 2.5-cm (1-in.) deep, 99-cm (39-in.) long split-hexagonal-shaped passageways. The large surface area of the many 2.5-cm (1-in.) deep tube settlers achieves an effective clarification overflow rate of less than 6.1 m^3/day/m^2 (150 gpd/ft^2). Adequate clarification is attained with less than 15 min detention times.

The clarified water then enters a gravity flow mixed-media filter. A constant filtration rate is maintained by a low-head filter effluent pump discharging through a float-operated, level control valve. After a preset head loss is reached, backwashing of the filter is initiated automatically. A manual backwash cycle can be initiated any time (if desired). Settled sludge from the tube settlers is flushed during the backwashing cycle. Combining backwashing and tube settler flushing simplifies operations and reduces operator skill requirements.

17.4.4 Adsorption Clarifier-Filter Package Plant

The adsorption clarifier-filter package plant is a concept developed in the early 1980s. It uses an upflow filter with low-density plastic bead media (called the *adsorption clarifier*) followed by a mixed-media filter to complete the water treatment. The adsorption clarifier replaces the flocculation and the sedimentation basins, combining flocculation and sedimentation into one step. A typical example is contained in Fig. 17.8

While passing through the adsorption media, the coagulant and water are mixed, contact flocculated, and clarified. The mixing intensity, as measured by the mean temporal velocity, ranges from 150 to 300 per second. Flocculation is accomplished through turbulence as the water passes through the adsorption media. In addition, flocculation is enhanced by the contact between the flocculated materials and the floc-coated media. Turbidity is reduced through adsorption of the coagulated and flocculated solids onto the adsorption media and the previously adsorbed materials. The adsorption

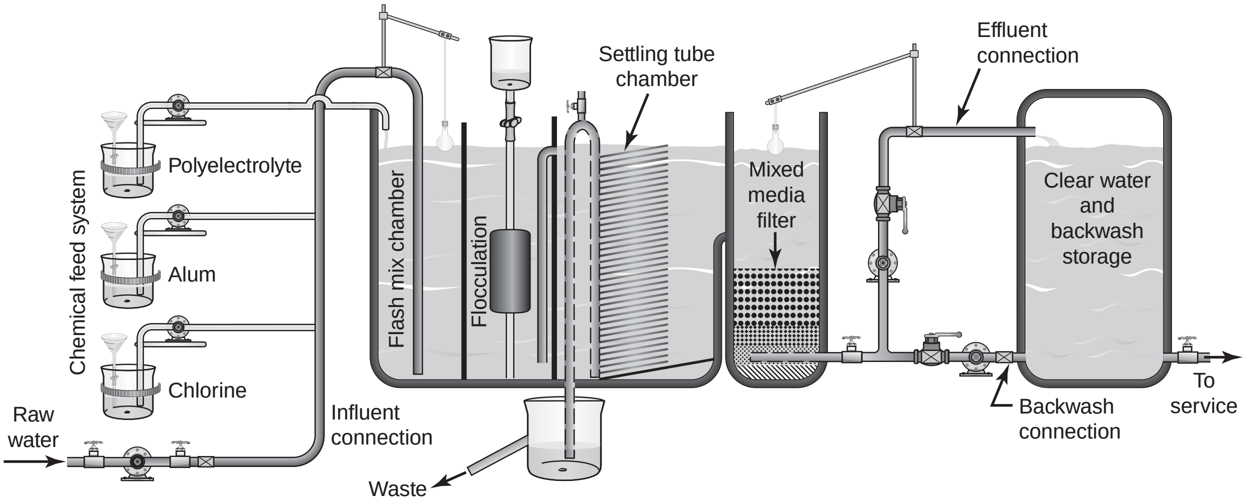


Figure 17.7 Flow diagram of a tube-type clarifier-filter package plant (*Source*: U.S. EPA, 1992).

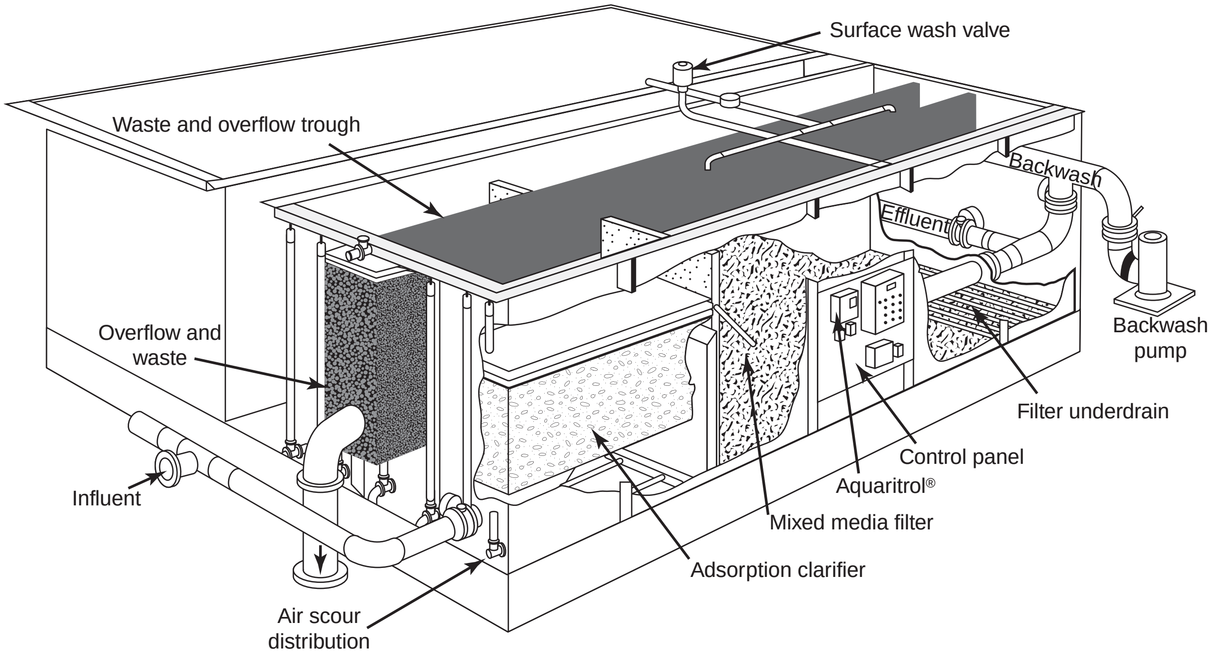


Figure 17.8 Adsorption clarifier-filter package plant (*Source*: U.S. EPA, 1990).

clarifier can achieve 95% or greater removal at 6.8 L/s/m^2 (10 gpm/ft^2). This highly efficient clarification method results in extremely compact designs.

Adsorption clarifiers are cleaned by a combination of air scouring followed by water flushing. The air scouring starts the cleaning process for the plastic media used in the clarifier. Adsorption clarifier cleaning is initiated more frequently than filter backwashing because more solids are removed by the clarifier. The clarifier cleaning process is automatically initiated either by a timer or a pressure switch that continuously monitors head loss across the adsorption media.

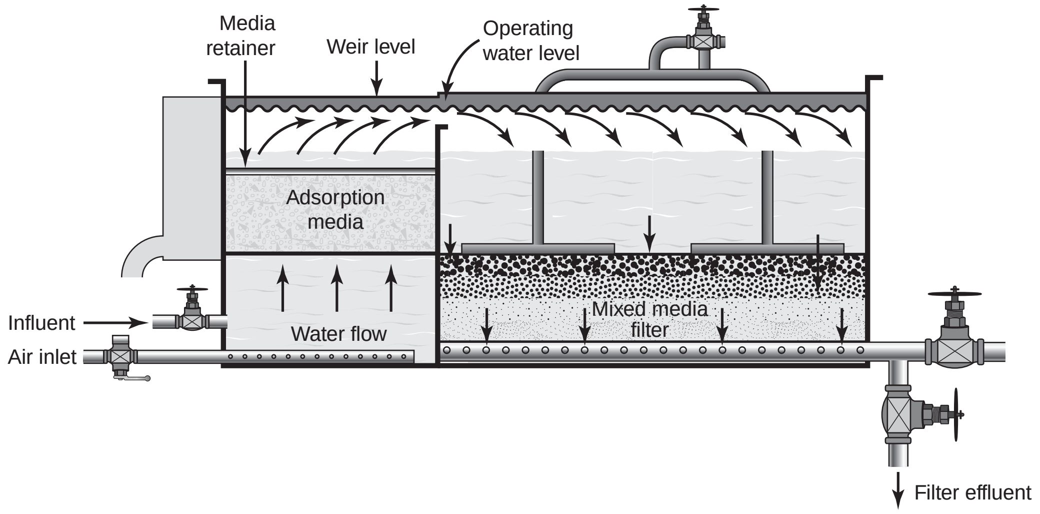
The air introduced under the adsorption media causes a vigorous scrubbing action. The scrubbing action dislodges solids, which are washed away by the flow of the incoming water. Flushing is generally timed to occur between every fourth and eighth hour. Complete cleaning of the adsorption media is not desired because performance is enhanced by some residual solids. Diagrams illustrating the various cycles of an adsorption clarifier package plant are contained in Fig. 17.9

17.4.5 Dissolved Air Flotation-Filtration Package Plant

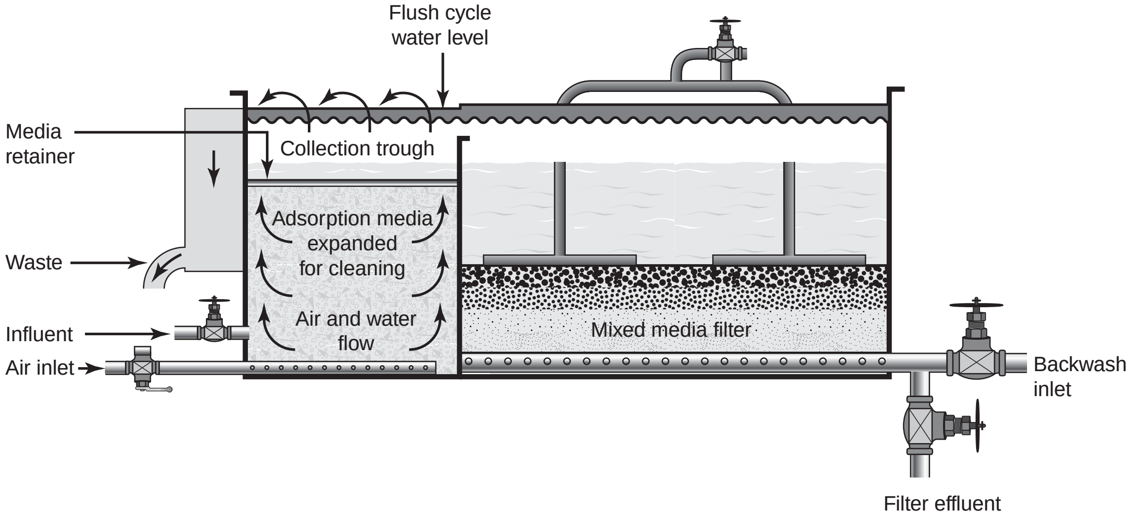
The dissolved air flotation-filtration (DAFF) package plant is a concept developed in the early 1980s for water purification. The process flow diagram of a potable water DAFF package plant consists of chemical feeding, mixing, flocculation dissolved air flotation (DAF) clarification, continuous automatic backwash filtration (ABF), and disinfection (see Fig. 11.11b). In comparison with a conventional water treatment plant, which consists of chemical feeding, mixing, flocculation, sedimentation clarification, sand filtration, and disinfection (see Figs. 11–15), the difference of the two plants in terms of unit processes is very small: the potable water DAFF package plant adopts DAF clarification and ABF, while the conventional water treatment plant adopts sedimentation clarification and conventional sand filtration. If both the DAFF package plant and the conventional plant treat the same raw water using same optimized chemicals and doses, both plants are expected to deliver similar effluent water quality; however, the DAFF package plant's foot print and detention time will be much smaller. Figure 17.10a shows the structure of the Lenox Water Treatment Plant (built in 1982 with a capacity of 1 MGD or 3.8 MLD) and the Pittsfield Water Treatment Plant (built in 1986 with a capacity of 37.5 MGD or 142 MLD). Figure 17.10b illustrates the second-generation Lee Water Treatment Plant. All the plants are DAFF package plants in Massachusetts, USA. Replacement of sedimentation with flotation allows the package plant to remove *Cryptosporidium* and *Giardia lamblia* cysts more efficiently. Water with flocculated impurities flows out of the mixing/coagulation/flocculation chamber, passing over an area where pressurized water is released. An air dissolving tube (ADT) generates microscopic-entrained air bubbles, which attach themselves to the floc particles causing them to float to the water surface. The DAF effluent is given a final cleansing through a foot (30 cm) of fine sand. The filter effluent is finally disinfected for storage and distribution.

Adoption of continuous ABF is another unique feature of the package plant. The filter media can be either single media (sand alone) or multimedia (anthracite coal and sand).

Filtration mode



Adsorption clarifier flush cycle



Backwash cycle

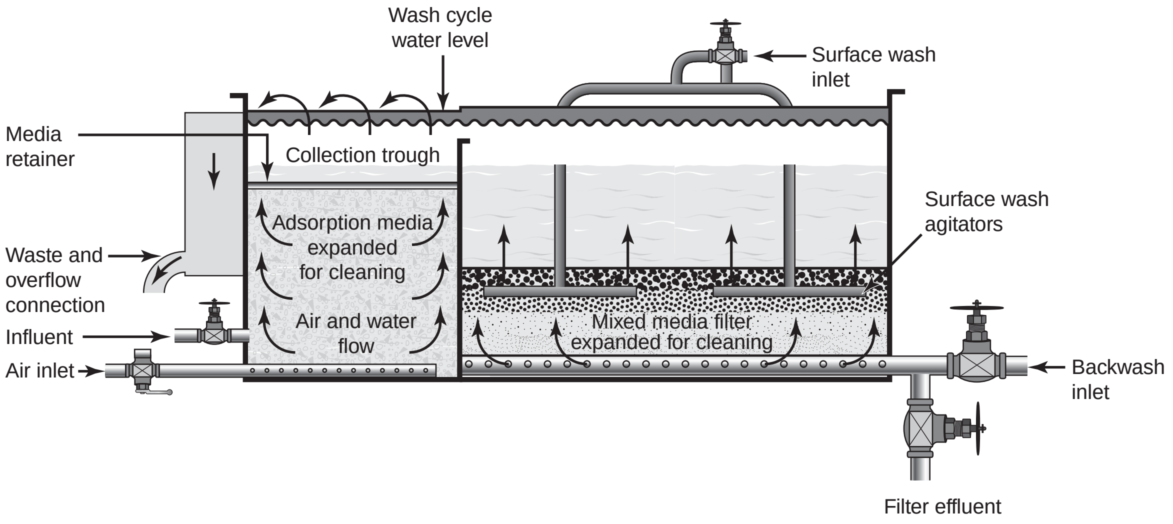
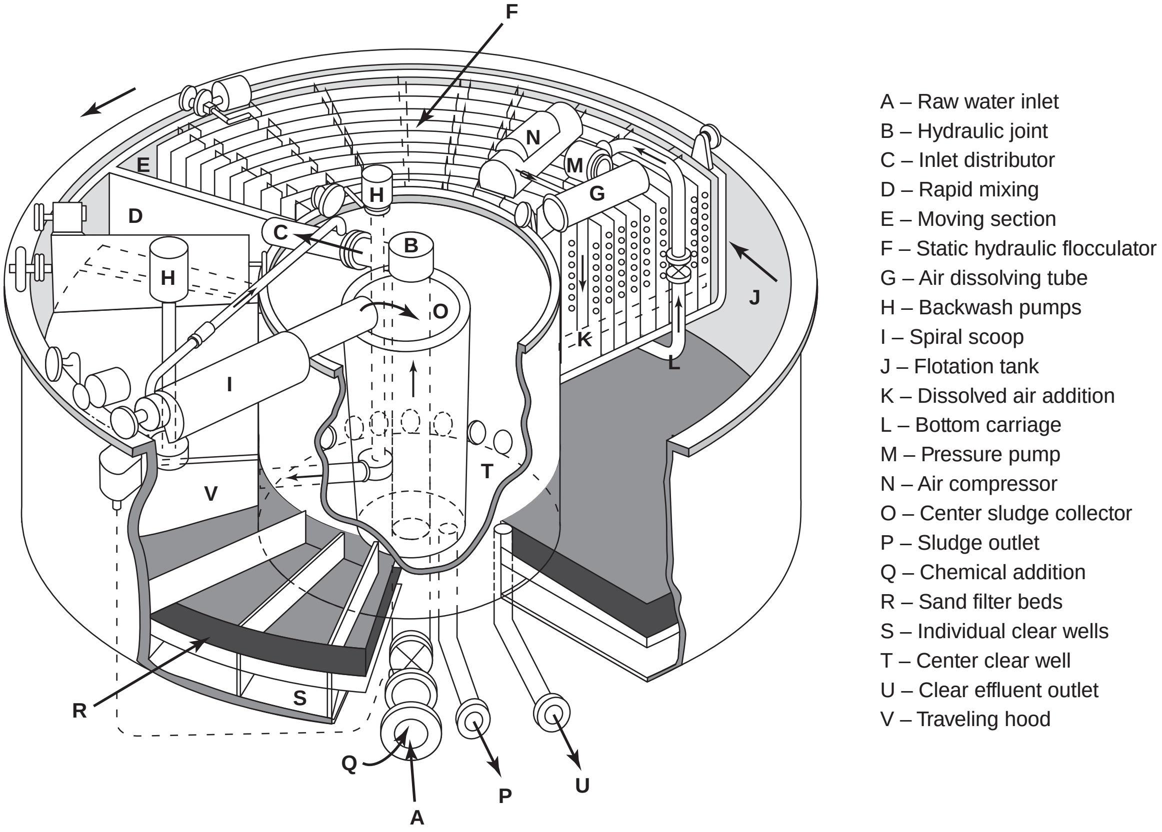


Figure 17.9 Various cycles of an adsorption clarifier-filter package plant (*Source*: U.S. EPA, 1990).



(a)

Figure 17.10 (**a**) Bird's view and process description of the first-generation potable water dissolved air flotation and filtration (DAFF) package plant (*Source*: UNIDO, 1995b).

The filtration bed is divided into multiple sections. Each filter section is individually backwashed, while the remaining sections continue to filter water. Filtered water from the bottom is withdrawn up through the sand, washing out impurities. The filter backwash water is recycled back to the coagulation/flocculation chamber for reprocessing. The only waste generated is the floated sludge (the *float*).

17.4.6 Operation and Maintenance of Package Plants

Package plant operation is simplified by automated features, and maintenance requirements are well documented in the manufacturer's manuals. However, the operator needs to be well acquainted with water treatment principles and the plant manual and should have attended a comprehensive training session.

Common automated devices found in package plants are effluent turbidimeters and chemical feed controls. The effluent turbidimeters and fail-safe controls are built into many plants to ensure that the finished water does not exceed set turbidity levels. Automated chemical feed systems are especially appropriate for plants without full-time operators or with highly variable influent characteristics.

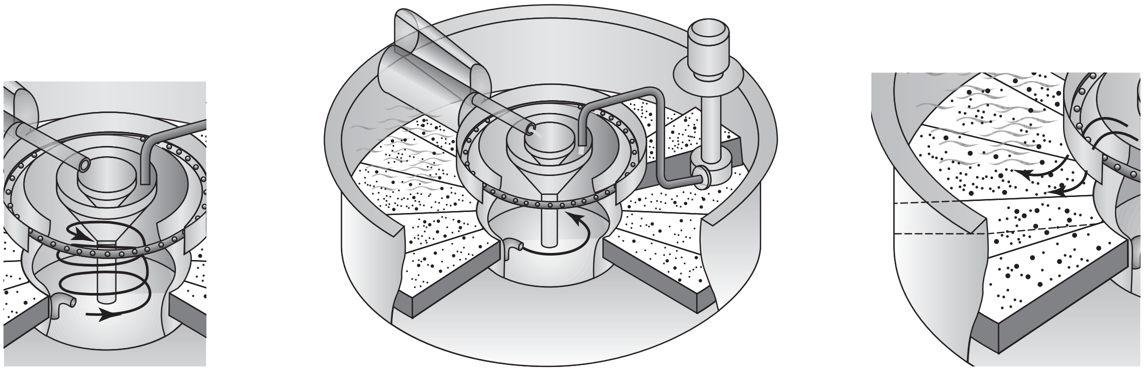
Typical plant operator and maintenance manuals contain operating principles, methods of establishing proper chemical dosages, operating instructions, and troubleshooting guides.

Periodic visits by the manufacturer to make adjustments to the plant and inspect the equipment operations and performance are recommended. The first visit should be no more than 6 months after initial operation; the next should follow in another 6 months. Subsequently, annual visits should be sufficient.

17.4.7 General System Performance of Package Plants

According to extensive manufacturer evaluations, system performance, in general, is improving because of better equipment, more highly skilled operators, and greater surveillance by regulators. Table 17.5 contains summaries of evaluations at three adsorption clarification-filtration package plants in Oregon. All finished water reported in the table had less than 0.53 NTU turbidity. Tables 17.6a and 17.6b show the typical performance of dissolved air flotation-filtration plants in Massachusetts and New York.

Process description

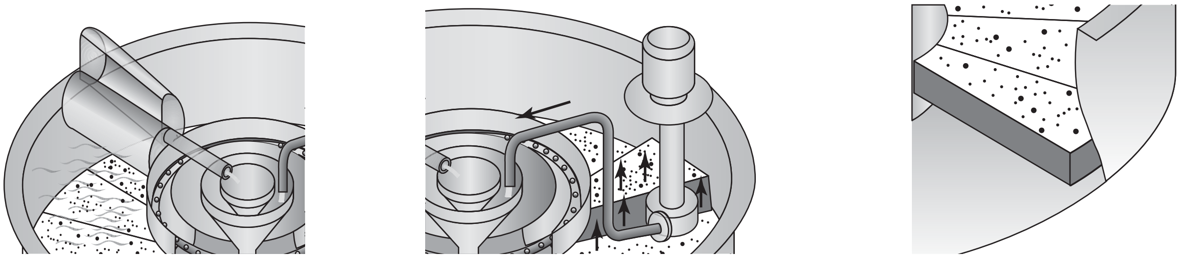


(1) Flocculation

Raw water mixed with flocculating agents enters through a nozzle into the flocculating chamber of the unit. The resulting gentle mixing velocity causes impurities to aggregate together forming snowflake-like flocks.

(2) Flotation

Water with flocculated impurities flows out of the flocculation tank, passing over an area where aerated water is released. An air dissolving tube system located outside the SASF unit generates these microscopic, entrained air bubbles which attach themselves to the floc particles, causing them to float to the surface.



(3) Sludge removal

The rotating KROFTA spiral scoop circulates around the SASF unit, gently lifting the floated impurities from the water surface. The sludge is emptied into the central collector for removal from the SASF unit.

(5) On-line backwashing

One sandfilter section is individually backwashed while the other sections continue to filter water. Filtered water from below is drawn up through the sand, washing out impurities. Backwash water is normally recycled back to the flocculation chamber for reflocculation.

(4) Filtration

Water is given a final cleansing through an eleven inch deep bed of fine sand which traps any impurities not removed by the flotation process. After filtration, the clarified water is held in the clearwell compartment under the sand for final removal from the SASF unit.

(b)

Figure 17.10 (**b**) Process description of the second-generation potable water dissolved air flotation and filtration (DAFF) package plant (*Source*: Krofta and Wang, LIWT, 2000).

Table 17.5 Summary results of adsorption clarification-filtration package plants

	Rates, gpm /ft^2 (m/h)		Raw water		Turbidity (NTU)		
Location	Adsorption clarifier	Mixed media	Temperature °F (°C)	Color units	Influent	Adsorption clarifier effluent	Mixed-media effluent
Corvalis, OR	5 (12.2)	9 (22.0)	68 (20)	0	100	8	0.27
	10 (24.4)	10 (24.4)	68 (20)	0	46	10	0.52
	8 (19.5)	7 (17.1)	57 (14)	0	103	21	0.30
Rainier, OR	10 (24.4)	5 (12.2)	41 (5)	30	9.3	1.4	0.24
	15 (36.6)	5 (12.2)	41 (5)	30	8.2	1.6	0.22
	20 (48.8)	5 (12.2)	41 (5)	30	8.1	1.7	0.20
Newport, OR	10 (24.4)	5 (12.2)	45 (7)	8	19	4.3	0.13
	15 (366)	7.5 (18.3)	45 (7)	8	15	37	0.11
	20 (48.8)	10 (24.4)	45 (7)	9	9	3.8	0.23

Source: US EPA.

Table 17.6a Water Quality Goals and Typical results of flotation-filtration package plants

Parameter (mg/L unless otherwise specified)	Massachusetts plant	New York plant
Influent		
Flow (gpm)	7.5–12	70–100
Temperature (°F)	39.2–62.6	33–48
pH (unit)	6.1–7.5	6.4–7.5
Turbidity (NTU)	1.0–3.5	0.08–14.2
Color (unit)	25–70	25–90
Microscopic count (#/mL)	1,967–17,328	410–34,623
Aluminum	0–0.17	0–0.074
Alkalinity, as $CaCO_3$	4–39	4–40
THMFP (ppb)	73–577	127–683
UV (254 nm)	0.056–0.303	0.11–0.28
Total coliform (#/100 mL)	360–1,000	<1-TNTC
Iron	0.03–0.078	0.114
Manganese	0.004–0.15	0.005
Humic substances	NA	1.5–9.6
Chemical Treatment		
Polymer 1849A	0.5–1.0	0.4–3.0
Sodium aluminate, as Al_2O_3	6–13	3.0–5.2
Alum, as $Al_2(SO_4)_3$	16.8–20.1	4.0–33.0
Lime	0–13.5	0
Filter effluent		
Flow (gpm)	7.38–11.9	67–99.7
pH (unit)	6.8–8.0	6.7–7.5
Turbidity (NTU)	0.1–0.7	0.05–0.39
Color (unit)	0–5	0–3
Microscopic count (#mL)	5–125	<1–267
Aluminum	0–0.1	0–0.11
Alkalinity, as $CaCO_3$	4–31	4–20
THMFP (ppb)	4–18	2–97
Chlorine demand	0.4–1.5	0.3–1.5
UV (254 nm)	0.056–0.097	0.002–0.089
Total coliform (#/100 mL)	0–2	<1
Iron	0–0.08	0
Manganese	0–0.09	0.004
Humic substances	NA	0–3.4
Chlorinated effluent		
pH (unit)	7.1–8.0	5.6–7.4
Turbidity (NTU)	0.2–0.4	0.17–0.5
Color (unit)	0–5	0–5
Microscopic count (#/mL)	NA	22–85
Aluminum residue	0–0.10	0–0.11
Chlorine residue	0.1–0.4	0.2–0.5
THMFP (ppb)	NA	34–65
Total coliform (#/100 mL)	< 1	< 1
Sludge from sandfloat		
Flow (gpm)	0.08–0.15	0.3–3.0
Total suspended solids	1145–3390	394–10,086

Source: Wang and Wang (1989).
ppb = μg/L.

Table 17.6b Typical results of flotation-filtration package plants

Water quality parameters	Water quality goals	Massachusetts plant	New York plant
Turbidity			
≤ 0.5 TU	Over 95% time	Over 99% time	100% time
≤ 0.3 TU	Over 75% time	Over 93% time	Over 97% time
≤ 0.2 TU	Over 50% time	Over 65% time	Over 97% time
Color			
≤ 5 CU	Over 95% time	Over 97% time	100% time
Aluminum			
≤ 0.15 mg/L	Over 95% time	100% time	100% time
≤ 0.09 mg/L	Over 75% time	Over 96%, time	100% time
≤ 0.05 mg/L	Over 50% time	Over 92% time	Over 83% time
Microscopic count			
≤ 1,000 #/mL	Over 95% time	100% time	100% time
≤ 400 #/mL	Over 75% time	100% time	100% time
≤ 300 #/mL	Over 50% time	100% time	100% time

Source: Wang and Wang (1989).

17.5 DIATOMACEOUS EARTH FILTRATION

17.5.1 Process Description

Diatomaceous earth filtration, also known as precoat or diatomite filtration, relies on a layer of diatomaceous earth about 3 mm (1/8 in.) thick placed on a septum or filter element. The septums may be placed in pressure vessels or operated under a vacuum in open vessels. A schematic diagram of a typical pressure system is shown in Fig. 17.11.

Diatomaceous earth filters are effective in removing cysts, algae, and asbestos. For water supplies with low amounts of suspended solids, they have lower initial costs than conventional rapid sand filtration systems. Diatomaceous earth filters are especially effective against *Cryptosporidium and Giardia* cysts.

The difficulties in maintaining a perfect film of diatomaceous earth as the filtering layer have discouraged wide usage of diatomaceous earth filters for potable water treatment, except in favorable circumstances (i.e., waters with low turbidity and low bacterial counts). Consequently, while this technology has been used extensively in specialized applications, such as swimming pools, it is a "new" technology for water supply treatment.

Diatomaceous earth filter plants have been chosen for projects with limited initial capital and for emergency or standby capacity to service large seasonal increases in demand. Since these systems are most suitable for applications where influent is low in turbidity and bacterial counts, water supplies presently receiving just chlorination may

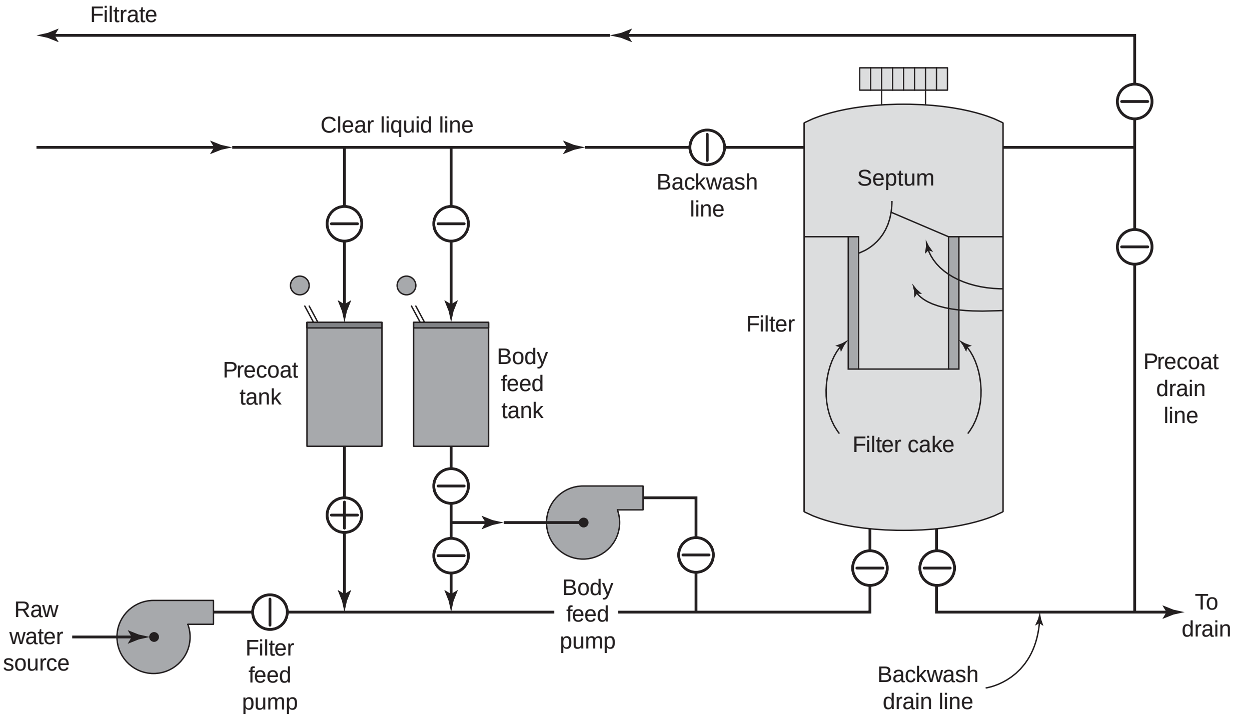


Figure 17.11 Typical pressure diatomaceous earth filtration system (*Source*: Wang and Wang, 1989).

consider using diatomaceous earth to meet the filtration requirements of the SWTR.

The majority of the minimum design criteria in the Ten State Standards for diatomaceous earth systems meet the SWTR. However, two design criteria in addition to the Ten State Standards are necessary to meet current regulations:

1. The minimum amount of filter precoat should be 1 kg/m^2 (0.2 lb/ft^2) to enhance *Cryptosporidium* and *Giardia* cyst removal.
2. The minimum thickness of the precoat should be increased from 3 to 5 mm (1/8–1/5 in.) (found to be more important than the size graduation of the diatomaceous earth), also to enhance *Cryptosporidium* and *Giardia* cyst removal.

An additional recommendation is to use coagulant (alum or a suitable polymer) to coat the body feed to improve removal rates for viruses, bacteria, and turbidity. Adding these chemicals to the coating does not improve *Giardia* and *Cryptosporidium* removal rates.

17.5.2 Operation and Maintenance

Operating a diatomaceous earth filter requires

1. Preparation of filter body feed (diatomaceous earth) and precoat
2. Verification of proper dosages
3. Periodic backwashing
4. Disposal of spent filter cake
5. Periodic inspection of filter septum for cleanliness and damage
6. Verification of the effluent quality

A common operating difficulty is maintaining a complete and uniform thickness of diatomaceous earth on the filter septum.

In some cases, alum precoating of the diatomaceous earth can improve performance. Typical alum doses range from 1% to 2% by weight (1–2 kg/100 kg) of diatomaceous earth applied. Typical precoats of diatomaceous earth of 0.49–0.98 kg/m^2 (0.1 to 0.2 lb/ft^2) are applied to prepare the filter. These filters also require a continuous supplemental body feed of diatomite because the filter is subject to cracking. If the filter has no body feed, there will be rapid increases in head loss due to buildup on the surface of the filter cake. Body feed rates must be adjusted for effective turbidity removal. Diatomaceous earth filters do not need a filter-to-waste cycle, because of the precoating process.

Regular cleaning will maintain up to 95% of the filter septum area available for filtration after 100 filter runs. The filter cake drops off the septum during an interruption in flow, such as occurs during cleaning. During operating interruptions, clean diatomaceous earth and filter water should be used to recoat the filter to reduce the potential for passage of pathogens.

Filter runs typically range from 2 to 4 days. The rate of the body feed and the diatomite media size are critical in determining the filter run length. Shorter filter runs will minimize filtered water odor and taste problems stemming from the decomposition of organic matter trapped in the filter.

Vacuum diatomaceous earth filters are a variation of this technology that offer the advantages of visibility during backwashing and of not requiring pressure vessels. Their primary disadvantage is that they run an increased risk of the release of gases in the filter cake that shorten filter runs.

17.6 CARTRIDGE FILTRATION

Cartridge filters are considered a technology suitable for removing microbes and turbidity in small systems. These filters are mechanically simple but manually operated, so they could be expensive to operate. They also require low turbidity influent. Cartridge filters use ceramic or polypropylene microporous filter elements that are packed into pressurized housings. They operate by the physical process of straining the water through porous membranes and can exclude particles down to 0.2 μm. The pore sizes that are suitable for producing potable water range from 0.2 to 1.0 μm. The ease of operation and maintenance of cartridge filters makes them very attractive for small systems.

One manufacturer uses single microporous ceramic filter elements packaged together in a cartridge housing with flow capacities of up to 1.5 L/s (24 gpm). This filter has pore sizes as small as 0.2 μm and is suitable for influent with moderate levels of turbidity, algae, and microbial contaminants. The clean filter pressure drop is about 3.2 kg/cm^2 (45 psi), while the pressure drop during cleaning reaches about 6.2 kg/cm^2 (88 psi).

17.6.1 Cartridge Filtration Applications

The City of Cape Coral Reverse Osmosis Water Treatment Facility, FL, USA has a capacity of 15 MGD (56,8 MLD), and adopts cartridge filtration as the pretreatment to RO. Cartridge filtration is used in small single family homes as well as in very large WTPs.

Roughing filters, for pretreatment prior to cartridge filtration, are sometimes necessary to remove larger suspended solids and prevent the rapid fouling of the cartridges. Roughing filters can be rapid sand filters, multimedia filters, or fine mesh screens.

Prechlorination is recommended to prevent surface-fouling microbial growth on the cartridge filters and reduce microbial pass-through. Except for chlorine, no chemical additions are necessary. There is a lack of data concerning the effectiveness of cartridge filters for viral removal.

17.6.2 Operation and Maintenance of Cartridge Filtration

These systems are operationally simple, apart from cleaning and membrane replacement. There is no need for skilled personnel; personnel are necessary only for general maintenance. Ceramic membranes may be cleaned and used for repeated filter cycles. Polypropylene cartridges become fouled relatively quickly and must be replaced with new units.

In one manufacturer's unit, cleaning the ceramic cartridge filters entails cleaning each vertical filter element with a hand-operated hydraulically driven brush that fits over the element. Some manufacturers use disposable polypropylene filter elements in multi-cartridge stainless steel housing to avoid the cleaning procedures. This type of unit is available with capacities ranging from 0.13 to 45.4 L/s (2–720 gpm). The primary disadvantage of the disposable polypropylene membrane is that it can only be cleaned once before disposal. Manufacturers' guidelines state that these filters have service periods ranging from 5 to 20 days with influent turbidities of 2 NTU or less, depending on the pore size of the filter. Another manufacturer incorporates particles of silver into the ceramic filters to prevent bacterial growth.

EXAMPLE 17.1 DETERMINATION OF THE TYPE OF A CARTRIDGE FILTRATION UNIT FOR SMALL COMMERCIAL OR RESIDENTIAL APPLICATIONS

How to select a suitable type of membrane cartridge filtration for a small private water system to remove *Cryptosporidium* and *Giardia* cysts from water?

Solution:

Many cartridge filtration devices meet the definition of membrane filtration. They can be subject to the same direct integrity testing in accordance with the state/local public health rule requirements and would also be eligible for *Cryptosporidium* removal credit as a membrane filtration process under the US EPA regulations. US EPA refers to these cartridge filtration devices as "membrane cartridge filtration." For the small private water systems, the users should check the filter label. Only those with the following messages are effective for *Cryptosporidium*: "Tested and certified by the National Sanitation Foundation (NSF) standard 53 for cyst removal," "Tested and certified by NSF standard 53 for cyst reduction," "Reverse osmosis," "Absolute micron size of one micron or smaller." To find out if a particular filter removes *Cryptosporidium,* contact NSF International, which is an independent group. Ask for a list of "Standard 52 Cyst Filters."

17.7 MEMBRANE FILTRATION

17.7.1 Process Description

Membranes processes involve the use of membranes for phase separation. The phases include solid phase (such as suspended solids, organic solids, and inorganic solids), liquid phase (such as water, ethanol, and chloroform), and gas phase (such as air, nitrogen, and oxygen). A membrane is a porous filtration medium, which can be cationic, anionic, or nonionic in nature and acts as a barrier to prevent mass movement of selected phases, but allows passage of remaining phases. The main applications of the membrane filtration processes are processing water and wastewater streams. Recently the membrane processes have been used for purification of gaseous streams.

Membrane filtration processes include at least the following five main subcategories for processing water (Fig. 17.12):

1. *Microfiltration* (MF): It is a pressure filtration process for the separation of mainly suspended solids in the particle size range of about 0.08–10 microns. The primary function affecting solids separation from water is the size of suspended solids. The hydraulic pressure applied in MF is about 1–2 bars (15–30 psig), primarily for overcoming resistance of the "cake" (1 micron = 1 micrometer = 1 μm = 10^{-3} mm = 10^{-6} m = 0.00004 in. = 10,000 Angstroms = 10^4 A).
2. *Ultrafiltration* (UF): It is another pressure filtration process for the separation of macromolecular solids in the particle size range of about 0.001–0.1 micron. The primary factor affecting solids separation from water relies on the size of macromolecular solids. The hydraulic pressure required by UF for overcoming hydraulic resistance of the polarized macromolecular layer on the membrane surface is about 1–7 bars (about 15–100 psig; 104–695 kPa gauge)
3. *Nanofiltration* (NF): NF membranes are multiple-layer thin-film composites of polymer consisting of negatively charged chemical groups and are used for retaining molecular solids (such as sugar) and certain multivalent salts (such as magnesium sulfate), but passing substantial amounts of most monovalent salts (such as sodium chloride), at an operating pressure of about 14 bars (200 psig; 1388 kPa gauge). Both molecular diffusivity and ionic charge play important roles in the separation process. The sizes of molecular solids and multivalent salts to be rejected by NF are normally in the range of 0.0005–0.007 microns.
4. *Reverse osmosis* (RO): RO membranes are mainly made of cellulose acetate with the pore sizes of about 5–20 Angstroms and are used for rejecting salts (as high as 98%) and organics (as high as 100%), at an operating pressure of about 20–50 bars (300–750 psig; 2082–5205 kPa gauge). The hydraulic pressure (through a pump) is used to provide the driving force for permeation or for overcoming the chemical potential difference between the concentrate and the permeate, expressed in terms of the osmotic pressure. The sizes of molecular solids and salts (multivalent as well as monovalent) to be rejected by RO are normally in the range of 0.00025–0.003 microns.
5. *Electrodialysis* (ED): ED uses voltage or current as the driving force to separate ionic solutes. The sizes of ionic solutes to be rejected or separated by ED are normally in the range of 0.00025–0.08 microns, depending on the pore size of ED membranes. EDR is the electrodialysis reversal (or reverse electrodialysis) process, which is similar to ED, but its cathodes and anodes can be reversed for automatic cleaning during operation. Figure 17.13 illustrates the effect of ED and EDR.

Membrane processes are characterized by two basic process parameters: (a) *flux*, which is the rate of transport of solvent or solution through the membrane; and (b) *rejection,* which is the degree of separation of a particular feed component. There are six major variables that affect the two basic process parameters:

1. Driving force in terms of applied transmembrane pressure and/or electric voltage/current
2. Flow velocity, which affects turbulence and mass transfer coefficient
3. Process water temperature, which has effects on physical properties such as density, viscosity, diffusivity, osmotic pressure, surface tension, and others
4. Feed stream characteristics in terms of particle concentration, particle size, viscosity, molecular weight, molecular configuration, ionic charges, and fouling potential
5. Membrane module in terms of materials, pore sizes, membrane configuration, membrane ionic charges, and feed compatibility.

17.7.2 System Design Considerations

There are basically six different designs of membrane modules:

1. Tubular modules with channel diameters greater than 3 mm
2. Hollow fiber or capillary modules made of self-supporting tubes, usually 2 mm or less in internal diameters
3. Plate modules
4. Spiral-wound modules
5. Pleated sheet modules
6. Rotary modules

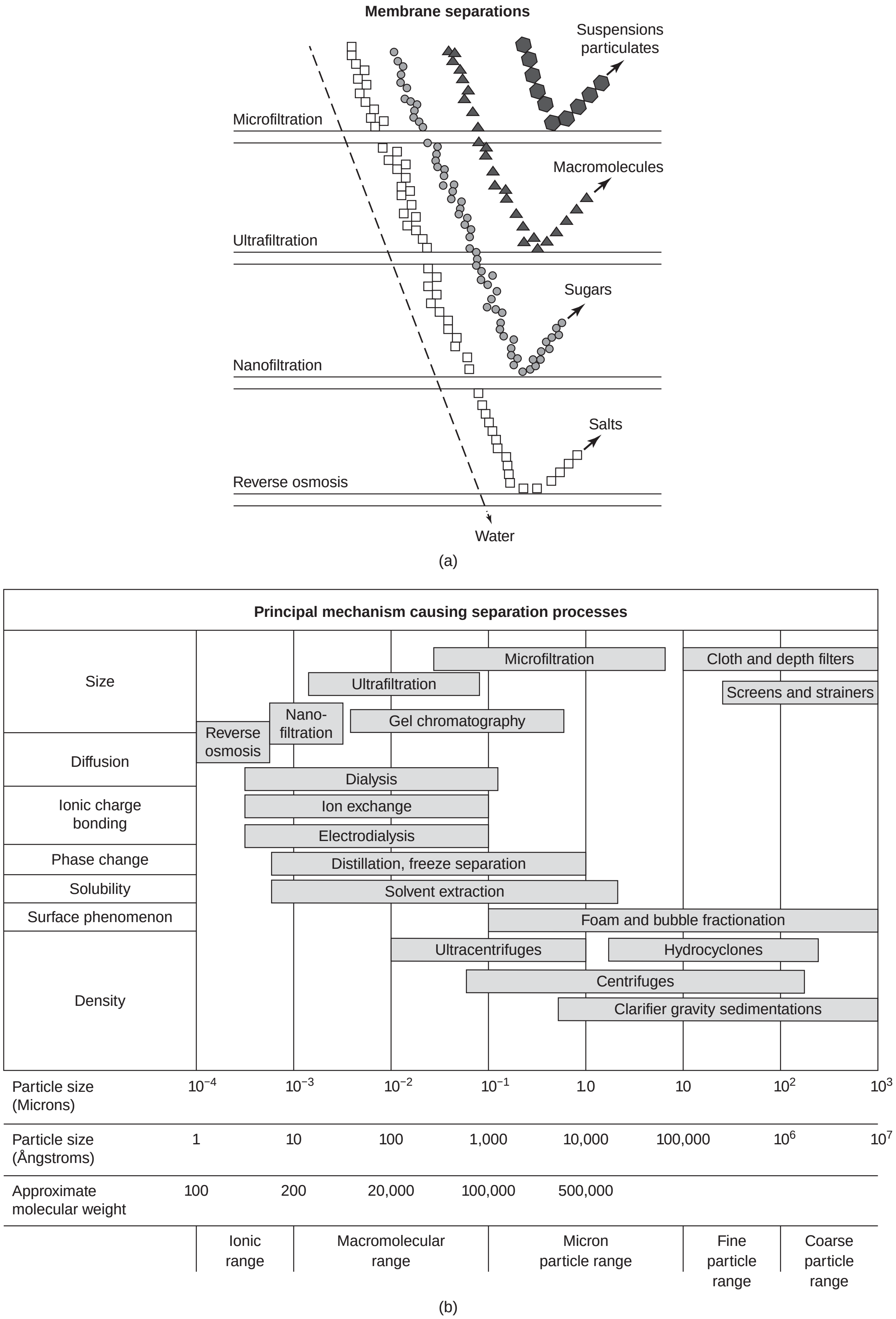


Figure 17.12 (**a**) Descriptive separation capabilities of membrane filtration processes (*Source*: UNIDO, 1995a). (**b**) Separation capabilities of membrane filtration processes (*Source*: UNIDO, 1995a).

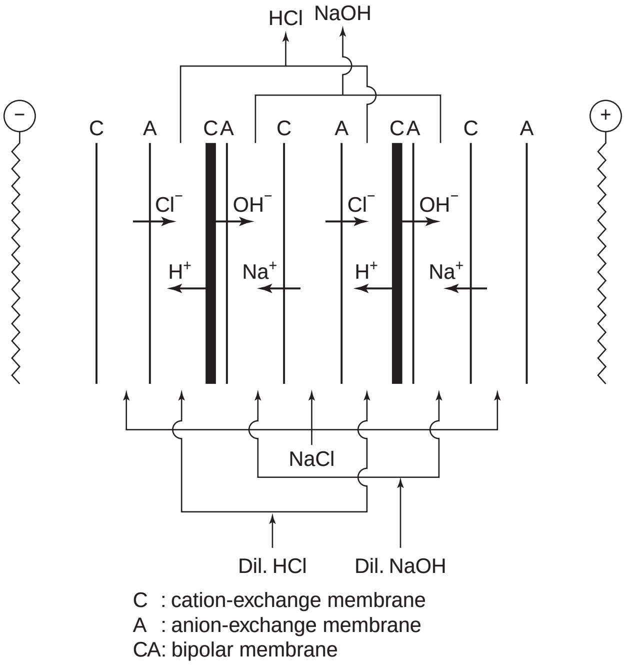


Figure 17.13 Separation capabilities of ED and EDR (*Source*: UNIDO, 1995a).

The latter four module designs use flat sheets of membrane in various configurations.

In selecting a particular membrane module and a particular membrane process, the major criteria are (a) feed stream characteristics, which affect the biocompatibility of the membranes; (b) flux requirements, which are controlled by the volumetric rate of a feed stream; (c) rejection requirements, which decide the process objectives and treatment efficiencies; and (d) cost requirements, which are affected by the energy consumption, membrane replacement cost, and operating and cleaning costs. Biocompatibility of the membranes relates to the interaction between the membrane module and the feed stream. Major biocompatibility factors include (a) stability to extremes in temperature, pressure, and pH, especially under cleaning and sanitizing conditions; (b) membrane–solute interactions, which affect the rate of fouling, cleaning, yields, and rejection of individual feed substances; and (c) acceptability of the membrane as a contact material for the final product, which essentially implies using membrane materials that are inert and do not leach out any toxic substances from the membrane into the final product. In this regard, there are new generations of membranes, made of expensive inorganic inert materials, such as ceramics, stainless steel, and carbon–zirconia.

MF membranes are made of a wide range of inorganic materials (such as alumina, zirconia–carbon composites, carbon–carbon composites, ceramics, stainless steel, and silica) and natural and synthetic polymers (such as polypropylene, polycarbonates, polysulfone, polyvinylchloride (PVC), copolymer, cellulose esters, and cellulose acetate).

UF membranes are mainly made of polysulfone-type materials (such as polyether sulfone, polyphenylene sulfone, and sulfonated polysulfone) although they are also available in a wide range of organic materials (such as PVC copolymer and cellulose acetate) and inorganic materials (such as ceramic composites and stainless steel).

Most NF membranes are multiple-layer thin-film composites of synthetic polymers. The active NF membrane layer usually consists of negatively charged chemical groups. NF membranes are of porous filter media with an average pore diameter of 2 nm. The nominal molecular weight cutoff ranges from 100 to 200. The active NF membrane layer can be made of polyamide, polyvinyl alcohol, sulfonated polysulfone, and sulfonated polyethersulfone. Salt rejection by NF membranes is mainly due to electrostatic interaction between the ions and the NF membrane. Rejection of neutral substances is by size.

Cellulose acetate and derivatives are widely used as the RO membranes, despite their real and perceived limitations. Thin-film composite membranes containing a polyamide separating barrier on a polysulfone or polyethylene supporting layer generally give better performance for RO applications with regard to temperature and pH stability and cleanability, but have almost zero chlorine resistance. In general, these thin-film composite membranes will be the material of choice for RO applications, unless there is a specific fouling problem with these membranes.

The membranes for UF, NF, and RO have been tried for ED with various results. Recently, the use of ionic membranes in ED and EDR processes have been emphasized (Fig. 17.13).

Figures 17.14, 17.15, 17.16, and 17.17 indicate special features of four types of membrane equipment: tubular membrane modules, hollow-fiber membrane modules, plate membrane modules, and spiral-wound membrane modules. Each design has its own special applications, advantages, and disadvantages.

The large-bore tubular membrane modules (Fig. 17.14) are suitable for food streams with high concentration of suspended solids such as citrus juices and animal waste streams, even though the tubular membrane modules have the lowest packing densities and highest energy consumption among all modules. The tubular designs with ceramic inorganic membranes are frequently used in the food processing industries.

The hollow-fiber membrane modules (Fig. 17.15) have extremely high packing density (surface area to volume ratios) and comparatively low energy consumption and are suitable for comparatively clean feed streams with low concentrations of suspended solids and macromolecules. Most food materials in the food processing feed streams, however, contain macromolecules displaying non-Newtonian behavior. Their viscosity will increase dramatically above certain concentrations, making pumping difficult and reducing mass transfer within the boundary layer. This will eliminate most

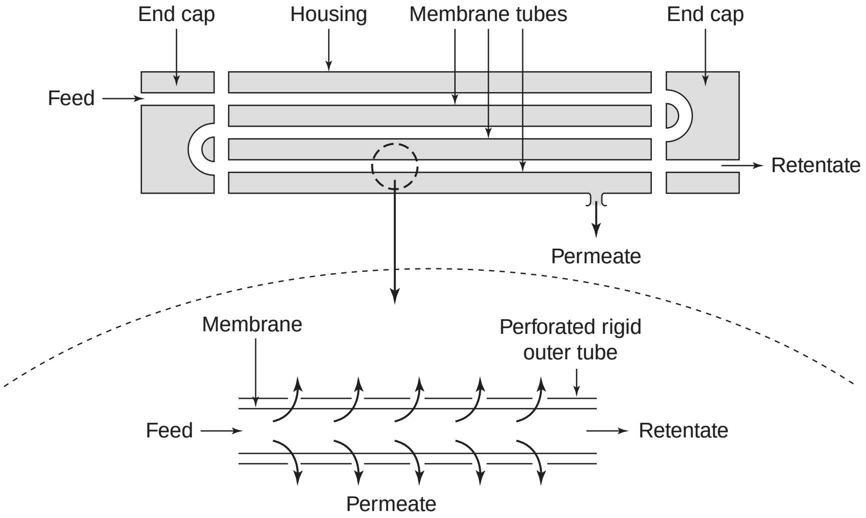


Figure 17.14 Schematic of tubular membrane module (*Source*: UNIDO, 1995a).

hollow fiber/capillary modules because they cannot withstand high pressure drops.

Membrane modules utilizing flat sheets (spiral-wound, plate, and pleated sheet modules) usually have a mesh-like spacer between sheets of membrane. This restricts their use to clear feed streams containing only fine suspended solids. Feed streams containing large suspended solids would be treated poorly in spiral-wound modules, owing to the spacers in their feed channels. On the other hand, spiral-wound membrane modules (Fig. 17.17) are the lowest in capital costs and energy consumption. The trend in the food and beverage in recent years seems to be away from plate modules (Fig. 17.16) and toward spiral-wound modules (Fig. 17.17), with ceramic tubular modules (Fig. 17.14) holding their own.

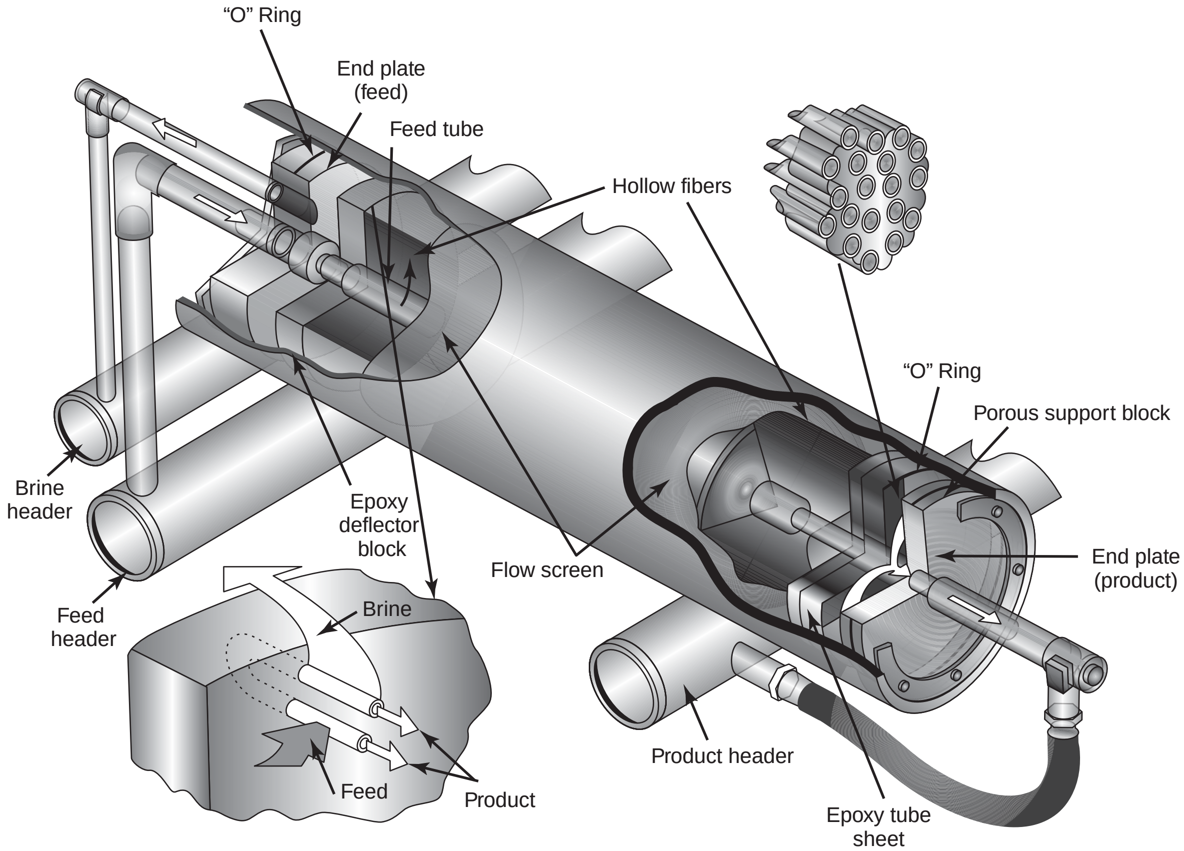


Figure 17.15 Schematic of hollow fiber membrane module (*Source*: UNIDO, 1995a).

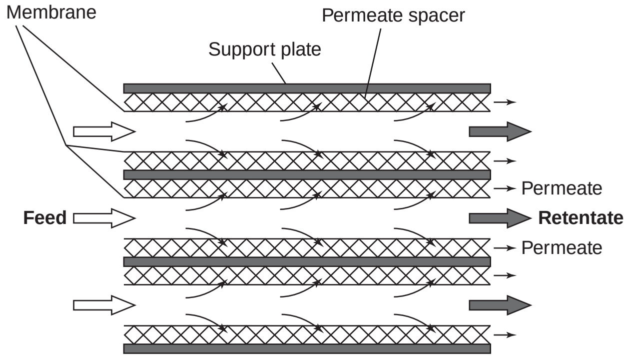


Figure 17.16 Schematic of plate membrane module (*Source*: UNIDO, 1995a).

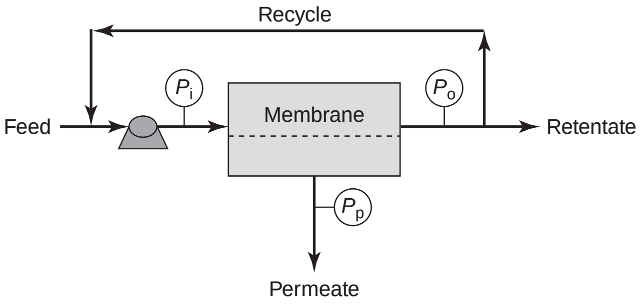


Figure 17.18 Simplified membrane process system (*Source*: UNIDO, 1995a).

17.7.3 Operation of Membrane Filtration

17.7.3.1 Basic Membrane Process System The major components of any force-driven membrane process system (MF, UF, NF, or RO) are the high-pressure booster pump and the membrane module. Figure 17.18 shows a simplified process flow scheme. Feedwater enters the suction side of the booster pump, which raises the pressure to a level required for operation, depending upon the membrane process (MF, UF, NF, or RO) and the application (desalination, fresh water treatment, or wastewater reclamation). Then feedwater enters the membrane module at pressure P_i and is split into two streams: (a) *permeate* at pressure P_p and (b) *retentate* at pressure P_0.

The permeate that passes through the membrane itself is the product water, which has drastically reduced suspended and/or dissolved solids to C_p, depending on the type of membrane process applied. The retentate does not actually pass through the membrane, but flows parallel to the surface, exiting the opposite end. The retentate is the concentrate or reject, retaining high concentrations of rejected solids (C_r). A certain portion of the retentate can be recycled for reprocessing. The transmembrane pressure, pressure drop, flux,

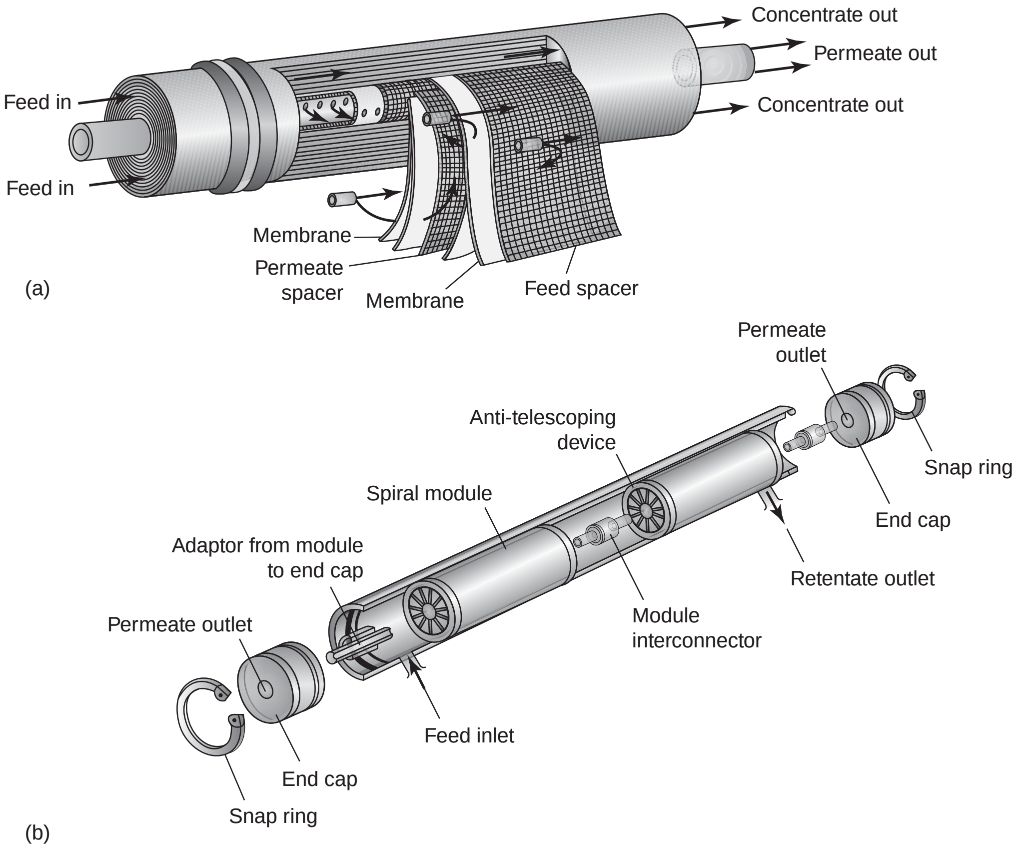


Figure 17.17 Schematic of spiral-wound membrane module (*Source*: UNIDO, 1995a).

rejection (R), 100% rejection, and 0% rejection are all defined in Fig. 17.18. The flow rate/velocity is a function of pressure drop. A mass balance of water flow is listed as follows:

$$Q_f = Q_p + Q_r \tag{17.1}$$

Recovery or system conversion (%) is then expressed as follows:

$$Y = (Q_p/Q_f) \times 100 \tag{17.2}$$

where Q_f = feed flow rate, gpd (L/h); Q_r = retentate flow rate, gpd (L/h); Q_p = permeate flow rate, gpd (L/h); and Y = recovery of system conversion, %.

Other operational equations are

$$\Delta P = P_i - P_0 \tag{17.3}$$

$$\Delta P_T = [(P_i + P_0)/2] - P_p \tag{17.4}$$

$$F = V_p/(A \times t) = Q_p/A \tag{17.5}$$

$$R = [1 - (C_p/C_f)] \times 100 \tag{17.6}$$

where ΔP = pressure drop, psi (kPa); P_i = membrane filtration unit's influent feed pressure, psi (kPa); P_0 = membrane filtration unit's effluent retentate pressure, psi (kPa); ΔP_T = transmembrane pressure (TMP), psi (kPa); P_p = permeate pressure, psi (kPa); F = flux, gpd/ft^2 (L/h/m^2); V_p = permeate volume, gal (L); A = membrane area, ft^2 (m^2); t = process detention time, d (h); R = mineral or impurity rejection, %; C_f = mineral or impurity in feed water, mg/L; C_p = mineral or impurity in permeate or product water, mg/L; and Q_p = permeate flow, gpd (L/h).

The product water flow through a semipermeable membrane of a membrane filtration system, such as MF, UF, NF, or RO, during operation can be expressed by the following equations:

$$F_w = C_{wp}(\Delta P_m - \Delta O_s) \tag{17.7}$$

$$F_s = C_{mp}(C_1 - C_2) \tag{17.8}$$

where F_w = water flux, g/cm^2/s; C_{wp} = water permeability constant, g/cm^2/s-atm; ΔP_m = pressure differential applied across the membrane, atm; ΔO_s = osmotic pressure differential across the membrane, atm; F_s = mineral flux, g/cm^2/s; C_{mp} = mineral permeability constant, cm/s; and $C_1 - C_2$ = mineral concentration gradient across the membrane, g/cm^3.

Note that the water flux, which is the rate of water flow through the membrane, is dependent on the applied pressure, while the mineral flux is independent of the applied pressure. As the pressure of the feedwater increases, the water through the membrane increases, while the flow of minerals almost remains constant. Due to dilution, the water quality of the product water is improved.

During membrane filtration process operation, the mineral content of the feedwater gradually increases due to mineral rejection; in turn, the osmotic pressure differential across the membrane ΔO_s increases and the water flux F_w decreases in accordance with Eq. (17.7). Finally since the membrane rejects a constant percentage of minerals, product water quality decreases with increasing feedwater concentration. An examination of Eq. (17.8) shows that the greater the concentration gradient ($C_1 - C_2$) across the membrane, the greater the mineral flux F_s, and the higher the mineral concentration in the product water.

For process operation, feedwater pressure, temperature, and pH must be properly controlled. The effects of pressure, temperature, and pH on membrane filtration process performance are shown in Tables 17.7, 17.8, and 17.9, respectively.

Table 17.7 indicates that the higher the operational pressure of a membrane filtration system, the higher the percent mineral rejection will be, at the expense of a higher power cost.

In membrane filtration operation, such as RO and NF, feedwater temperature has a significant effect on membrane performance. Table 17.8 indicates that as the temperature of the feedwater increases, flux increases. For the purpose of comparison, flux of a membrane is reported at a fixed reference temperature, such as 25°C.

Cellulose acetate membrane is subject to long-term hydrolysis, resulting in a lessening of mineral rejection capability. Table 17.9 indicates that the hydrolysis rate of a

Table 17.7 Typical mineral rejection of reverse osmosis for three different feedwater TDS concentrations

Pressure		Mineral rejection (%)		
psi	kPa	TDS 1,000 mg/L	TDS 5,000 mg/L	TDS 10,000 mg/L
200	1,388	86.5	84.0	80.0
300	2,082	90.6	89.8	87.2
400	2,776	92.8	92.3	90.8
500	3,470	94.4	93.8	93.2
600	4,164	95.3	94.8	94.4
700	4,858	95.9	95.6	95.3
800	5,552	96.4	96.1	95.8
900	6,246	96.8	96.6	96.4
1,000	6,940	97.2	96.8	96.6

Source: US EPA.

Table 17.8 Effect of water temperature on water flux rate of a cellulose acetate RO membrane unit operated at 400 psi (2758 kPa or 28 kg/cm^2)

Water flux		Feedwater temperature	
gpd/ft^2	L/h/m^2	°F	°C
8.0	13.6	50	10
9.5	16.1	60	15.6
11.0	18.7	70	21.1
12.6	21.4	80	26.7
14.1	23.9	90	32.2

Source: US EPA.

Table 17.9 Effect of water temperature and pH on hydrolysis rate for cellulose acetate membrane

	Relative hydrolysis rate (s^{-1})		
	Water temperature		
Water pH value unit	34°F (1°C)	77°F (25°C)	100°F (37.8°C)
2.0	7.30	27	80
3.0	0.73	3.30	9.0
4.0	0.24	0.86	2.8
4.5	0.22	0.68	2.4
5.0	0.23	0.70	2.2
6.0	0.70	1.60	3.6
7.0	4.30	8.00	17
8.0	36	70	150
9.0	600	–	–

Source: US EPA.

cellulose acetate membrane is accelerated by increased water temperature and is also affected by feedwater pH. Optimum pH range (5–6) ensures a lower hydrolysis rate, in turn, a longer membrane life. Thin film composite membranes are not subject to hydrolysis, but pH adjustment of feedwater may be needed for scale control.

17.7.3.2 Uniform Transmembrane Pressure Membrane Process System

Figure 17.19 shows an innovative double lube constant pressure membrane process system, also known as the *Bactocatch process.* It requires the simultaneous operation of a retentate pumping loop and a permeate pumping loop, to simulate a backwashing operation, but in a continuous manner rather than the periodic or intermittent manner practiced traditionally. With two parallel flows adjusted so that the pressure drop is the same on the permeate and retentate sides of the module, the pressure profile would be more like that shown in Fig. 17.20b instead of the conventional membrane process system's pressure profile shown in Fig17.20a. The flux would remain high and the rejection would remain low.

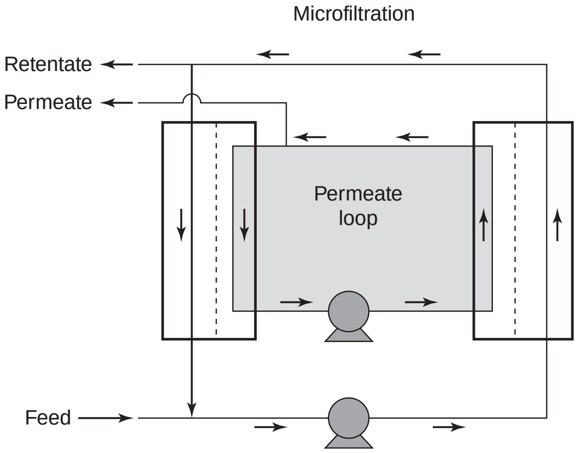


Figure 17.19 Uniform transmembrane pressure (UTP) membrane process system (*Source*: UNIDO, 1995a).

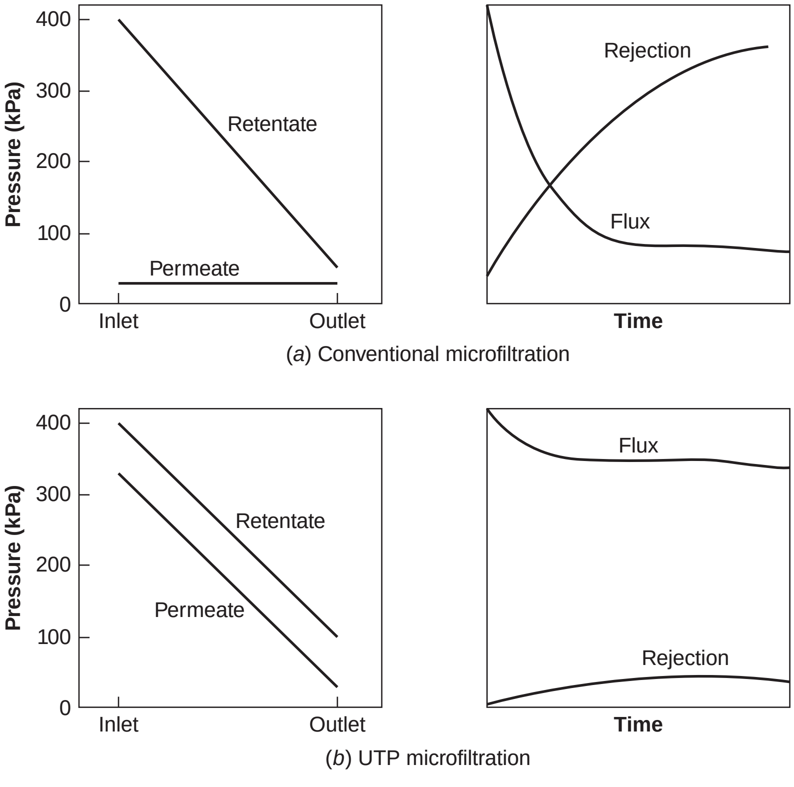


Figure 17.20 Pressure profile and performance characteristics of conventional MF and UTP-MF (*Source*: UNIDO, 1995a).

EXAMPLE 17.2 FLUX DETERMINATION IN MEMBRANE FILTRATION

The permeate flow through an array of reverse osmoses (RO) pressure vessels is 1.50 MGD (5.677 MLD). Feedwater first flows to 30 vessels operated in parallel. The concentrate from the 30 first-pass vessels is combined and sent to a set of 10 second-pass vessels. Each vessel is 8 in. (200 mm) in diameter and 40 in. (1,000 mm) long, thus providing 325 ft^2 (30.2 m^2) of membrane surface area per element. Determine the average membrane flux rate for the RO system in gpd/ft^2 and in L/h/m^2.

Solution 1 (US Customary System):

Q_p = permeate flow = 1,500,000 gpd.

Number of vessels = 30 + 10 = 40.

Number of elements per vessel = 6.

Membrane area per element = 325 ft^2.

Membrane area in RO system = $40 \times 6 \times 325 = 78{,}000$ ft^2.

Average flux rate $F = Q_\text{p}/A$

= 1,500,000/78,000

= 19.2 gpd/ft^2.

Solution 2 (SI System):

Q_p = permeate flow = 5,677,000 L/d = 236,600 L/h.

Number of vessels = 30 + 10 = 40.

Number of elements per vessel = 6.

Membrane area per element = 30.2 m^2.

Membrane area in RO system = $40 \times 6 \times 30.2 = 7{,}250$ m^2.

Average flux rate $F = Q_\text{p}/A$

= 236,600/7,250

= 32.6 L/h/m^2.

EXAMPLE 17.3 DETERMINATION OF MINERAL REJECTION

A membrane mineral filtration system is suggested to reduce the total dissolved solids (TDS) from the feedwater at low rejection rates (in the vicinity of 80%) and low operating pressure (less than 150 psi or 1,040 kPa). The membranes that fit these criteria are nanofiltration membranes. Determine the % mineral rejection rate of the NF plant assuming the feedwater contains 1,400 mg/L TDS and the product water concentration is 200 mg/L.

Solution:

C_p = 200 mg/L.

C_f = 1,400 mg/L.

$R = [1 - (C_\text{p}/C_\text{f})] \times 100$

$= [1 - (200/1{,}400)] \times 100$

$= (1 - 0.14) \times 100$

= 86% mineral rejection rate.

EXAMPLE 17.4 DETERMINATION OF WATER RECOVERY

Recovery is defined as the percentage of feedwater which is recovered as product water. Estimate the % recovery of an RO system with a 4–2–1 arrangement (Fig. 17.21). If the feedwater flow is 6 MGD (22.7 MLD) and the product flow is 5 MGD (18.9 MLD). Discuss the water recovery rate versus the brine concentration.

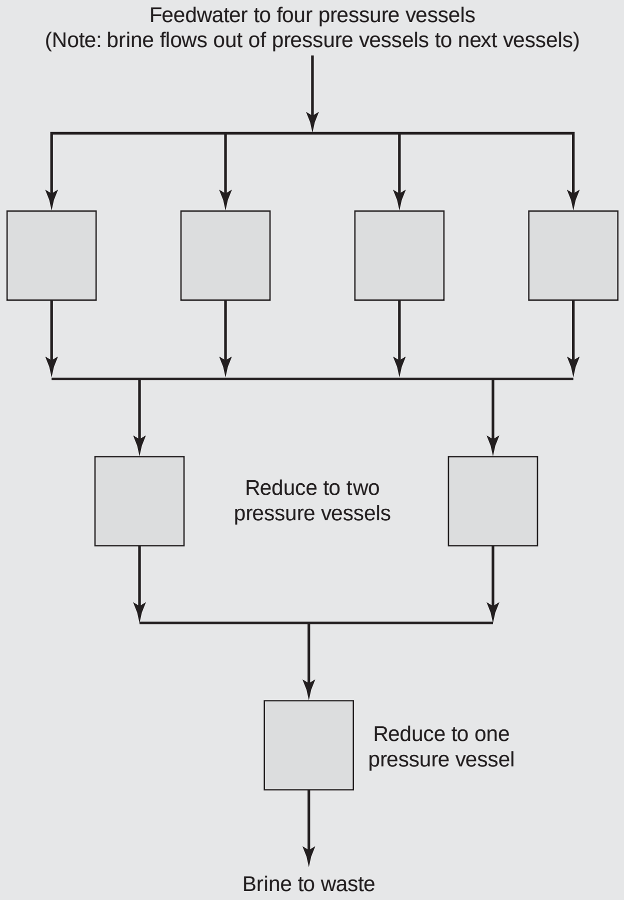


Figure 17.21 Typical 4–2–1 arrangement of pressure vessels. Note: Product water (not shown) flows out of each pressure vessel into a common header.

Solution 1 (U S Customary System):

$Q_f = 6$ MGD.

$Q_p = 5$ MGD.

Water recovery $Y = (Q_p/Q_f) \times 100$

$= (5/6) \times 100$

$= \mathbf{83.3\%}$.

Solution 2 (SI System):

$Q_f = 22.7$ MLD.

$Q_p = 18.9$ MLD.

Water recovery $Y = (Q_p/Q_f) \times 100$

$= (18.9/22.7) \times 100$

$= \mathbf{83.3\%}$.

The higher the water recovery, the higher the brine concentration will be. However, there is a limit. The water recovery rate must be determined taking the following two conditions into consideration:

1. Desired water quality: The amount of mineral passing through the membrane is influenced by the concentration differential between the brine concentration and the product mineral content. It is possible that the product water quality criteria are exceeded with excessive water recovery.
2. Solubility limits of minerals in the brine: The operator should not concentrate the brine to a degree that precipitates minerals on the membrane, causing the so-called concentration polarization.

EXAMPLE 17.5 DETERMINATION OF THE TYPE OF A FULL-SCALE MEMBRANE FILTRATION PLANT

A membrane filtration plant (MF, UF, NF, or RO) is to be installed for a municipality. How do you decide the type of membrane filtration plant required for removal of *Cryptosporidium* (3–7 microns) and *Giardia* cysts (6–10 microns)?

Solution:

The rules and regulations vary in different countries. The readers should inspect the specific national (Federal), provincial (State), and local public health laws. In the United States under the Long Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR), water systems may be required to achieve as much as 3 log additional *Cryptosporidium* removal and/or inactivation credit depending on the results of source water quality monitoring and the subsequent Bin assignment. Thus when combined with the prescribed *Cryptosporidium* treatment credit awarded to a system in compliance with the existing public health laws, as applicable, the total *Cryptosporidium* treatment credit required for a system in Bins 2, 3, and 4 is 4 log, 5 log, and 5.5 log, respectively.

Since *Giardia* cysts (6–10 microns) are bigger than *Cryptosporidium* (3–7 microns), it is assumed that if *Cryptosporidium* can be removed, *Giardia* cysts will be removed simultaneously.

Membrane filtration is one of several toolbox options that has been determined to be capable of achieving the maximum required credit for *Cryptosporidium* removal as a stand-alone process. Membrane filtration is defined under the rule as a pressure- or vacuum-driven separation process in which particulate matter larger than 1 μm is rejected by an engineered barrier, primarily through a size exclusion mechanism, and which has a measurable removal efficiency of a target organism that can be verified through the application of a direct integrity test. This definition includes the following membrane processes commonly used in drinking water treatment: MF, UF, NF, and RO. In order to receive *Cryptosporidium* removal credit under the rule, a membrane filtration system must meet the following three criteria: (a) the process must comply with the definition of membrane filtration as stipulated by the US Federal rule, (b) the removal efficiency of a membrane filtration process must be established through a product-specific challenge test and direct integrity testing, and (c) the membrane filtration system must undergo periodic direct integrity testing and continuous indirect integrity monitoring during operation.

The suitable type of membrane filtration MF, UF, NF, or RO for the removal of *Cryptosporidium* in compliance with the US Federal rule can then be selected based on the testing results of (a) challenge testing, (b) direct integrity testing, and (c) continuous indirect integrity monitoring.

The manufacturers of the membrane filtration units or the responsible consulting engineer, in cooperation with the municipal engineer, should provide the testing results to the local and state public health agencies for approval of the selected membrane filtration plant.

EXAMPLE 17.6 DETERMINATION OF MEMBRANE FILTRATION PRESSURES

An RO plant, shown in Fig. 17.18 is operated under high feed pressure of 297.5 psi (2,067 kPa). The retentate (concentrate) pressure and the permeate (product water) pressure are 257.5 psi (1,787 kPa) and 200 psi (1,388 kPa), respectively. What are the transmembrane pressure and the pressure drop across the membrane unit?

Solution (U S Customary System):

$$\begin{aligned}\text{Transmembrane pressure } \Delta P_{\mathrm{T}} &= [(P_{\mathrm{i}} + P_0)/2] - P_{\mathrm{p}}\\ &= [(297.9 + 257.5)/2] - 200\\ &= 77.7 \text{ psi.}\end{aligned}$$

$$\begin{aligned}\text{Pressure drop} &= \Delta P = P_{\mathrm{i}} - P_0\\ &= 297.9 - 257.5\\ &= \mathbf{40.4\ psi}.\end{aligned}$$

Solution (SI System):

$$\begin{aligned}\text{Transmembrane pressure } \Delta P_{\mathrm{T}} &= [(P_{\mathrm{i}} + P_0)/2] - P_{\mathrm{p}}\\ &= [(2{,}067 + 1{,}787)/2] - 1{,}388\\ &= 539 \text{ kPa.}\end{aligned}$$

$$\begin{aligned}\text{Pressure drop} &= \Delta P = P_{\mathrm{i}} - P_0\\ &= 2{,}067 - 1{,}787\\ &= \mathbf{280\ kPa}.\end{aligned}$$

EXAMPLE 17.7 SITUATION OF ZERO REJECTION FOR A MEMBRANE FILTRATION SYSTEM

What is the rejection rate of an MF system for removal of sodium chloride from an aqueous solution if the feed concentration C_f equals the permeate (product water) concentration C_p?

Solution:

$$\text{Rejection rate} = R = [1 - (C_p/C_f)] \times 100$$
$$R = [1 - 1] \times 100$$
$$= \mathbf{0\%}.$$

The MF system cannot reject any sodium chloride.

EXAMPLE 17.8 SITUATION OF TOTAL REJECTION FOR A MEMBRANE FILTRATION SYSTEM

What is the rejection rate of an RO system for removal of contaminants from a feedwater if the feed concentration C_f is 22,000 mg/L and the permeate (product water) concentration C_p is zero?

Solution:

$$\text{Rejection rate} = R = [1 - (C_p/C_f)] \times 100$$
$$= [1 - (0/22{,}000)] \times 100$$
$$= \mathbf{100\%}.$$

The RO system has 100% rejection rate for removal of a specific contaminant.

17.7.4 Maintenance of Membrane Filtration Systems

All aspects of fouling involve the trapping of some type of material within the membrane process device itself or on the surface of the membrane. Since the causes, symptoms, and cures are different, the following five types of fouling are generally considered: (a) membrane scaling; (b) metal oxides fouling; (c) colloidal fouling; (d) biological fouling; and (e) cleaning agent fouling.

The problems of membrane scaling and metal oxides fouling are common to all membrane process systems, regardless of the type or source of feedwater stream to the systems (i.e., sea water or fresh water). Colloidal and biological foulings are specifically important in the design of a membrane process system for treating food processing water and wastewater. Cleaning/sanitation is an art or science of its own. To establish the optimal cleaning regime, it behooves the operator to understand clearly the nature and extent of the interaction of cleaning agents with the membrane.

1. *Membrane scaling*: Precipitation of dissolved salts resulting from exceeding the solubility limits in a concentrated stream. This may occur because the salts in feedwater are normally concentrated from two to four times. This is a function of system recovery, 50% resulting in a concentration of 2 and 75% resulting in a concentration of 4. The most common precipitates encountered are those of calcium carbonate, calcium sulfate, and silica. Calcium carbonate precipitation can be prevented by lowering the feed pH with addition of hydrochloric acid. Carbonates and bicarbonates are then both converted to carbon dioxide gas, and pass through the membrane without difficulty. The precipitation of calcium sulfate can be retarded with addition of a sequestering agent, such as sodium hexametaphosphate. The silica precipitation can be prevented by reducing system recovery to keep silica concentrations below saturation.
2. *Metal oxides fouling*: The oxide or hydroxide form of iron, manganese, or aluminum are the most common problem compounds. Operation of the membrane process system at acidic pH levels and possibly iron and manganese removal by oxidation are recommended pretreatment alternatives.
3. *Colloidal fouling*: Colloidal fouling is caused by the entrapment of colloids on the membrane surface in a membrane process device, or specifically by the coagulation of the colloids during the membrane process. Reduction of the concentration of colloids can be accomplished by the following pretreatment schemes: (a) filtration, using sand, carbon, or other filter media; (b) in-line coagulation with iron, salts, and

polyelectrolyte aids, followed by pressure filtration; and (c) iron salt coagulation, flocculation, flotation (or sedimentation), and filtration.

4. *Biological fouling*: Biological fouling is caused by the growth of microorganisms in membrane modules. Two major factors are considered: (a) biological fouling or plugging, which interferes with the hydraulic flow of the membrane modules; and (b) biological attack of the membrane, which is extremely serious when cellulose acetate membranes are attacked by the enzyme system of microorganisms. Periodically feeding a cleaning solution containing free chlorine residuals of 0.5–1.0 mg/L is recommended to prevent detrimental biological growths on cellulose acetate membrane surface. Gas chlorine or liquid sodium hypochlorite has been used successfully for this purpose. It should be noted that thin-film composite membranes containing a polyamide separating barrier on a polysulfone or polyethylene supporting layer generally give better RO performance with regard to temperature stability, pH stability, and cleanability, but have almost zero chlorine resistance. Generally, hydrophobic membranes consisting of polysulfone and polyamides may be more prone to biological fouling than hydrophilic membranes consisting of cellulosics.
5. *Cleaning agent fouling*: Sometimes membranes "fouling" problems may actually be "cleaning" problems. Ideally, the membrane material, and all other impurities contact surfaces, should be compatible with normal process equipment cleaners, such as mild caustic or acid solutions. Severe cleaning problems may require the use of enzyme detergents.

17.7.5 Membrane Filtration Applications

One of the membrane filtration processes, ultrafiltration (UF) is selected for the illustration of the process application. It is extremely compact and does not require coagulation. Membrane filters use hollow fiber membranes to remove undissolved, suspended, and emulsified solids. The membranes are typically able to exclude all particles greater than 0.2 microns from the water stream.

Membrane filters are typically used for specialized applications that require highly purified water, and often serve as

1. Pretreatment processes for reverse osmosis
2. Pretreatment to remove colloidal silica from boiler feed water
3. Treatment for drinking water supplies with influent turbidities of 1 NTU or less, and fouling indexes of less than 10

Typically, ground water and surface water of high clarity have fouling indexes of less than 10. Fouling of the hollow fibers by turbidity is the major problem preventing widespread application of this technology.

Traditional membrane filters introduce feed water to the inside of the hollow fiber membrane, with the *permeate* or *filtrate* emerging on the outside of the membrane. State-of-the-art membrane filters are designed to pass influent to either the inside or outside of the membrane. The hollow fiber membranes are contained in a pressure vessel or cartridge and operate over a pressure range of 10–100 psi. Contaminants collect on the end of the hollow fiber and are discharged to waste by reversing the water flow. A sample membrane filtration system is shown in Fig. 17.22.

Periodic backflushing and occasional chemical cleaning are necessary to maintain the membrane fibers. Treatment of the flush water containing solids requires separate coagulation and clarification. The clarified flush water is either recycled or discharged after treatment. The sludge collected from this type of system is typically dried and disposed of in a landfill.

Customarily, 90% of the feed water is collected as permeate; the other 10% is discharged along with the contaminants. These filters can be designed to exclude particles larger than 0.01 μm. Unlike reverse osmosis, this process does not exclude inorganic salts and electrolytes. Hollow fiber filters with the finest membranes remove bacteria, *Giardia, Cryptosporidium*, and some viruses.

The hollow fiber membranes vary in size and porosity and in their corresponding effectiveness, yet all membrane filter fibers are sensitive to influent concentrations of suspended and colloidal solids. Specifically, the *flux* level (the volume of permeate produced per unit area of membrane filter per day) and the flux stability are affected by the (a) quality of the influent, (b) filter cycle duration, and (c) quality of the backwashing water.

In general, influent water with a fouling index of less than 10 will permit filter cycles of 8 h with a 10% reduction in flux. In between backflushes, a high degree of the original fiber porosity can be retained with fast-forward flushes of the influent, which can then be routed to waste. After two to three fast-forward flushes, a full backflush is required to restore the initial flux level.

Membrane filtration is an attractive option for small systems because of its small size and automated operation. Because it is best suited for small systems, many typical membrane systems are skid-mounted units including the following elements:

1. Hollow membranes in cartridges
2. Automatic and manual valves for backwashing and unit isolation
3. Flow meters
4. Pressure gauges
5. Integral supply pump
6. Control panel

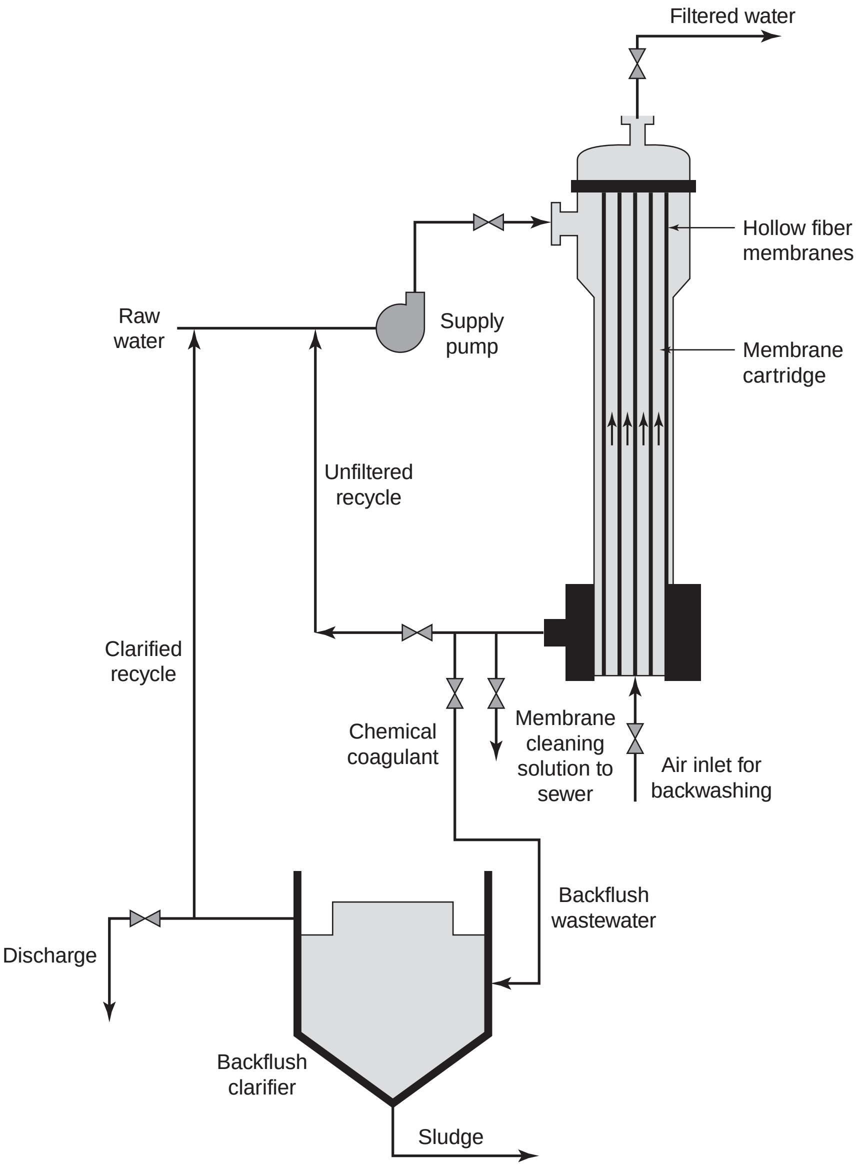


Figure 17.22 Flow diagram of membrane filtration system (*Source*: U.S. EPA, 1990).

Three other necessary components of membrane systems are (a) a separate supply pump and interconnecting piping for plants with multiple filter units, (b) storage tanks and chemical feed pumps for membrane cleaning solutions, and (c) filtered water storage with chlorination capacity.

17.7.6 System Performance of Membrane Filtration

System performance data for membrane filter systems include data on *Giardia, Cryptosporidium,* coliform, and turbidity removal.

1. *Cryptosporidium and Giardia cyst removal*: One Colorado State University's test evaluated *Giardia* removal effectiveness for one manufacturer's UF hollow membrane unit. The 0.2 μm membrane filters were found to be 100% effective in removing *Giardia* cysts at influent concentrations of 1,100 cysts/L. In addition, researchers found that the unit's "radial pulse" cleaning mechanism was effective in preventing membrane fouling and caused no reductions in *Giardia* cyst removal. For *Cryptosporidium* removal, an NF membrane is required.
2. *Coliform removal*: Coliform removal was evaluated for a manufacturer's UF hollow membrane unit. This evaluation used water seeded with *Escherichia coli* bacteria ranging from 20 to 30 million organisms per 100 mL. During a 130-minute test run, the effluent contained less than 1 coliform bacterium per 100 mL. No membrane breakthrough was experienced during three test runs.

3. *Turbidity removal*: Turbidity removal was also evaluated by Colorado State University. The test results of this system found that (a) turbidity up to 30 NTU created from the introduction of bentonite clay was reduced to 0.2 NTU, (b) actual field tests with 190 NTU influent turbidity resulted in effluent with 0.6 NTU, and (c) tests with effluent ranging from 2.4 to 3.0 NTU resulted in effluent of 0.25–0.57 NTU.

17.7.7 Potential Problems of Membrane Filtration

The primary application concerns with membrane filters are membrane failure and organics removal effectiveness. Pilot testing is needed to qualify each application, because of the lack of experience with the process. Membrane failure is a major concern because the membrane is the only barrier between potentially pathogenic microbial contaminants and the finished water. Most other treatment systems have multiple barriers to pathogenic breakthrough. To guard against this potential problem, some membrane systems include features that trigger an operational shutdown or activate an alarm. One manufacturer includes a manual or automatic device to verify the integrity of each membrane.

The second concern is that membrane filters may not be effective in removing certain organic components. Larger membranes of about 0.2 μm will not effectively remove organic materials contributing color. Smaller inside-to-outside membranes of about 0.1 μm will remove smaller particles, but when influent contains organic materials that contribute to color, such as humic or fulvic acids, supplemental treatments may be necessary.

Pilot testing of membrane systems is generally necessary to establish design criteria and operating parameters.

17.8 SELECTING THE APPROPRIATE FILTRATION TREATMENT SYSTEM

This section discusses considerations in selecting an appropriate filtration technology. First, it discusses the steps involved in selecting any filtration system. Next, it discusses the role and objectives of pilot studies, flocculation and sedimentation studies, and filtration studies in selecting the specific operating characteristics for selected filtration technologies.

17.8.1 Steps in an Evaluation

The first step in selecting a new water treatment technology is to review all raw water quality data to establish the requirements for the potential alternatives. Next, a list of alternative technologies should be compiled. The considered alternatives must be able, in theory or as proven under similar conditions, to solve the problems identified with the current filtration system. Once the potential alternatives are selected, one must determine the necessity of pilot or bench-scale tests. If the desired performance of one or more of the alternatives is in doubt, testing is appropriate. (Testing is always useful if time and budget allow.) Otherwise, literature surveys, bench-scale studies, or pilot test results can be used to derive performance characteristics and design considerations for each alternative. For small systems, the alternatives for a particular application may include slow sand filters, package plants, diatomaceous earth filtration, or membrane filters.

Following this initial selection, the basic process concerns for the various alternatives should be identified, including the following:

1. Turbidity removal performance
2. Color removal performance
3. *Giardia* and *Cryptosporidium* removal performance
4. Cleaning cycle frequency
5. Necessary chemicals
6. Chemical dosages
7. Requisite operational skill
8. Applicable regulatory standards
9. Necessary sludge management

In the next stage, conceptual designs and preliminary layouts for selected alternatives are prepared. Also, construction costs for the alternatives should be developed. One should compare all alternatives for reliability, simplicity, flexibility, and ease of implementation, to select the appropriate application. Finally, one can proceed with the final design.

17.8.2 Need for Pilot Studies

A pilot study is a broad term used for small-scale testing of either complete water treatment processes or merely individual processes. Pilot studies are used to evaluate alternate treatment options and operating techniques. Pilot tests can be relatively short or very long in duration. For example, time requirements for pilot tests of rapid filtration are about 2 weeks; for slow sand filtration, 2–4 months; and for corrosion inhibitors, up to 6 months. These tests may be continuous or intermittent. The longer, more involved tests are more expensive. To avoid unnecessary costs, pilot tests should have clearly delineated objectives to prevent including extraneous evaluations. Yet they should be performed long enough to encounter the full range of raw water conditions and process design parameters.

Simple jar tests are usually sufficient to evaluate procedures for traditional coagulation, flocculation, sedimentation, and filtration processes. However, new or innovative technologies usually require the more extensive pilot tests.

Pilot tests have been endorsed by field experts for many years to assess precise design specifications, operational recommendations, and chemical applications procedures. In addition, pilot tests are required for some technologies to adjust individual processes to specific local water conditions. Pilot tests are also used to identify unforeseen design and operating problems, demonstrate operation to regulatory authorities, and develop better information concerning capital and operating costs. Pilot studies may also be a prerequisite to obtaining conditional regulatory approval. They are especially needed for new and emerging technologies and for accepted technologies with innovative elements, such as tube settlers or mixed-media filters.

Pilot studies are important to ensure the suitability of a small-scale prototypical plant for a particular application. This is especially true for raw water with difficult treatment aspects or poor quality, such as highly polluted water with high concentrations of organics, iron, manganese, and algae. Pilot tests are also particularly necessary for plants that use new short-detention and high-rate processes or where direct filtration is being considered.

17.8.3 Flocculation, Sedimentation, and Flotation Studies

Full-scale flocculation and sedimentation/flotation studies are necessary because it is difficult to extrapolate the performance of pilot-scale flocculation and sedimentation/flotation tanks to full-scale plants. Since flocculator efficiency is directly related to volume, smaller flocculators perform more efficiently than their full-scale counterparts. Consequently, pilot flocculation studies do not provide results applicable to full-scale facilities.

Traditional sedimentation basins present even greater extrapolation problems. Since small-scale versions cannot duplicate the process of traditional basins, which are generally 2.7–4.5 m (9–15 ft) deep, full-scale sedimentation studies are also necessary. However, tube settlers can be evaluated on a pilot scale.

17.8.4 Filtration Studies

Filtration studies can successfully employ pilot tests. They are necessary for plants considering direct filtration and very useful for plants with unusual raw water characteristics. One of the problems that can be identified and evaluated with pilot tests is the presence of diatoms (filter clogging algae) that do not show up as high turbidity, yet can clog filters. Another such problem involves the presence of paper fiber, a common situation for water intakes below paper plant effluent outfalls. These fibers also may not show up as high turbidity but present filter clogging problems. Filtration pilot tests establish whether the presence of paper fiber or diatoms will cause problems.

Side-by-side pilot filters will provide useful comparisons of different filter media designs being considered for a particular application. Each of the filters can be operated at the same flow rate and is designed to maintain a constant flow as head loss increases. Pilot filters also include provisions to measure head loss. Measurements of the filtered water turbidity and filter head loss are two of the most useful results of side-by-side pilot tests to predict filtration efficiency and filter run length.

Pilot tests are especially recommended when high-rate short contact time clarification/filtration processes are being considered.

PROBLEMS/QUESTIONS

17.1 In a modern world, new knowledge and technologies are discovered and/or developed every day. A modern engineer learns alternative filtration technologies everyday and everywhere using all means. Study this chapter carefully and conduct an Internet/library search to prepare yourself for answering the following questions. Write an engineering report to summarize all questions and answers. The sources of your answers must be documented in detail. Graphical illustrations for clarification of explanations are encouraged.

- **(a)** What is reverse osmosis? What is osmosis?
- **(b)** What is demineralization?
- **(c)** What is brackish water? Why is sea water more expensive to treat than brackish water using the membrane process?
- **(d)** What are the common membrane materials?
- **(e)** What are the common membrane configurations?
- **(f)** What are the pressure-driven membrane processes?
- **(g)** What are the electrically driven membrane processes?
- **(h)** What are dead-end filtration and cross-flow filtration?
- **(i)** What is the electrolysis process? What is the dialysis process? What is the difference between the two?
- **(j)** What is an ion exchange membrane? What are the ion exchange membrane's three components?
- **(k)** What are the anion exchange membrane, cation exchange membrane, and bipolar membrane?
- **(l)** What is membrane fouling? Is membrane fouling reversible?
- **(m)** What are the physical/mechanical factors affecting the flux of a membrane filtration process?
- **(n)** What is membrane process concentration polarization?
- **(o)** What is the rejection rate of a membrane filtration process? What are the factors affecting mineral rejection?
- **(p)** What are the following: (i) membrane module pressure loss or pressure drop, (ii) transmembrane pressure, and (iii) operating pressure for a membrane filtration system?
- **(q)** What is the molecular weight cut-off in the membrane filtration process?
- **(r)** What are the actual pore size and nominal pore size in the membrane filtration process?
- **(s)** What are microporous membranes and asymmetric membranes? What are the differences between the two?
- **(t)** What are the similarities and differences between ultrafiltration and reverse osmoses in terms of operating pressure ranges, membrane requirements, contaminants/impurities to be rejected, chances of membrane fouling, chances of concentration polarization, volumetric flux, and cost?

(u) What is membrane bioreactor (MBR)? Are there any potable water treatment applications?

(v) What is the problem of membrane hydrolysis? How does hydrolysis influence the mineral rejection capability of a membrane?

(w) What are the general reverse osmoses pretreatment requirements?

(x) What are the common methods for handling and disposal of the brine wastewater generated from a membrane filtration plant.

17.2 Convert a membrane's water flux of 25.5 gpd/ft^2 to g/s/cm^2.

17.3 Estimate the ability of a reverse osmosis plant to reject minerals by calculating the mineral rejection as *a*%, assuming that the feedwater contains 2,000 mg/L total dissolved solids (TDS) and the product water TDS is 130 mg/L.

17.4 Estimate the percent recovery of a reverse osmosis plant with a 4–2–1 arrangement if the feedwater flow is 4 MGD (15.14 MLD), and the product flow is 3.5 MGD (13.24 MLD).

17.5 Take a field trip to visit Pittsfield Water Treatment Plant in Pittsfield, MA, USA (or a similar local alternative filtration plant), either in person or visually through the Internet. Determine and discuss the following:

(a) Complete process flow diagram, water supply sources, flow rates, and raw water quality

(b) Pretreatment, coagulation/flocculation and flotation clarification, chemical feeding, chemicals, detention times, and equipment prior to sand filtration

(c) Automatic backwash filtration operational procedures and backwash rates (gpd/ft^2 and L/d/m^2)

(d) Filtration media details (sand type, area, depth, etc.)

(e) Filter backwash water handling and disposal methods

(f) Typical water quality data (effluent, float, and sludge)

(g) Total waste sludge flow versus influent raw water flow

(h) Dimensions and detention time of each flotation-filtration package plant unit

(i) Filtration rate (gpd/ft^2 and L/d/m^2) with and without filter backwash

17.6 Take a field trip to visit the Reverse Osmosis Water Treatment Plant of the City of Cape Coral, FL, USA (or a similar local reverse osmosis plant), either in person or visually through the Internet or other means. Determine and discuss the following:

(a) Complete process flow diagram, water supply sources, flow rates, and raw water quality

(b) Pretreatment details (carbonate scale prevention, sulfate scale prevention, and cartridge filtration) prior to membrane filtration

(c) Membrane filtration details (membrane types, number of trains/skids, configurations, operating pressures, etc.)

(d) Percentage of water recovery of RO

(e) Post-reatment after RO (air stripping, disinfection, and neutralization)

(f) Brine discharge practice

(g) Current operation and maintenance costs (chemicals, labor, power, and total costs)

17.7 What are the options of pretreatment prior to membrane filtration?

17.8 What are the options of posttreatment after membrane filtration?

17.9 Conduct an Internet/library study to survey at least the following commercially available innovative and alternative filtration technologies and write an engineering report with published references. Prepare a Power Point file for a technical presentation to the class. The instructor may also assign each student to research one or two innovative and alternative filtration process (es) for detailed investigation.

(a) Granular activated carbon filtration

(b) Biological granular activated carbon filtration

(c) Pressure filtration

(d) Green sand filtration

(e) Automatic backwash filtration

(f) Bank filtration

(g) River bed filtration

(h) Self-cleaning filtration

(i) Electroflotation-filtration

(j) Dual sand filtration

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Chapter 18

Disinfection and Disinfection By-products Control

18.1 PURPOSE OF DISINFECTION

The disinfection of water and wastewater is without doubt the most potent weapon in the hands of health and water authorities against waterborne infection. Since 1908, the chlorination of water, in particular, has led to the substantial eradication of waterborne diseases in the United States at small cost in equipment, materials, and personnel.

For convenience of presentation, the chemical oxidation of odors, the reduction of BOD (biochemical oxygen demand), and the boosting of treatment efficiency by chlorine and other chemical agents are also made part of this chapter.

Sterilization is not synonymous with *disinfection*. It implies the destruction of all living things in the medium sterilized. Production of sterile water is generally confined to research, to medical practice, and to the manufacture of pharmaceuticals and fine chemicals. Drinking water need not be sterile. The purpose of potable water disinfection is to produce water suitable for human consumption as well as commercial and industrial applications. The purpose of wastewater and sludge disinfection is to destroy *pathogens* and thus prevent the spread of waterborne diseases in the environment.

18.2 PATHOGENS, DISINFECTION, AND DISINFECTANTS

Pathogens of all kinds and classes are removed physically from water, wastewaters, and sludge in varying degree by most conventional treatment processes. Pathogens also die away or are destroyed in significant numbers in the course of treatment. By contrast, purposeful disinfection of water and wastewater has the specific mission of killing, selectively if necessary, those living organisms that can spread or transmit infection through or in water. Within the context of this book, this primary concern is twofold: (1) to prevent direct transmission of disease to man through water and (2) to break the chain of disease and infection by destroying responsible infective agents before they reach the water environment. Disinfection of wastewaters is receiving increased attention as populations grew and recreation as well as urbanization intensified. The safety of swimming-pool waters and of market shellfish and farm produce pose questions of their own.

As shown in Table 18.1, three categories of human enteric pathogens (pathogenic organisms) are normally of consequence: bacteria, viruses, and intestinal parasites. Most of these pathogens and the diseases they transmit are no longer a problem in the United States due to proper environmental protection. However, many developing countries still experience serious outbreaks of various waterborne diseases shown in Table 18.1. Purposeful disinfection must be capable of destroying all three. Fortunately there are disinfectants that can do this.

To be of practical service, such water and wastewater disinfectants must possess the following properties:

1. They must destroy the kinds and numbers of pathogens that may be introduced into municipal water or wastewater and do so within a practicable period of time, and over an expected range in water temperature, while meeting possible fluctuations in composition, concentration, and condition of the waters or wastewaters to be treated.
2. They must be neither toxic to man and his domestic animals nor unpalatable or otherwise objectionable in required concentrations.
3. They must be dispensable at reasonable cost, and safe and easy to store, transport, handle, and apply.
4. Their strength or concentration in the treated water must be easily, quickly, and preferably automatically determinable.
5. They must either persist within disinfected water in a sufficient concentration to provide reasonable

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Table 18.1 Pathogenic organisms and diseases transmitted by water, wastewater, and sludge

Bacteria
Salmonella (salmonellosis)
Shigella (bacillary dysentery)
Bacillus typhosus (typhoid fever)
Salmonella paratyphi (paratyphoid)
Vibrio cholerae (Cholera)

Viruses
Enterovirus
Poliovirus
Coxsackievirus
Echovirus
Andenovirus
Reovirus
Infectious hepatitis

Intestinal parasites
Entamoeba histolytica (amoebic dysentery)
Giardia lamblia (giardiasis)
Ascaris lumbricoides (giant roundworm)
Cryptosporidium (cryptosporidiosis)

Source: US EPA.

> *residual* protection against its possible recontamination before use; or, because this is not a normally attainable property, the disappearance of residuals must be a warning that recontamination may have taken place.

As a rule the concentration of disinfectants, depending on their nature, is determined by physical measurements or chemical analyses, whereas their disinfecting efficiency in all but experimental studies is assessed by the reduction of *indicator organisms* (usually *coliform organisms*) to numbers implying statistically acceptable safety against possible infection. Correlative laboratory studies of the destruction of pathogens are performed either with laboratory cultures or with organisms harvested from carriers or cases of disease. To be equally significant with harvested organisms, cultured organisms must be known or shown to be equally resistant.

Water, wastewater, and sludge can be disinfected in a number of categorical ways. A list of the most practical ones includes heat and light, as well as chemicals.

Primary disinfectants are those used for the inactivation of *Giardia* cysts, viruses, and bacterial contaminants, while *secondary disinfectants* suppress biological regeneration in the distribution system or environment. Common primary disinfectants are chlorine, chlorine dioxide, ozone, and UV radiation. Secondary disinfectants include chlorine, chlorine dioxide, and chloramines.

18.3 DISINFECTION BY HEAT

The boiling of water at 127°C or 220°F for 20 min at 20 psi (1,034 mm Hg) pressure, as in an autoclave, will provide sterilization of the autoclave contents. Raising water to its boiling point (100°C or 212°F) for 5 min will disinfect it. This is resorted to as an emergency measure in the form of boil-water orders by health and water authorities.

18.4 DISINFECTION BY ULTRAVIOLET LIGHT

Sunlight is a natural disinfectant, principally as a desiccant. Irradiation by ultraviolet (UV) light intensifies disinfection and makes it a manageable undertaking. The most common source of UV light is a mercury-vapor lamp constructed of quartz or special glass likewise transparent to the intense and destructive, invisible light of 2,537 angstrom units (10^{-8} cm) emitted by the mercury-vapor arc. To ensure disinfection, the water must be free from light-absorbing substances, from phenolic and other aromatic compounds, and from suspended matter that shades the organisms against the light; the time–intensity product of *exposure* must be adequate, and the water must be well mixed while it is being exposed in relatively thin films in order to counter its own absorptivity.

UV radiation is an effective bactericide and virucide, but an ineffective cysticide. Consequently, it is recommended as a primary disinfectant mainly for groundwaters not directly influenced by surface waters (where there is no risk of *Giardia cyst* contamination). UV radiation (254 nm wavelength) penetrates the cell wall and is absorbed by the cellular nucleic acids. Radiation absorption prevents replication, thus killing the cell. Since UV radiation is not a chemical agent, it produces no toxic residual, known as *disinfection by-products* (DBP). Major advantages of UV radiation are its simplicity, lack of impact on the environment and aquatic life, and minimal space requirements. In addition, the required UV contact times are seconds rather than minutes. The UV equipment is simple to operate and maintain if the apparatus is cleaned properly on a regular basis.

18.5 DISINFECTION BY CHEMICALS

Exposing water long enough to adequate concentrations of chemicals of the following kinds will disinfect it.

18.5.1 Oxidizing Chemicals

These comprise (1) the halogens—chlorine, bromine, iodine, monochloramine, and chlorine dioxide—released in suitable form from acceptable sources; (2) ozone; and (3) other oxidants such as potassium permanganate and hydrogen peroxide, but these are not as effective as the halogens and ozone.

Among the halogens, gaseous chlorine and a number of chlorine compounds are economically most useful. Bromine (Br_2) has been employed on a limited scale for the disinfection of swimming-pool waters; iodine has been used for the disinfection of swimming pools and small quantities of drinking water in the field. Tablets of tetraglycine hydroperiodide have been developed for field use. Ozone is a good but relatively expensive disinfectant that normally leaves no measurable monitoring residual. However, the ability of ozone to destroy a number of objectionable odors and to bleach color effectively is in its favor. Potassium permanganate, too, is a relatively expensive disinfectant.

It is an axiom of disinfection by chemicals that the oxidizing capacity of a compound is not necessarily a measure of its disinfecting efficiency. Thus hydrogen peroxide is a strong oxidant but a poor disinfectant.

18.5.2 Metal Ions

Silver ions are neither viricidal nor cysticidal in acceptable concentrations, but they are bactericidal. Disinfection at the low concentrations employed as low as 15 μg/L (micrograms per liter or, closely, parts per billion) is slow. This is a weakness of silver. Moreover, silver is costly at practicable concentrations. Copper ions are strongly algicidal but only weakly bactericidal.

18.5.3 Alkalis and Acids

Pathogenic bacteria do not last long in highly alkaline or highly acid waters, for example, at very high (>11) or very low (<3) pH values. The destruction of bacteria by caustic lime incidental to lime softening is an example.

18.5.4 Surface-Active Chemicals

Among surfactants, the cationic quaternary ammonium compounds are strongly destructive, the anionic surfactant only weakly so. The neutral surfactant occupy an intermediate position. Cationic surfactant have been applied selectively as disinfectants for disinfecting wash waters and rinse waters of eating establishments, swimming pool water, WTP facilities and instrumentation due to cationic surfactants, non-oxidative, non-corrosive and long-lasting nature (Wang et al., 1975).

18.5.5 Advanced Oxidation Processes

Advanced oxidation processes (AOPs) involve the use of more than one disinfectant for oxidation and disinfection. Typical effective AOPs combinations include ozone plus hydrogen peroxide, ozone plus UV, ozone plus permanganate, hydrogen peroxide plus UV, and chlorine plus UV. Various AOPs have been used considerably to disinfect pathogens and to remove both low and high concentrations of organic compounds from diverse sources such as water, wastewater, and sludge.

For the routine disinfection of municipal and industrial waters and wastes, only chlorine is both efficient and reasonably cheap; ozone is efficient but relatively expensive and not persistent enough for monitoring purposes; and heat and UV light are relatively still more expensive and without monitoring properties. What makes heat especially useful is that it can usually be made available at a moment's notice in times of emergency. Chlorine and hypochlorite compounds are the disinfectants of choice in US practice. Table 18.2 is an overview of common disinfectants for water treatment. Table 18.3 shows the advantages and disadvantages of chlorine, ozone, UV, chlorine dioxide, and chloramines.

Table 18.2 Overview of water filtration and disinfection technologies to control microbial contamination

Technological options to meet regulatory requirements	Stage of acceptability	Size suitability	Comments
Conventional filtration	Established	All	Most common; adaptable for adding other processes
Direct filtration	Established	All	Lower cost alternative to conventional filtration
Slow sand filtration	Established	Especially small, but all sizes	Operationally simple; low cost, but requires large land areas
Package plant filtration	Established	Mostly small	Compact, variety of process combinations available
Diatomaceous earth filtration	Established	Mosty small	Limited applicability; potentially expensive for small systems
Membrane filtration	Emerging	Mostly small	Expensive
Cartridge filtration	Emerging	Small	Expensive
Chlorine	Established	All	Most widely used method; concerns about health effects of by-products
Chlorine dioxide	Established	All	Relatively new to the United States; concerns about inorganic by-products
Monochloramine	Established	All	Secondary disinfectant only; some by-product concerns
Ozone	Established	All	Very effective and requires a secondary disinfectant
Ultraviolet radiation	Established	All	Simple, no established harmful by-products and requires secondary disinfectant
Advanced oxidation (ozone plus H_2O_2 and ozone plus ultraviolet radiation)	Emerging	All	Not much information concerning disinfection aspects of this process

Source: US EPA.

Table 18.3 Advantages and disadvantages of five disinfectants

Disinfectant	Advantages
Chlorine	Effective. Widely used. Variety of possible application points. Inexpensive. Appropriate as both primary and secondary disinfectant.
Ozone	Very effective. Minimal harmful by-products identified to date. Enhances slow sand and GAC filters. Provides oxidation and disinfection in the same step.
Ultraviolet radiation	Very effective for viruses and bacteria. Readily available. No harmful residuals. Simple operation and maintenance.
Chlorine dioxide	Effective. Relatively low cost. Generally does not produce THMs.
Chloramines	Mildly effective for bacteria. Long-lasting residual. Generally does not produce THMs.
Disinfectant	**Disadvantages**
Chlorine	Harmful halogenated by-products. Potential conflict with corrosion control pH levels, when used as a secondary disinfectant.
Ozone	Requires secondary disinfectant. Relatively high cost. More complex operations because it must be generated on-site.
Ultraviolet radiation	Inappropriate for water with *Giardia* cysts, high suspended solids, high color, high turbidity, or soluble organics. Requires a secondary disinfectant.
Chlorine dioxide	Some harmful by-products. Low dosages may make it ineffective. Must be generated on-site.
Chloramines	Some harmful by-products. Toxic effects for kidney dialysis patients. Only recommended as a secondary disinfectant. Ineffective against viruses and cysts.

Source: US EPA.

Table 18.4 indicates the application consideration of the same five common disinfectants.

Although the disinfection of water and wastewater is aimed almost wholly at the destruction of single-celled organisms, the killing of the cercariae of the schistosomes and of the adult guinea worms infesting *Cyclops* are of much importance in some parts of the world. Even if these large organisms are included, however, the protoplasmic mass involved is relatively so small that chemical disinfectants can perform successfully in minute concentrations in short periods of time (fractions of a mg/L of free chlorine in otherwise clean water, for example, and no more than 15 min of contact). Important, too, is the fact that disinfecting chemicals such as chlorine are not toxic to man in the concentrations employed.

Table 18.4 Disinfectant application considerations

	Cl_2	ClO_2	Mono-chloramine	O_3	UV
Optimum water pH	7	6–9	7–8	6	N/A
By-products present	Yes	Yes	Yes	Yes	No
Operational simplicity	Yes	No	No	No	Yes
Maintenance required	Low	Low	Low	High	High

Source: US EPA.
NA = not applicable.

18.6 THEORY OF CHEMICAL DISINFECTION

The disinfecting species of chlorine react with *enzymes* that are essential to the metabolic processes of bacterial cells. Cells die when these key substances are inactivated. Enzyme destruction also remains the primary lethal mechanism of disinfectants when a radical process, such as heat, coagulates cell contents. Because enzymes are generated in the cell cytoplasm, chemical disinfection of bacteria proceeds theoretically in two steps: (1) penetration of the cell wall and (2) reaction with the cell enzymes.

Factors governing the chemical disinfection technology of water fall essentially into the following three categories:

1. The nature of the organisms to be destroyed and their concentration, distribution, and condition in the water to be disinfected. Although non-sporeforming bacteria are less resistant to disinfection than sporeforming

bacteria, they are normally unimportant in water disinfection. Among the enteric bacteria, *Escherichia coli* appears to be somewhat more resistant than the pathogenic bacteria. This makes it a useful test organism. The cysts of *Entamoeba histolytica* are quite resistant. A number of enteric viruses are also measurably more resistant to chlorination than *E. coli.* Poliomyelitis virus type 1 and Coxsackie A2 virus are examples. The virus of infectious hepatitis appears to be an especially hardy organism. Adenovirus type 3 has been found less resistant than *E. coli.* The concentration of organisms normally becomes important only when it is so high that oxidation of constituent cell matter competes for the disinfectant. When bacteria, such as the staphylococci, and viruses form clumps of cells, protected inner cells may remain untouched and viable. To be reached by disinfectants of average strength or intensity, the organisms must be distributed uniformly through the water and be shifted in their position. Stirring will do this.

2. The nature, distribution, and concentration of the disinfecting substance and its reaction products in the water to be disinfected. Chlorine and hypochlorite compounds can form in water a number of chlorine species of quite different disinfecting efficiencies. Chloride with zero disinfecting power is formed when chlorine reacts with reducing agents; chloramine is formed in reactions with nitrogen; and hypochlorite, free chlorine formed above pH 7, is only one-eightieth as efficient as hypochlorous acid. To be of average strength or intensity, disinfectants must be uniformly distributed through the water. This, too, may require stirring the water.
3. The nature and condition of the water to be disinfected. Suspended matter may shelter embedded organisms against chemical disinfectants as well as against destructive light rays. Organic matter uses up oxidizing chemicals. Other substances react with chemical disinfectants and change their structure. Some of the resulting compounds may be inefficient and some even innocuous. In water chlorination, pH is of controlling importance. The higher the temperature of the water to be disinfected, the more rapid is the kill; and the longer the time, the greater is the opportunity for destruction.

18.7 KINETICS OF CHEMICAL DISINFECTION

Under ideal conditions all cells of a single species of organism are discrete units equally susceptible to a single species of disinfectant; both cells and disinfectant are uniformly dispersed in the water; the disinfectant stays substantially unchanged in chemical composition and substantially constant in concentration throughout the period of contact; and the water contains no interfering substances. Under such conditions the rate of disinfection is a function of the time of contact, the concentration of the disinfectant, and the temperature of the water.

18.7.1 Time of Contact

When under ideal conditions an exposed cell contains a single active center vulnerable to a single unit of disinfectant, the time-rate of kill follows *Chick's law* of disinfection. This states that y, the number of organisms destroyed in unit time, is proportional to N, the number of organisms remaining, the initial number being N_0, or

$$\frac{\mathrm{d}y}{\mathrm{d}t} = k(N_0 - y) \qquad \textbf{(18.1)}$$

where k is the coefficient of proportionality or the rate constant with dimension t^{-1}. By integration between the limits $y = 0$ at $t = 0$ and $y = y$ at $t = t$

$$\begin{aligned} \mathrm{In}\,\frac{N_0 - y}{N_0} &= \mathrm{In}\,\frac{N}{N_0} = -\,kt \\ N/N_0 &= \mathrm{e}^{-kt} \qquad \textbf{(18.2)} \\ \text{or} \qquad N/N_0 &= 10^{-k't} \end{aligned}$$

Therefore, a plot of log N/N_0 against t traces a straight line with a slope of $-k \log \mathrm{e} = -k'$ and an intercept of 1 (or 100%) at $t = 0$. When $kt = 1$ or $k't = 0.4343$, the surviving fraction is 0.368.

Understandably, departures from Chick's law are not uncommon, even when test conditions are nearly ideal. Rate of kill, rather than being constant, may increase or decrease with time. Increase in rate of kill can be explained in at least two ways: (1) as a combination of slow diffusion of chemical disinfectants through the cell wall and a rate of kill accelerating with the accumulation of disinfectant within the cell and (2) as the consequence of a time lag before the disinfectant can reach a lethal number of vital centers in the organism. Decrease in rate of kill is generally explained as a variation in cell resistance within the culture. However, declining concentrations of disinfectant, poor distribution of organisms and disinfectant, and other interfering factors may also account for it. The experience may be generalized by adding to k or to t coefficients that will linearize functional plots of N/N_0. A simple and often successful assumption is that N/N_0 varies logarithmically with time, that is, as t^m, where $m > 1$ when rates of kill rise in time and $m < 1$ when rates of kill fall in time (Fig. 18.1).

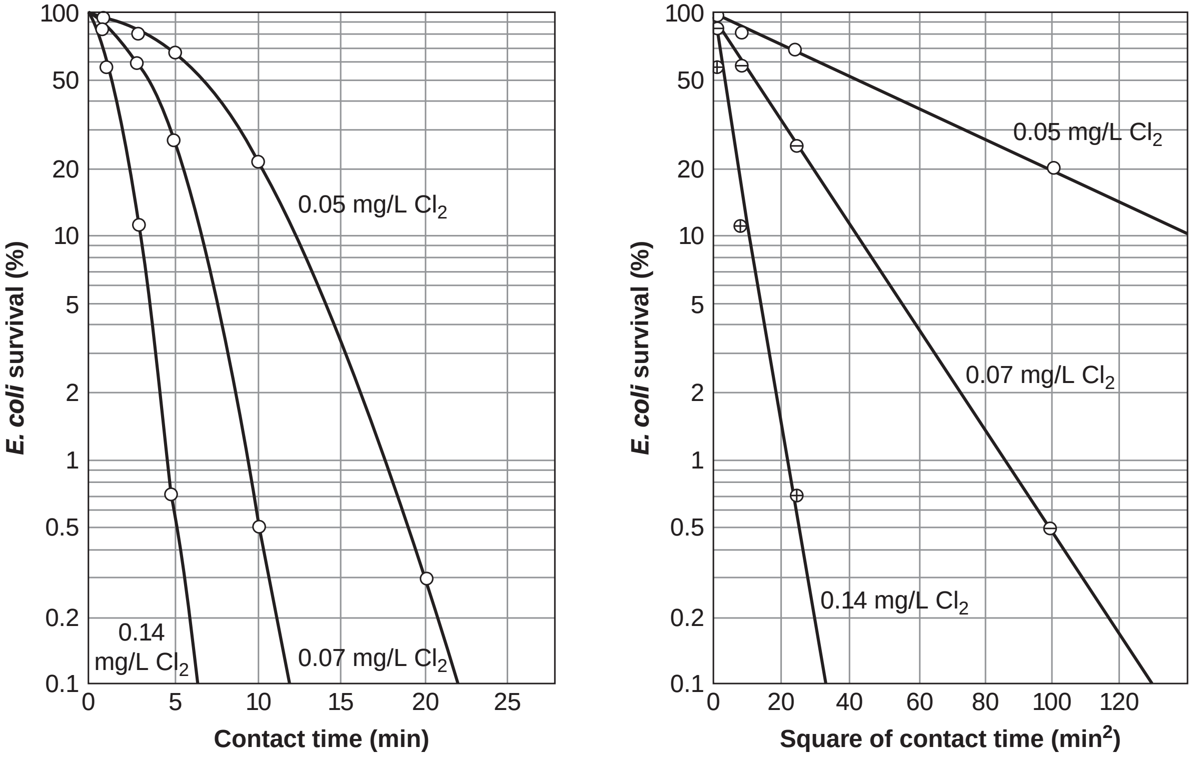


Figure 18.1 Length of survival of *E. coli* in pure water at pH 8.5 and 2–5°C (After Fair et al., 1971).

EXAMPLE 18.1 CALCULATION OF CONTACT BASIN VOLUME

A disinfection contact basin is to be designed to provide sufficient contact time for 99% kill of *E. coli* for a water/wastewater flow of 1 m^3/s. A disinfection test shows that 99.9% kill of *E. coli* can be accomplished in 30 min at a chlorine dosage of 10 mg/L. Determine the required volume of the contact basin.

Solution:

$$N/N_0 = 10^{-k't} \tag{18.2}$$

$$\log N/N_0 = -k't.$$

$$\text{For 99.9 kill, } N = N_0/1{,}000 \quad \log 1/1{,}000 = -k'(30).$$

$$\text{For 99\% kill, } N = N_0/100 \quad \log 1/100 = -k't.$$

$$\text{Therefore, } \quad \frac{\log 10^{-2}}{\log 10^{-3}} = \frac{t}{30}.$$

$$\frac{-2}{-3} = \frac{t}{30}.$$

$$\text{Hence, } \quad t = 20\,\text{min}.$$

$$V = Qt$$

$$V = 1 \times 60 \times 20 = \mathbf{1{,}200\,m^3}.$$

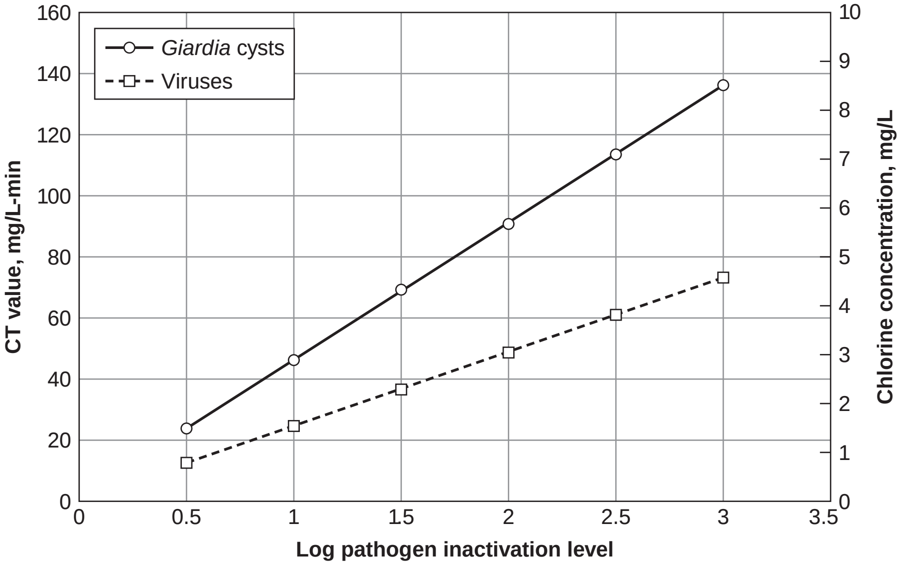


Figure 18.2 Free chlorine *Giardia* and virus *CT* requirements (*Source*: U.S. EPA, 1999).

18.7.2 Concentration of Disinfectant

For changing concentrations of disinfectant, the observed disinfecting efficiency is generally approximated by the relationship

$$C^n t_p = \text{constant} \tag{18.3}$$

Here C is the concentration of the disinfectant, t_p is the time required to effect a constant percentage kill of the organisms, and n is a coefficient of dilution or a measure of the order of the reaction. Examples of time–concentration relationships for chlorine as HOCl and 99% kill of *E. coli* and enteric viruses at 0–6°C are $c^{0.86}t_p = 6.3$ for Coxsackie virus A2; $c^{0.86}t_p = 1.2$ for poliomyelitis virus 1; and $c^{0.86}t_p = 0.098$ for adenovirus 3—all in comparison with $c^{0.86}\ t_p = 0.24$ for *E. coli*. When $n > 1$, the efficiency of the disinfectant decreases rapidly as it is diluted; when $n < 1$, time of contact is more important than dosage. When $n = 1$, concentration and time are of equal weight and a first-order reaction may be in progress.

Equation (18.3) is empirical. For a straight-line plot on double logarithmic paper of concentration versus contact time for fixed disinfection efficiency, the slope of the line is (–1/n) (Fig. 18.2).

18.7.3 Temperature of Disinfection

If the rate of disinfection is determined by the rate of diffusion of the disinfectant through the cell wall or by the rate of the reaction with an enzyme, temperature effects usually conform to the van't Hoff–Arrhenius relationship. A convenient form is

$$\log\frac{t_1}{t_2} = \frac{E(T_2 - T_1)}{2.303RT_1T_2} = \frac{E(T_1 - T_2)}{4.56T_1T_2} \tag{18.4}$$

Here T_2 and T_1 are two absolute temperatures (normally Kelvin) for which the rates are to be compared; t_1 and t_2 are the times required for equal percentages of kill at fixed concentrations of disinfectant; E is the activation energy (normally in calories) and a constant characteristic of the reaction; and R is the gas constant (e.g., 1.99 cal/°C,). For T_2–T_1 = 10, the useful ratio t_1/t_2, called Q_{10}, is related to E about as follows at normal water temperatures:

$$\log Q_{10} = \log(t_1/t_2) = E/39{,}000 \tag{18.5}$$

The work of Butterfield and his associates identifies the temperature dependence of disinfecting concentrations of aqueous chlorine and chloramines in destroying *E. coli* shown in Table 18.5.

Table 18.5 Temperature dependence of disinfecting concentrations of aqueous chlorine and chloramines in the destruction of *E. coli* in clear water

Type of chlorine	pH	E (cal[a])	Q_{10}
Aqueous chlorine	7.0	8,200[b]	1.65[b]
	8.5	6,400	1.42
	9.8	12,000	2.13
	10.7	15,000	2.50
Chloramines	7.0	12,000	2.08
	8.5	14,000	2.28
	9.5	20,000	3.35

Source: Fair et al. (1971).

[a] The higher the value of E, the slower the reaction.

[b] The rate of reaction at pH 7.0 is relatively so fast that these values are probably unreliable. The magnitude of E throw some light on the nature of the disinfecting process for the chlorine species released in water.

EXAMPLE 18.2 TEMPERATURE EFFECT ON DISINFECTION

A water disinfection contact tank has a 20-min contact time for disinfection in the summer season when the water temperature is 20°C and its pH is 8.5. What will be the minimum required detention time in winter when the water temperature is at 10°C. Assume that the water pH stays the same.

Solution:

$$\log\frac{t_1}{t_2} = \frac{E(T_2 - T_1)}{2.303RT_1T_2} = \frac{E(T_1 - T_2)}{4.56T_1T_2} \tag{18.4}$$

From Table 18.5, at pH = 8.5 the value of $E = 6{,}400$ cal.

$$\log\frac{20}{t_2} = \frac{6{,}400(20 - 10)}{4.56(20 + 273)(10 + 273)} = \frac{64{,}000}{378{,}110} = 0.17$$

Therefore,

$$20/t_2 = 10^{0.17} = 1.48$$

$$t_2 = 20/1.48 = 13.5$$

Say **14 min** is the required minimum detention time.

18.7.4 *Ct* Values for Disinfection Process Control

"*Ct* values" indicate the effectiveness of disinfectants in achieving primary disinfection. They describe the attainable degree of disinfection as the product of the disinfectant residual concentration (in mg/L) and the contact time (in min). For chlorine, chlorine dioxide, or monochloramine, the contact time can be the time required for the water to move from the point at which the disinfectant is applied to the point it reaches the first customer (at peak flow). This is the total time the water is exposed to the chlorinous residual before being used. Ozone, however, has a short half-life in water; therefore, the contact time is considered the time water is exposed to a continuous ozone residual during the water treatment process only. The *Final Surface Water Treatment Rule* (SWTR) states: Systems may measure "*C*" (in mg/L) at different points along the treatment train, and may use this value, with the corresponding "*t*" (in min), to calculate the total percent inactivation. In determining the total percent inactivation, the system may calculate the *Ct* at each point where "*C*" was measured and compare this with the $Ct_{99.9}$ value (the *Ct* value necessary to achieve 99.9% inactivation of *Giardia* cysts) in the rule for specified conditions (pH, temperature, and residual disinfectant concentration). Each calculated *Ct* value (Ct_{calc}) must be divided by the $Ct_{99.9}$ value found in the SWTR tables to determine the inactivation ratio. If the sum of the inactivation ratios or

$$\sum \frac{Ct_{\text{calc}}}{Ct_{99.9}} \tag{18.6}$$

at each point prior to the first customer where *Ct* was calculated is equal to or greater than 1.0, that is, there was a total of at least 99.9% inactivation of *Giardia lamblia*, the system is in compliance with the performance requirement of the SWTR.

Table 18.6 *Ct* values for achieving 90% inactivation of *Giardia lamblia* (1-log inactivation)

		Temperature (°C)					
Disinfectant	pH	≤1	5	10	15	20	25
Free chlorine (2 mg/L)	6	55	39	29	19	15	10
	7	79	55	41	28	21	14
	8	115	81	61	41	30	20
	9	167	118	88	59	44	29
Ozone	6–9	0.97	0.63	0.48	0.32	0.24	0.16
Chlorine dioxide	6–9	21	8.7	7.7	6.3	5	3.7
Chloramines (preformed)	6–9	1,270	735	615	500	370	250

Source: US EPA.

Table 18.7 *Ct* values for achieving 99.9% inactivation of *Giardia lamblia* (3-log inactivation)

		Temperature (°C)					
Disinfectant	pH	≤1	5	10	15	20	25
Free	6	165	116	87	58	44	29
chlorine	7	236	165	124	83	62	41
(2 mg/L)	8	346	243	182	122	91	61
	9	500	353	265	177	132	88
Ozone	6–9	2.9	1.9	1.4	0.95	0.72	0.48
Chlorine dioxide	6–9	63	26	23	19	15	11
Chloramines (preformed)	6–9	3,800	2,200	1,850	1,500	1,100	750

Source: US EPA.

The final Guidance Manual for the SWTR recommends that systems determine contact time based on the time it takes water with 10% of an approximate tracer concentration (t_{10}) to appear at the sampling site at peak hourly flow. For groundwater not under direct influence of surface water, *Ct* is determined in the same manner using enteric viruses or an acceptable viral surrogate as the determinant microorganism, since *Giardia* cysts will not be present. Table 18.6 presents the *Ct* values required to attain 1-log reductions of *Giardia* cysts, for four disinfectants. As shown, lower temperatures require higher *Ct* values; with chlorine, an increase in pH also increases necessary *Ct* values. If more than one disinfectant is used, the percent inactivation achieved by each is additive and can be included in calculating the total *Ct* value.

When direct filtration is included in the water treatment process, disinfection credit can be taken by the filtration step for a 2-log inactivation of *Giardia* cysts and a 1-log inactivation of viruses. This means that the primary disinfectant must provide an additional 1-log inactivation of *Giardia* cysts and 3-log inactivation of viruses. In the specific instance of a conventional treatment process that includes coagulation, flocculation, sedimentation, and filtration, an inactivation credit of 2.5 logs for *Giardia* cysts and 2 logs for viruses may be taken. This means that the primary disinfectant must provide an additional 0.5-log inactivation of *Giardia* cysts but a 2-log inactivation of viruses.

If a water supply system does not use filtration, the 99.9% inactivation of *Giardia* and 99.99% inactivation of enteric viruses must be achieved by the primary disinfecting agents alone. Table 18.7 presents *Ct* values for the four disinfectants for achieving 99.9% reductions of *Giardia* cysts. Table 18.8 presents the *Ct* values for virus inactivation. For groundwater disinfection regulations, these values will apply to systems treating groundwater determined by the state not to be under direct influence of surface water.

Table 18.8 *Ct* values for achieving inactivation of viruses at pH 6–9

		Temperature (°C)					
Disinfectant	Log inactivation	0.5	5	10	15	20	25
Free	2	6	4	3	2	1	1
chlorine	3	9	6	4	3	2	1
	4	12	8	6	4	3	2
Ozone	2	0.9	0.6	0.5	0.3	0.25	0.15
	3	1.4	0.9	0.8	0.5	0.4	0.25
	4	1.8	1.2	1.0	0.6	0.5	0.3
Chlorine	2	8.4	5.6	4.2	2.8	2.1	–
dioxide	3	25.6	17.1	12.8	8.6	6.4	–
	4	50.1	33.4	25.1	16.7	12.5	–
Chloramines	2	1,243	857	643	428	321	214
	3	2,063	1,423	1,067	712	534	356
	4	2,883	1,988	1,491	994	746	497

Source: US EPA.

EXAMPLE 18.3 *Ct* VALUES CALCULATION

A 10 MGD (37.85 ML/d) water treatment plant using direct filtration applies free chlorine as a disinfectant. The disinfectant has a contact time of 22 min under peak flow conditions. The pH of the water is 7.0 and the temperature is 10°C. The free chlorine residual is 2.0 mg/L. Determine the *Ct* value provided and the *Ct* required both in units of (mg/L)-min or mg-min/L. Assume the local government requires a 3-log inactivation (99.9%) of Giardia, and gives direct filtration a credit of 2-log (99%) removal of Giardia cysts.

Solution:

a. Determine the *Ct* required:

Direct filtration is capable of achieving 2-log (99%) removal of *Giardia* cysts. Therefore 1-log (90% or *Ct* 90) *Giardia* inactivation must be achieved by disinfection. Using Table 18.6 with a pH of 7.0, a temperature of 10°C, 1.0-log inactivation, and a free chlorine residual of 2.0 mg/L, find the required *Ct* value of 41 (mg/L)-min.

b. Calculate the *Ct* provided:

$$Ct \text{ Provided} = (C)(t)$$

where

Ct Provided = Ct value provided by the treatment process, (mg/L)-min

C = disinfectant concentration, mg/L

t = contact time, min

Given:

C = disinfectant concentration = 2 mg/L of free chlorine.

t = contact time = 22 min at peak flow of 10 MGD (37.85 ML/d).

Results:

Ct Provided = $(C)(t)$ = (2 mg/L) (22 min) = 44 (mg/L)(min).

Since 44 (mg/L)-min is greater than the required 41 (mg/L)-min, the free chlorine residual is adequate for the peak flow conditions.

18.8 DISINFECTION BY OZONE

Purposeful production of ozone, from the Greek word *ozein*, to smell by the corona discharge of high-voltage electricity into dry air was introduced by the German electrical engineer Werner von Siemens. Both in air and in water, ozone breaks down rapidly in the presence of oxidizable matter. It is corrosive and poisonous in strong concentrations in the atmosphere, and its photochemical genesis in conjunction with gasoline vapors from automobile exhausts is responsible for oxidant smogs that are eye, throat, and lung irritants. Contrary to popular belief and common usage of the word, ozone is a toxic, not a tonic, substance. The danger limit in treatment plant operation is commonly set at 0.2 mg of O_3/m^3 of air.

If ozone is to be employed effectively and efficiently as a deodorant, decolorant, and disinfectant of drinking water, its physical and chemical properties in water solution and their influence on pathogenic microorganisms need to be known over the full range of possible exposures. We can get a feeling for the usefulness of ozone and constraints on its use from information such as the following.

The weight of ozone is about 2,154 g/m^3. At normal temperatures ozone residuals disappear rapidly from water. This is shown by the following relationship

$$p_t = 100\,e^{-0.275t} \qquad \textbf{(18.7)}$$

where p_t is the percentage of residual ozone at time t.

Only in the absence of organic matter does ozone follow the laws of ideal, that is, nonreacting, gases in water. The distribution coefficient of ozone between air and water, that is, the ratio of the equilibrium concentration of ozone in the liquid phase to that in the gas phase at like temperature and pressure, is then about 0.6 at 0°C and 0.2 at 20°C. Increasing either the total pressure of the system or the partial pressure of ozone in the air raises the concentration of ozone in water in direct proportion to these pressures. In the presence of oxidizable substances, their nature and concentration in water rather than the distribution coefficient govern the amount of entering ozone.

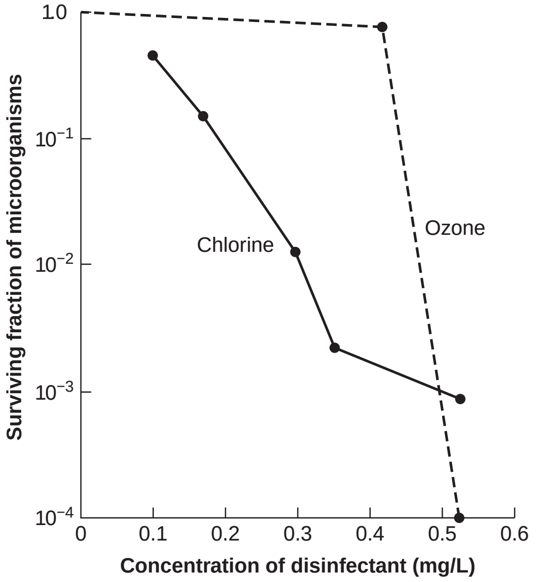


Figure 18.3 Comparative disinfection by ozone and chlorine.

As disinfectant ozone is said to possess an *all-or-none property*, implying that it produces essentially no disinfection below a critical concentration but substantially complete disinfection above that concentration. This property is illustrated in Fig. 18.3, in which the decontamination factor of ozone for *E. coli*, that is, the reciprocal of the surviving fraction, is seen to be substantially zero at a concentration of 0.42 mg/L and 10^4 at 0.53 mg/L. For comparison, similar tests of the behavior of chlorine are seen to trace a more or less straight-line increase of the decontamination factor from 10 at a concentration of 0.2 mg/L of chlorine to 10^3 at 0.48 mg/L. It is also reported that coliforms and other bacteria are normally wiped out by the time an ozone residual can be detected. A first estimate of ozone requirements for the disinfection of clean water may be placed at 1–2 mg/L.

On the air side, modern ozonation plants (Fig. 18.4) comprise air cleaners, blowers, and refrigerative as well as adsorptive air driers. On the electrical side, a transformer steps up the voltage to produce the corona in the air supply. On the water side, a contact chamber or ozone tower effects the transfer of ozone from the gas phase to the water phase. This is the key hydraulic and phase-transfer operation and, therefore, also the principal manageable component once the ozonizing unit has been selected. Because the absorption of ozone from the air into the water to be disinfected is a matter of contact opportunity, *contact chamber* design aims at a maximization of (1) effective interface, (2) driving force or concentration differential, and (3) time of exposure with due consideration of advantages to be gained by countercurrent operation.

To be decided is whether droplets of water shall fall through a column of rising ozonized air or bubbles of air shall be injected into a column of water flowing in the same or opposite direction; moreover, whether the air shall be injected into the entrant water or into the ozonizing tower itself; and, finally, whether mixing other than that inherent in the air–water streams shall be provided by mixing devices. Detention times are of the order of 10 min.

As a rule, capital and running costs of ozonation equipment cannot compete with those of comparable chlorination equipment for the treatment of a given water unless ozone is called upon and able to remove objectionable odors and tastes and reduce the color of the water more effectively than chlorine in combination with activated carbon and coagulants. Comparisons of operating expenses derive from the cost of power versus the cost of chlorine and auxiliary chemicals in specific circumstances. The energy used in converting 0.5–1% of atmospheric oxygen into ozone is 0.025–0.030 kWh/g of O_3). If water is to be deodorized and decolorized as well as disinfected, the required dosage may be nearer to 2–4 mg/L.

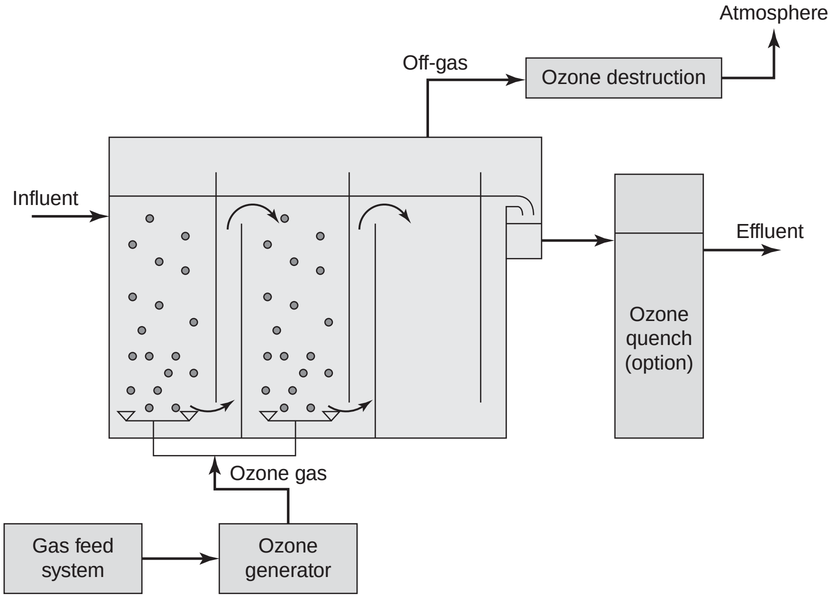


Figure 18.4 Simplified ozonation system schematic (*Source*: U.S. EPA, 1999).

EXAMPLE 18.4 DISSIPATION OF OZONE

If 5 mg/L of ozone is added to clean unpolluted water, what will be the concentration remaining after 10 min?

Solution:

$$p_t = 100\,\mathrm{e}^{-0.275t}$$

$$p_t = 100\,\mathrm{e}^{-0.275\times 10} \qquad \textbf{(18.7)}$$

$$p_t = 100\,\mathrm{e}^{-2.75}$$

$$p_t = 100 \times 0.064$$

Therefore, percentage remaining = 6.39%.
Ozone concentration remaining after 10 min = **0.32 mg/L**
= (5 mg/L) × 0.0639.

18.9 DISINFECTION BY CHLORINE

Chlorine was first used for day-in, day-out disinfection of a municipal water supply in America in 1908, when George A. Johnson and John L. Leal (Fair et al., 1971) added chloride of lime to the water supply of Jersey City, NJ.

The following substances are released when chlorine or its hypochlorite compounds are added to water:

1. Hypochlorous acid (HOCl), hypochlorite ion (OCl^-), and elemental chlorine (Cl_2): Distribution of the three species depends on pH. Elemental chlorine, from chlorine gas, lasts but a fleeting moment within the normal pH zone. The two prevailing species HOCl and OCl^- are referred to in practice as free available chlorine.
2. Monochloramine (NH_2Cl), dichloramine ($NHCl_2$), and nitrogen trichloride (NCl_3): Ammonia, or organic nitrogen, is essential to the production of these compounds. The distribution of the three species is again a function of pH. Nitrogen trichloride is not formed in significant amounts within the normal pH zone except when the breakpoint is approached. The two prevailing species, NH_2Cl and $NHCl_2$, are referred to in practice as combined available chlorine.
3. Complex organic chloramines, especially in heavily contaminated waters.
4. Chloride, formed on reaction with reducing compounds, such as ferrous iron, sulfides, and organic reducing compounds.

Because the disinfecting power of the different species of chlorine varies widely, the chemistry of chlorination must be fully understood. Otherwise chlorine and its compounds will not be employed intelligently and efficiently in the disinfection of water and wastewaters.

Reducing substances react with chlorine to produce the chlorine demand. Depending on the nature of the substances present in water, the chlorine atom, by gaining electrons, is changed into chloride ion or organic chloride. Reducing substances may include inorganic Fe^{2+}, Mn^{2+}, NO_2^-, and H_2S along with the greater part of the organic material (living and dead). The inorganic substances generally react rapidly and stoichiometrically; the response of the organic material is generally slow, and its extent depends on how much available chlorine is present in excess of requirements. Because the organic material in drinking water supplies is closely related to their natural color or stain, their probable organic chlorine demand may be estimated from the depth of color. In an analogous fashion, the organic chlorine demand of wastewaters bears some relation to their BOD or, more closely, to their COD (chemical oxygen demand).

These reactions are complicating factors in water chlorination. Enough chlorine must be added to take care of them so that sufficient free chlorine remains for the disinfecting reactions. To assure this, chlorine residuals remaining after a specific time of contact, rather than initial chlorine doses, are made standards of accomplishment or comparison. Ten minutes are specified in most testing. Because the chlorine demand is a function of temperature, concentration, and time, its determination must take all three factors into account.

18.10 FREE AVAILABLE CHLORINE AND FREE CHLORINATION

Free chlorine residual or *free available chlorine* is the amount of chlorine remaining in a water purification system after the chlorine demand is satisfied. It is the amount of free chlorine in the water that is available to oxidize, sanitize, or disinfect the water. The disinfection process is free chlorination in which free residual chlorine or free available chlorine is used for disinfection process control by water purification system

Table 18.9 Values of the ionization constant of hypochlorous acid at different temperatures

Temperature (°C)	0	5	10	15	20	25
$K_i \times 10^8$ (moles/L)	1.5	1.7	2.0	2.2	2.5	2.7

Source: Fair et al. (1971).

operators. The following equilibrium equations are obtained when elemental chlorine is dissolved in pure water:

Hydrolysis:

$$\mathrm{Cl_2 + H_2O \leftrightarrow HOCl + H^+ + Cl^-} \tag{18.8}$$

$$\mathrm{(HOCl)(H^+)(Cl^-)/(Cl_2)} = K_h = 4.5 \times 10^{-4} (\mathrm{mole/L})^2 \text{ at } 25°\mathrm{C} \tag{18.9}$$

Ionization:

$$\mathrm{HOCl \leftrightarrow H^+ + OCl^-} \tag{18.10}$$

$$\mathrm{(H^+)(OCl^-)/(HOCl)} = K_i \text{ or } \mathrm{(OCl^-)/(HOCl)} = K_i/\mathrm{(H^+)} \tag{18.11}$$

Solutions of hypochlorites, such as chloride of lime and calcium hypochlorite, establish the same ionization equilibrium in water. Taking calcium hypochlorite as the example, the reactions leading up to equilibrium are

$$\mathrm{Ca(OCl)_2 \leftrightarrow Ca^{2+} + 2OCl^-} \tag{18.12}$$

and

$$\mathrm{H^+ + OCl^- \leftrightarrow HOCl}$$

as in Eq. (18.10). The hydrolysis constant K_h, Eq. (18.9), is of such magnitude that no measurable concentration of Cl_2 remains in solution when the pH of the chlorinated water is more than about 3.0 and the total chloride concentration is less than about 1,000 mg/L.

At ordinary water temperatures the hydrolysis of chlorine is essentially complete within a few seconds, and the ionization of hypochlorous acid produced is in essence an instantaneous, reversible reaction. The ionization constant K_i varies in magnitude with temperature, as shown in Table 18.9. The percentage distribution of HOCl and OCl^- at various pH values is shown in Fig. 18.5. It is calculated from Eq. (18.11) and Table 18.9 as

$$\frac{\mathrm{(HOCl)}}{\mathrm{(HOCl) + (OCl^-)}} = \frac{1}{1 + \mathrm{(OCl^-)/(HOCl)}} = \frac{1}{1 + \frac{K_i}{(H^+)}} \tag{18.13}$$

At 20°C and pH 8, for instance, the percentage distribution of HOCl is (see Fig. 18.5)

$$100[1 + 2.5 \times 10^{-8}/10^{-8}]^{-1} = 100/3.5 = 29\%$$

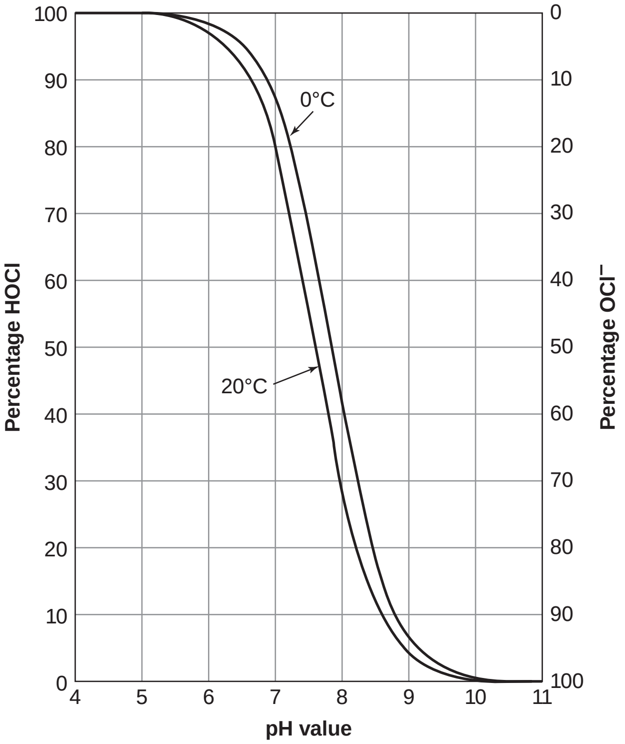


Figure 18.5 Distribution of HOCl and OCl^- in water at different pH and *T* values (*Source*: UNIDO).

The chlorine species that constitute free available chlorine are identified in this way.

Observed *colicidal* efficiency (concentration of aqueous, or free available, chlorine required to kill 99% of *E. coli* in 30 min at 2–5°C) is illustrated in Fig. 18.6. That the two curves in Fig. 18.6 are mirror images suggests a higher

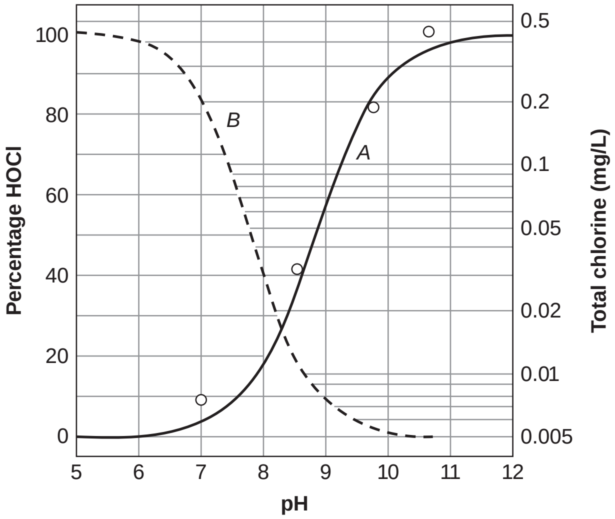


Figure 18.6 Observed concentration of free available chlorine required for 90% kill of *E. coli* in 30 min at 2–5°C (curve A and right-hand scale), and percentage of HOCl in the total chlorine (curve B and left-hand scale) (After Fair et al., 1971).

killing efficiency for HOCl than for OCl^- in the approximate ratio of 80:1 for the conditions of test. If the efficiencies of the two species of chlorine are additive, the total amount of chlorine, R_T, required to produce a given percentage of kill in a specified time at various pH values becomes

$$R_T = (HOCl) + (OCl^-) = (HOCl)[1 + (OCl^-)/(HOCl)]$$

By Eq. (18.11), therefore, $R_T = HOCl[1 + K_i/(H^+)]$. If c is the killing concentration of HOCl,

$$c = [HOCl] + r[OCl^-] = \frac{R_T}{1 + \frac{K_i}{(H^+)}} + r\frac{K_i}{(H^+)}\frac{R_T}{1 + \frac{K_i}{(H^+)}}$$

and

$$R_T = c\frac{1 + K_i/(H^+)}{1 + rK_i/(H^+)} \tag{18.14}$$

Here brackets stand for the required concentration of chlorine and r is the proportionate efficiency of OCl^- ions relative to HOCl.

18.11 COMBINED AVAILABLE CHLORINE AND CHLORAMINATION

The treatment of drinking water with a chloramine disinfectant is termed *chloramination*. Both chlorine and small amounts of ammonia are added to the water one at a time which react together to form chloramine (also called *combined chlorine*), a long lasting disinfectant. As such, chloramine disinfection is sometimes used in large distribution systems.

The most important reaction of chlorine with compounds of nitrogen in water is the reaction of hypochlorous acid with ammonia. This is a stepwise process, for which the successive reactions are the following:

$$NH_3 + HOCl \rightarrow NH_2Cl + H_2O \tag{18.15}$$

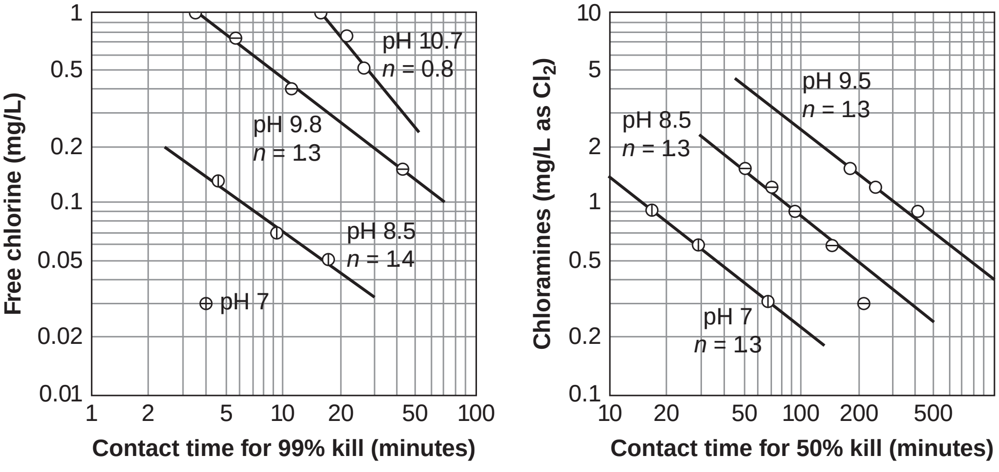
$$NH_2Cl + HOCl \rightarrow NHCl_2 + H_2O \tag{18.16}$$

$$NHCl_2 + HOCl \rightarrow NCl_3 + H_2O \tag{18.17}$$

When the pH is above 6 and the molar ratio of chlorine (or hypochlorite) to ammonia (or ammonium ions) is not more than 1.0, the formation of monochloramine predominates. The rate of the first step depends greatly on the pH of the solution. The rate is maximal at pH 8.3 and decreases rapidly in both directions. Its measured magnitude per minute is 8.9×10^{-3} at pH 4.6, 5.8×10^{-1} at pH 6.5, and 7.4×10^{-3} at pH 12.1. At pH 8.3, 25°C, 0.8 mg /L of chlorine, and 0.32 mg/L of ammonia nitrogen, the reaction is 99% complete in about 1 min; at pH 5 the corresponding time is 210 min; at pH 11 it is 50 min. The reaction rate varies greatly also with temperature. Depending on pH, Q_{10} lies between 2.0 and 2.5.

Both Palin (1949) and Morris (1967) have shown that distribution is actually governed by the relative rates of formation of monochloramine and dichloramine. These change with the relative concentrations of chlorine and ammonia as well as with pH and temperature.

A comparison of Figs. 18.7a and 18.7b shows that combined available chlorine is a much less efficient colicidal agent than free available chlorine. The dilution coefficient $n = 1.3$ suggests that disinfectant concentration is somewhat



(*a*) (*b*)

Figure 18.7 Time–concentration relationship in disinfection: (a) concentration of free available chlorine required for 99% kill of *E. coli* at 2–5°C; (b) concentration of combined available chlorine required for 50% kill of *E. coli* at 2–5°C) (After Fair et al., 1971).

EXAMPLE 18.5 CHARACTERISTICS OF HOCl AND OCl$^-$ ION

From the data presented in Table 18.5 and Figs. 18.6 and 18.7a, draw a comparison between the disinfecting characteristics of (1) HOCl and (2) OCl$^-$ ion based on the results for pH values of 7.0 and 10.7.

Solution:

1. At pH 7.0 about 80% of the disinfecting chlorine is present as HOCl; the coefficient of dilution, n, is 1.5, indicating that disinfectant concentration is more important than contact time; the energy of activation, $E = 8{,}200$ cal, falls within the range of activation energies for diffusion processes; and the residual chlorine concentration required for a decontamination factor of 100 or 99% kill of *E. coli* in 10 min at 2–5°C is 0.017 mg/L, or $C^n t = 0.017^{1.5} \times 10 = 2.2 \times 10^{-2}$.
2. At pH 10.7 almost 100% of the disinfecting chlorine is present as OCl$^-$ ion; the coefficient of dilution, n, is 0.8, indicating that contact time is more important than disinfectant concentration; the energy of activation, $E = 15{,}000$ cal, falls within the range of activation energies for chemical reactions; and the residual chlorine concentration for 99% kill of *Esch. coli* in 10 min at 2–5°C is 1.7 mg/L, or $C^n t = 1.7^{0.8} \times 10 = 15.3$.

Therefore it appears that the rate-determining processes are different for HOCl and OCl$^-$ ions.

more important than contact time, and the magnitudes of $E = 12{,}000$–$20{,}000$ recorded in Table 18.5 lie within the range of chemical reactions.

18.12 BREAKPOINT REACTIONS OF AMMONIA

There is oxidation of ammonia and reduction of chlorine when the molar ratio of chlorine to ammonia is greater than 1.0. A substantially complete oxidation–reduction process occurs in the neighborhood of a 2:1 ratio and leads, in the course of time, to the disappearance from solution of all the ammonia and oxidizing chlorine. (This is the reason that the reaction in Eq. (18.17) and trichloramine is unimportant in disinfection.) This is called the *breakpoint* phenomenon. It is illustrated in Fig. 18.8. Between points 1 and 2, molar ratios of chlorine to ammonia are less than 1.0, and the residual oxidizing chlorine is essentially all monochloramine. Between point 2 and the breakpoint, oxidation of ammonia and reduction of chlorine increase until complete oxidation–reduction occurs at the breakpoint. In this region, again, the residual oxidizing chlorine is essentially all monochloramine. Beyond the breakpoint, unreacted hypochlorite remains in solution, and some nitrogen trichloride is formed, if the pH is low.

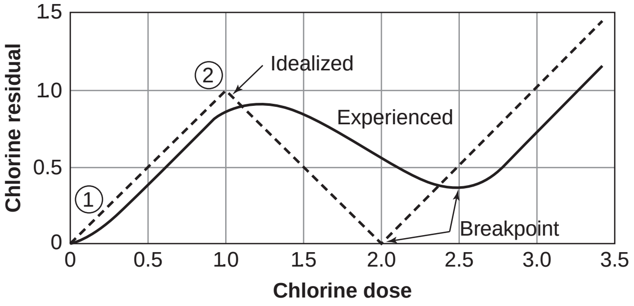


Figure 18.8 Schematic diagram of breakpoint chlorination (After Fair et al., 1971).

In contrast to the chlorination of ammonia, which results in chloramines with substantial disinfection efficiency for bacteria, the chlorination of organic nitrogen such as amino acids present as decomposition products of proteins and urea results in organic chloramines that often have much lower disinfecting efficiencies than do ammonia chloramines. Unfortunately, monochloroglycine, the simplest amino acid chloramine, although much less powerful as a disinfectant, is analytically indistinguishable from ammonia monochloramine.

The rate of the breakpoint reaction is strongly affected by pH. The maximum lies between pH 6.5 and 8.5. In practice, time requirements are determined by test; times of 30 min or more are common.

Important advantages of chlorinating to and beyond the breakpoint or otherwise obtaining free available chlorine residuals are that (1) most odors and tastes normal to water are destroyed and (2) rigorous disinfection is assured. However, in the presence of undecomposed urea, nitrogen trichloride is very likely to be formed and give rise to bad odors and tastes.

18.13 DECHLORINATION

When large amounts of chlorine have been added to water, for example, to ensure disinfection before the water is to be consumed or to destroy odors and tastes, unwanted residuals can be removed by *dechlorination*. Intensive use of chlorine in this manner without the breakpoint reaction is called *superchlorination and dechlorination.* Some methods of dechlorination are the addition of reducing chemicals, passage through beds of granular activated carbon (GAC), and aeration. The reducing agents include sulfur dioxide (SO_2), sodium bisulfite ($NaHSO_3$), and sodium sulfite (Na_2SO_3). The bisulfite is ordinarily used in practice. It is cheaper and more stable than the sulfite. Samples of water collected for bacteriological analysis are usually dechlorinated by including sodium thiosulfate ($Na_2S_2O_3$) in the sampling bottles either in solution or as crystals. GAC sorbs chlorine and is oxidized by it to carbon dioxide. In practice, contact with powdered activated carbon is generally too short to do this. Chlorine, hypochlorous acid, chlorine dioxide, and nitrogen trichloride are sufficiently volatile to be removed by aeration. Some species of chlorine are not.

The stoichiometric reactions of dechlorinating agents follow:

$$SO_2 + Cl_2 + 2H_2O \rightarrow H_2SO_4 + 2HCl \quad \textbf{(18.18)}$$

$$NaHSO_3 + Cl_2 + H_2O \rightarrow NaHSO_4 + 2HCl \quad \textbf{(18.19)}$$

$$2Na_2S_2O_3 + Cl_2 \rightarrow Na_2S_4O_6 + 2NaCl \quad \textbf{(18.20)}$$

$$C + 2Cl_2 + 2H_2O \rightarrow CO_2 + 4HCl \quad \textbf{(18.21)}$$

In addition to the above chemical reducing agents, dechlorination can also be accomplished effectively by the UV process.

18.14 DISINFECTION BY-PRODUCTS

18.14.1 Formation of Disinfection By-products

DBPs are formed by two basic mechanisms: (1) reduction, oxidation, or disproportionation of the disinfecting agent and (2) reaction of oxidation by the disinfectant with materials already in the water. Reduction, oxidation, or disproportionation can occur when the disinfecting agent is added to water. Three examples of this reaction are the formation of chlorite and chlorate ions associated with chlorine dioxide, the formation of dissolved oxygen associated with ozone, and the formation of chloride ions associated with chlorine.

Oxidation of humic acids (in the water from organic materials) produces aldehydes, ketones, alcohols, and carboxylic acids upon the addition of ozone, chlorine, chlorine dioxide, or potassium permanganate. Halogenation of organic materials can occur in the presence of free chlorine to produce trihalomethanes and other halogenated organics. Chlorine can also form organic chloramines by reacting with nitrogen-containing organic compounds (amino acids and proteins). In addition, monochloramine can produce organic chloramines in the presence of organonitrogen compounds.

If bromide ion is present in the untreated water, it may be oxidized by ozone or chlorine (but apparently not by chlorine dioxide or chloramine) to form hypobromous acid, which in turn can brominate organic materials. Bromine-containing trihalomethane, for example, are known to form in this manner.

By-products are also produced when oxidants, like ozone or chlorine, are used for oxidation purposes other than disinfection. For instance, breakpoint chlorination is sometimes used early in the water treatment process to remove ammonia. In the presence of organic compounds considered precursors, the same by-products that are formed during chlorine disinfection are also formed in this oxidation step.

US EPA has surveyed 10 operating water utilities for the presence of 22 halogenated DBPs in chlorine-treated water. Table 18.10 presents the frequency and range of concentrations of those by-products of greatest concern. Table 18.11 summarizes the current knowledge of health effects of selected chlorination by-products.

18.14.2 Strategies for Controlling Disinfection By-products

The formation of halogenated *by-products* is affected by a number of factors, including the concentration and types of organic materials present when chlorine is added, the dosage of chlorine, the temperature and pH of the water, and the reaction time. US EPA has identified three strategies for controlling formation of halogenated materials during chlorination:

1. Remove the by-products after they are formed.
2. Use alternate disinfectants that do not produce undesirable by-products.
3. Reduce the concentration of organics in the water before oxidation or chlorination to minimize the formation of by-products.

The first approach, removing the by-products after they are formed, can be difficult and costly. The second approach, using alternative disinfectants, is often the most cost-effective. The third approach, reducing the concentrations of organic precursors before adding chlorine or other oxidants, will provide the highest quality finished water.

The second approach, using other than chlorine for disinfection, is sound if the replacements do not produce undesirable by-products of their own and if they perform equally as both primary and secondary disinfectants. Cost is also a consideration. Alternative disinfectants currently being considered by water treatment specialists are chlorine dioxide, monochloramine, UV radiation, and ozone. Both ozone and UV radiation do not provide stable residuals for the distribution system and, therefore, cannot be used as substitute disinfectants by themselves. Monochloramine is a weak disinfectant. The very high *Ct* values required to inactivate 99.9% *Giardia* and 99.99% enteric viruses make monochloramine impractical for use as a primary

Table 18.10 Occurrence of chlorination by-products at 10 water utilities

Compound	Number of locations	Range of values (μg/L)
High confidence		
Chloroform	10 of 10	2.6–594
Bromodichloromethane	10 of 10	2.6–77
Chlorodibromomethane	10 of 10	0.1–31
Bromoform	6 of 10	0.1–2.7
Dichloroacetonitrile	10 of 10	0.2–9 5
Dibromoacetonitrile	3 of 7	0.4–1.2
Bromochloroacetonitrile	7 of 7	0.2–4.0
Chloropicrin	8 of 10	0.2–5.6
Low confidence		
Chloroacetic acid	6 of 10	<10
Dichloroacetic acid	10 of 10	<10–>100
Trichloroacetic acid	6 of 10	10–100
Trichloroacetaldehyde (as chloral hydrate)	10 of 10	10–100
1,1,1-Trichloropropanone	10 of 10	10–100
2-Chlorophenol	0 of 10	—
2,4-Dichlorophenol	0 of 10	—
2,4,6-Trichlorophenol	0 of 10	—
Qualitative only		
1,1-Dichloropropanone	0 of 8	—
1,1-Dichloro-2-butanone	0 of 8	—
3,3-Dichloro-2-butanone	1 of 8	—
1,1,1-Trichloro-2-butanone	0 of 8	—
Cyanogen chloride	1 of 7	—
Dichloroacetaldehyde	0 of 10	—

Source: US EPA.

Table 18.11 Health effects associated with chlorination by-products

Chemical class	Example	Toxicological effects
Trihalomethanes	Chloroform	C, H, RT
	Dichlorobromomethane	H, RT
	Dibromochloromethane	H, RT
	Bromoform	H, RT
Haloacetonitriles	Chloroacetonitrile	G, D
	Dichloroacetonitrile	M, G, D
	Trichloroacetonitrile	G, D
	Bromochloroacetonitrile	M, G, D
	Dibromoacetonitrile	G, D
Haloacid derivatives	Dichloroacetic acid	MD, C, N, OL, A
	Trichloroacetic acid	HPP
Chlorophenols	2-Chlorophenol	F, TP
	2,4-Dichlorophenol	F, TP
	2,4,6-Trichlorophenol	C
Chlorinated ketones	1,1-Dichloropropanone	M
	1,1,1-Trichloropropanone	M
	1,1,3,3-Tetrachloropropanone	M
Chlorinated furanones	MX	M, Cl
Chlorinated aldehydes	2-Chloroacetaldehyde	G

Key toxicological effects:

C = Carcinogenic
H = Hepatotoxic
RT = Renal Toxic
G = Genotoxic
D = Developmental
M = Mutagenic
MD = Metabolic Disturbance
N = Neurotoxic
OL = Ocular Lesions
A = Aspermatogenesis
HPP = Hepatic Peroxisome Proliferation
F = Fetotoxic
TP = Tumor Promoter
Cl = Clastogenic

Source: US EPA.

disinfectant. Therefore, monochloramine should only be considered as a secondary disinfectant.

The third approach for controlling DBPs is to reduce the concentration of organic materials before adding chlorine or any oxidant. This approach will minimize the formation of by-products. Another option is to use an oxidant that does not contain chlorine, such as ozone, potassium permanganate, or hydrogen peroxide, before or during rapid mix and/or filtration to partially oxidize organics. This will improve the flocculation and filtration processes that follow. However, if the water contains substantial amounts of bromide ion, brominated organics may be produced.

Because all disinfectants and oxidants produce some types of by-products, the most efficient approach to lowering organic by-product precursors is to optimize physical process treatment steps before adding oxidants.

It is important to note that extensive oxidation converts *natural organic materials* (NOMs) into simpler oxidation products, which are much more biodegradable than the initial organic materials. Consequently, a biological treatment step following oxidation can mineralize the oxidized materials, removing them from solution, thus avoiding the incorporation of these by-products into the finished water.

Examples of effective biological treatment steps are filtration, specifically through sand; dual media filters; GAC/sand filters (GAC on top of sand); and postfiltration GAC adsorbers. The biological efficiencies of these types of filters increase in the order listed. To allow biological filtration, it is critical that no residual disinfectant be present in solution. Otherwise, microbial activity present in the filter media will be eliminated or at least adversely affected.

18.15 CHEMICAL TECHNOLOGY OF DISINFECTION

For storage and shipment in steel cylinders or tanks, chlorine gas (Cl_2) can be liquefied at room temperatures, at pressures of 5–10 atm (3,800–7,600 mm Hg). One pound of the liquid produces 5 ft^3 (0.1416 m^3) of gas. Under conditions of use, gas withdrawal lowers the temperature of the stored fluid. To keep the rate of withdrawal constant, heat loss must be balanced from without. For large rates of use, chlorine may be withdrawn as a liquid and vaporized in equipment similar to a hot-water heater. Direct application of heat at temperatures above 125°F (52°C) is dangerous. Because reliquefaction of chlorine in measuring and dosing equipment produces erratic results, chlorine containers and gas lines must be kept cooler than the dispensing equipment.

The solubility of chlorine gas in water is about 7300 mg/L at 68°F and 1 atm. Below 49.2°F (9.6°C) chlorine combines with water to form chlorine hydrate (usually $Cl_2.8H_2O$), called chlorine ice. The hydrate may obstruct feeding equipment. Therefore, feed or sealing water coming into contact with the gas should be kept above 49.2°F (9.6°C).

Chlorine gas is a highly toxic irritant and must be handled with great care and under adequate safeguards. Its odor threshold in air is about 3.5 ppmv (parts per million by volume). Concentrations of 30 ppmv or more induce coughing, and exposures for 30 min to concentrations of 40–60 ppmv are dangerous. At 1,000 ppmv the gas is rapidly fatal.

For large uses liquid chlorine is cheapest. Yet the gas is so dangerous that its transport through crowded communities and its use in works within built-up areas must receive careful study and decision. Some species of chlorine other than liquid chlorine may be more satisfactory, among them the hypochlorites of calcium, $Ca(OCl)_2$, and sodium (eau de Javelle) (NaOCl); and chlorinated lime, CaClOCl. Purposeful combination of chlorine with ammonia and release of chlorine dioxide, ClO_2, from sodium chlorite, $NaClO_2$, create other useful chlorine disinfectants or deodorants. Chlorinated lime (a loose combination of chlorine with slaked lime), calcium hypochlorite, and sodium chlorite are generally marketed as solids; sodium hypochlorite as a liquid.

Of the substances employed in combination with chlorine, or as *antichlors* (chlorine-reducing substances), ammonia and sulfur dioxide are gases. They can be liquefied, stored, handled, and dispensed like chlorine. Ammonia is also available as ammonium hydroxide (aqua ammonia) and ammonium sulfate. Sodium bisulfite ($NaHSO_3$), a solid, may take the place of SO_2.

The strength of chlorine compounds, that is, their oxidizing capacity, is commonly expressed as available chlorine. This is analogous to reporting alkalinity as $CaCO_3$. Chlorine equivalent would be a more accurate designation. The percentage of available chlorine is based on (1) the moles of equivalent chlorine or number of moles of chlorine that would have an oxidizing capacity equivalent to 1 mole of the compound and (2) the proportion by weight of the pure compound present in the commercial product. Chlorinated lime, for example, contains as its essential constituent about 62.5% of calcium oxychloride, CaClOCl (mol weight 127). Because each mole of CaClOCl is equivalent to 1 mole of Cl_2 (mol weight 71), the hypothetical weight of chlorine is $62.5 \times 71/127 = 35\%$ of the total and the available chlorine is also 35%.

18.16 OPERATIONAL TECHNOLOGY OF CHLORINATION

Liquid chlorine, ammonia, and sulfur dioxide are generally added to water in controlled amounts through orifice flow meters or dosimeters called chlorinators, ammoniators, and sulfonators, respectively. For given dosages, pressure drops across the orifice are kept constant. In dosimeters operated under pressure this is done by a pressure-reducing, pressure-compensating valve that keeps the influent pressure constant regardless of pressure changes in the container from which the gas is drawn. In dosimeters operated under a vacuum, the pressure drop across the orifice is regulated by a controlled

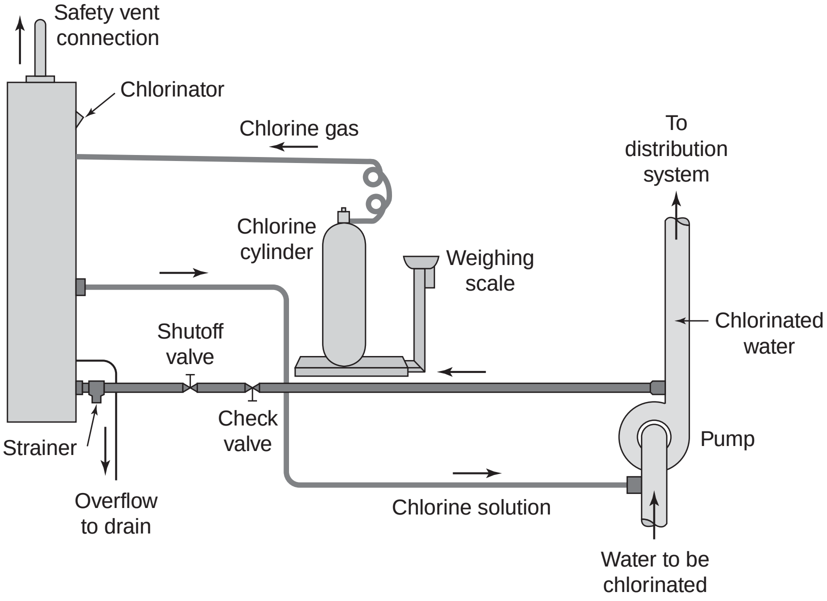


Figure 18.9 Typical chlorinator installation (After Fair et al., 1971).

vacuum on the inlet and outlet sides of the orifice. The purpose of vacuum feed is to lessen gas leakage. Some simplified pressure devices regulate the volumetric displacement of the gas (bubblers) rather than its rate of flow.

Chlorine gas may be fed directly into water through diffusers. However, because some gas may escape, chlorine gas is normally dissolved in a small flow of water that is passed through the gas-flow-regulating device to transport the dissolved gas to the point of application. A typical solution-feed chlorinator installation is shown in Fig. 18.9. Newly constructed or newly repaired water mains, wells, tanks, and masonry reservoirs can be chlorinated with the help of portable chlorinators. Initial chlorine concentrations are made high enough (about 50 mg/L) to overcome the chlorine demand of pollutants accumulated during construction. Chlorination is repeated, if necessary, until a residual of about 1 mg/L after 24 h is reached. After this the structure is flushed out thoroughly before being placed in service.

Solutions of chlorinating, ammoniating, and sulfonating compounds are commonly added to water through chemical-reagent feeders. For the chlorination of new mains, effective amounts of stabilized calcium hypochlorite may be spotted at suitable distances in the main during construction. Although chlorine can be manufactured at treatment works by the electrolysis of brine in electrolytic cells, it seldom pays to do this.

EXAMPLE 18.6 GAS CHLORINE DOSAGE DETERMINATION

Calculate the chlorine dose in both mg/L and lb/MG when a gas chlorinator is set to feed 8.172 kg/d (18 lb/d) of chlorine continuously to 2.157 million liters of water per day (2.157 MLD; 0.025 m^3/s; 0.57 MGD).

Solution 1 (US Customary System):

Chemical disinfectant dose = (Disinfectant feed rate)/(Water flow rate)
Chlorine dose = (Chlorine feed rate)/(Water flow rate)
1 mg/L = 1 ppm = 8.34 lb/MG

$$\mathrm{CD} = Q_\mathrm{d}/Q_\mathrm{w} \tag{18.22}$$

where

CD = chemical disinfectant dose, mg/L (lb/MG)
Q_d = disinfectant feed rate, mg/d (lb/d)
Q_w = water flow rate, L/d (MGD)

Given:

Q_d = disinfectant (chlorine) feed rate = 8.172 kg/d = 8,172,000 mg/d
Q_d = disinfectant (chlorine) feed rate = 18 lb/d
Q_w = water flow rate = 2.157 MLD = 2,157,000 L/d
Q_w = water flow rate = 0.57 MLD = 0.57 MGD

Accordingly,

CD = chemical disinfectant (chlorine) dose
= (8,172,000 mg/day) / (2,157,000 L/d)
= 3.78 mg/L.
CD = chemical disinfectant (chlorine) dose
= (18 lb/d) / (0.57 MG/d)
= 31.58 lb/MG
= (31.58) / (8.34) mg/L
= 3.78 mg/L.

Solution 2 (SI System):
Chemical disinfectant dose = (Disinfectant feed rate)/(Water flow rate)
Chlorine dose = (Chlorine feed rate)/(Water flow rate)
1 mg/L = 1 ppm

$$CD = Q_d/Q_w \tag{18.22}$$

where

CD = chemical disinfectant dose, mg/L
Q_d = disinfectant feed rate, mg/d
Q_w = water flow rate, L/d

Given:

Q_d = disinfectant (chlorine) feed rate = 8.172 kg/d = 8,172,000 mg/d
Q_w = water flow rate = 2.157 MLD = 2,157,000 L/d

Accordingly,
CD = chemical disinfectant (chlorine) dose
= (8,172,000 mg/d) / (2,157,000 L/d)
= 3.78 mg/L.

18.16.1 Water Chlorination Applications

Rough estimates of chlorine dosage for *marginal* (minimal) treatment of water may be based on the values listed in Tables 18.12 and 18.13, respectively, if reasonable allowances are made for water temperature (Table 18.5) and water quality or wastewater strength. However, correct disinfectant dosage determination must be conducted based on (1) chlorine demand (or disinfectant demand) and (2) *Ct* values. The *C* value is chlorine residual (or disinfectant residual) after the chlorine demand (or disinfectant demand) has been

Table 18.12 Minimum chlorine residuals for drinking water at 20°C

pH value	6–7	7–8	8–9	9–10	10–11
Free available chlorine (mg/L) after 10 min	0.2	0.2	0.4	0.8	0.8
Combined available chlorine (mg/L) after 60 min	1.0	1.5	1.8	1.8	—

Source: Fair et al. (1971).

Table 18.13 Probable amounts of chlorine required for producing a chlorine residual of 0.5 mg/L after 15 min in wastewater and wastewater effluents

Type of wastewater or effluent	Probable amounts of chlorine (mg/L)
Raw wastewater, depending on strength and staleness	6–24
Settled wastewater, depending on strength and staleness	3–18
Chemically precipitated wastewater, depending on strength	3–12
Trickling-filter effluent, depending on performance	3–9
Activated-sludge effluent, depending on performance	3–9
lntermittent-sand-filter effluent, depending on performance	1–6

Source: Fair et al. (1971).

satisfied, and is measured at the end of the contact time (t). The following are the equations for chlorine (or chemical disinfectant) dosage and Ct value determinations:

$$\text{Chlorine dosage (mg/L)} = \text{Chlorine demand (mg/L)} + \text{Chlorine residual (mg/L)} \quad \textbf{(18.23)}$$

$$\text{Disinfectant dosage (mg/L)} = \text{Disinfectant demand (mg/L)} + \text{Disinfectant residual (mg/L)} \quad \textbf{(18.24)}$$

$$Ct = [\text{Chlorine residual (mg/L)}]\,[\text{Contact time (min)}] \quad \textbf{(18.25)}$$

$$Ct = [\text{Disinfectant residual (mg/L)}]\,[\text{Contact time (min)}] \quad \textbf{(18.26)}$$

The Ct value which is the product of the concentration of disinfectant (such as, chlorine, in mg/L) and the length of contact time (in min) must be sufficient to ensure adequate disinfection.

Dosimeter capacity must be flexible enough to meet both common and unusual chlorine demands. Examples are both sudden and longitudinal changes in water and wastewater quality by drought and flood or by accidental spills of industrial chemicals.

In drinking water, chlorine residuals of 0.2–1.0 mg/L after 15–30 min of contact will generally produce 99.9% destruction for *E. coli* and 37°C bacterial counts. A 15-min free chlorine residual of 0.5 mg/L appears to be a safe average.

In treatment works, disinfecting chlorine may be added to the raw water (*prechlorination*), the partially treated water, or the finished water (*postchlorination*). The water may be chlorinated more than once within the treatment plant and afterward. In-plant, pre-, and postchlorination, and the rechlorination of the effluent from open distribution reservoirs are examples.

EXAMPLE 18.7 CHLORINE DEMAND CALCULATION

Estimate the chlorine demand for a water supply in mg/L if the chlorine dose is 3.9 mg/L and the chlorine residual is 0.9 mg/L.

Solution:

Disinfectant demand (mg/L) = Disinfectant dose (mg/L) – Disinfectant residual (mg/L)
Chlorine demand (mg/L) = Chlorine dose (mg/L) – Chlorine residual (mg/L)

$$\text{DD} = \text{CD} - \text{DR} \quad \textbf{(18.27)}$$

where

DD = disinfectant demand, mg/L
CD = chemical disinfectant dose (chlorine dose), mg/L
DR = disinfectant residual (chlorine residual), mg/L

Given:

CD = chemical disinfectant dose (chlorine dose) = 3.9 mg/L
DR = disinfectant residual (chlorine residual) = 0.9 mg/L

Accordingly,

DD = disinfectant demand = chlorine demand
= 3.9 mg/L – 0.9 mg/L
= **3.0 mg/L.**

18.16.2 Additional Chlorination Applications

Chlorine is a useful disinfectant of bathing waters and shellfish as well as of drinking water and wastewaters. The heavy load of bathers imposed on swimming pools, the necessity of breathing through the mouth during speed swimming, and the plunger action exerted by air and water on the nasal passages and sinuses during diving combine to release large amounts of mucus and vast numbers of nasopharyngeal organisms into swimming pools. To keep contagion at minimum level, effective disinfectants must be present in all parts of swimming pools in sufficient concentration to ensure rapid disinfection of their waters without becoming irritants.

For the chemical disinfection of edible shellfish, the mollusks are transferred from their growing grounds to tanks to which enough chlorine is added to disinfect the water. The shellfish uptake disinfected water whenever the concentration of chemicals drops to tolerable levels. Through successive exposures of this kind, the shellfish rid themselves of organisms ingested before they are harvested from their growing grounds.

Among uses of chlorine other than disinfection are the following:

1. Destruction or control of undesirable growths of algae and other organisms in water and wastewaters, examples being iron-fixing and slime-forming bacteria in pipelines and other water conduits, slime-forming bacteria in sewers and wastewater treatment works, freshwater mussels and clams in water conduits, and filter flies (*Psychoda*) and ponding slime growths on trickling filters
2. Improvement of the coagulation of water and wastewaters and of the separation of grease from wastewaters
3. Control of odors in water and wastewater
4. Stabilization of settling-tank sludges in water purification works and the control of odors associated with sludge treatment, including its drying
5. Prevention of anaerobic conditions in sewerage systems and wastewater treatment works by delaying or reducing decomposition
6. Conversion of cyanides to cyanates, such as NaOCN, in alkaline industrial wastes
7. Destruction of hydrogen sulfide in water and wastewater, and the protection of concrete, mortar, and paint against the corrosive action of this gas
8. Reduction of the immediate oxygen requirements of returned activated sludge and of digester liquor within treatment plants
9. Reduction or delay of the BOD of wastewaters discharged into receiving waters
10. Preparation at the plant of chlorinated copperas, a useful coagulant

Of these purposes, the reduction of BOD by chlorination deserves amplification here. Four kinds of reactions are conceivably involved: (1) direct oxidation of BOD-exerting compounds; (2) formation with nitrogen compounds of bactericidal chloramines by substitution of chlorine for hydrogen; (3) formation with carbon compounds of substances that are no longer decomposable, again by substitution of chlorine for hydrogen; and (4) addition of chlorine to unsaturated compounds to form nondecomposable substances. Often quoted is the observation that the application to municipal wastewaters of enough chlorine to produce a measurable residual after 15 min will reduce the 5-day, 20°C BOD by 15–35%—or in the ratio of about 2 mg/L of BOD to 1 mg/L of chlorine. Snow has shown that BOD reduction depends not only on chlorine concentration, but also on the condition of the wastewater. Among his observations are the following. Chlorination of fresh wastewater to a trace of residual at 15 min gave a reduction of but 10%; in stale wastewater it was 25–40%. Doses of 100–300 mg/L were required for a reduction of 35% in fresh wastewater; breakpoint dosage eliminated 7.5%. Aeration prior to chlorination improved the BOD reduction of fresh wastewater. For all aerobic samples, a lowering of both the first-stage demand and its rate of BOD exertion was observed. In anaerobic, chlorinated wastewater, the BOD rate was stepped up.

Destruction of odors by oxidizing chemicals is successful when the reactions involved produce nonodorous substances. This is not always so. The reaction between chlorine and hydrogen sulfide is an example of a successful reaction. Elementary, nonodorous sulfur is precipitated. The stronger the oxidizing agent, the more certain is the elimination of the offending substances. That is why breakpoint chlorination is so effective. The production of chlorophenol by marginal chlorination and the intensification of tastes when water containing *Synura* or other algae is chlorinated speak against halfway measures.

18.16.3 Manageable Variables in Halogenation

All disinfection processes involving the use of chlorine, sodium hypochlorite, calcium hypochlorite, bromine, chlorine dioxide, and so on are *halogenation processes*.

As stated before, degree of mixing or stirring, the length of detention, the pH, and the disinfectant concentration or intensity are the principal manageable variables in water disinfection. Although it is generally impractical to modify water quality or temperature for the specific purpose of improving disinfection, it is practical to choose a disinfecting process that is optimal for the water to be treated and to take advantage of in-plant conditions that may optimize disinfecting efficiency. Examples are

1. Split treatment with chemical disinfectants before and after filtration in order to ensure lasting residuals in the product water

2. Adding of chlorine well in advance of water stabilization by lime in order to take advantage of a low pH value
3. Postponing chlorination until recarbonation has lowered the pH in lime-soda softening works
4. Chlorinating fresh urban wastewaters in preference to stale or septic ones
5. Preforming chloramines for the disinfection of urban and other wastewaters that have a large chlorine but low chloramine demand (useful for bacteria disinfection but not for virus disinfection)
6. Chlorinating stormwater overflows, main outflows from separate stormwater systems, and effluents from wastewater treatment plants in such fashion as to make use of available times of storage

Disinfectant concentration is a matter of required *dosimetry*, that is, the selection of feeding equipment of sufficient capacity to cover not only normal requirements but also unusual demands that may occur when water and wastewater systems are placed under stress. Chlorine demands large enough to exceed the capacity of available equipment have been produced by (1) flash floods that pass rapidly through storage reservoirs by displacing the waters of small reservoirs or underrunning or overrunning even large reservoirs when the floodwaters are, respectively, colder or warmer than the stored waters; (2) overflow of swamps during spring freshets; (3) break-up of ice jams and release of pools of polluted water accumulating behind them; and (4) shifts in stream and lake currents that transport wastewater effluents to water intakes rapidly and in high concentration.

To ensure sufficient time of contact, holding or contact units should be given enough capacity, unless it is possible to capitalize on detention times otherwise available in the treatment works or in transmission and distribution systems. Quiescence and associated reduction in short-circuiting are not necessarily optimal conditions. In continuous treatment stirring or controlled power input may be more certain and more effective.

18.17 OPERATIONAL TECHNOLOGY OF SODIUM HYPOCHLORINATION

Sodium hypochlorite solution is usually supplied commercially in concentrations of 5% and 15% chlorine. It is easier to handle than gaseous chlorine or calcium hypochlorite. Metered chlorinators deliver the solution directly into the water.

Sodium hypochlorite solutions lose their disinfecting (oxidizing) power during storage, and thus should be stored in a cool, dark, dry area. No more than a 1-month supply of the chemical should be purchased at one time to prevent loss of available chlorine. The material is supplied in glass or plastic bottles, carboys, or lined drums ranging in size from 1.89 to 208.2 L (0.5–55 gal). Bulk shipment by tank truck is also a common form of transport.

Sodium hypochlorite solution is more costly per pound of available chlorine and does not contain the high concentrations of chlorine available from chlorine gas. However, the handling and storage costs are lower than for chlorine gas.

An onsite generation technique for hypochlorite solutions has been developed. This system consists of a two-cell unit, in which a brine solution (salt in water) is electrolyzed, producing a solution of hypochlorous acid in one cell and a solution of caustic (sodium hydroxide) in the other, according to the following equation:

$$\mathrm{Na^+ + Cl^- + 2H_2O + e- \rightarrow HOCl + NaOH + H_2} \tag{18.28}$$

Using onsite generation avoids the purchase and storage of large volumes of chlorine gas or hypochlorite solutions, but there are significant disadvantages. The generation process produces hydrogen, which poses fire and explosion hazards, and sodium hydroxide, which is a caustic solution that requires proper disposal. Also, the cost per pound of available chlorine updated to 2014 is typically much higher for onsite generation (e.g., USD 1.37–1.60/kg) [USD 0.62–USD 0.73/lb] for onsite generation compared to USD 0.37–0.68/kg [USD 0.17–USD 0.31/lb] for chlorine gas. However, certain site-specific considerations may make onsite generation a preferred disinfection technique.

18.17.1 Equipment Costs

Table 18.14 presents estimated capital costs for sodium hypochlorite chlorination systems. This table provides estimates for basic and complex systems, both electrically and hydraulically activated. The basic liquid hypochlorination systems include two metering pumps (one serving as a standby), a solution tank, diffuser, and appropriate quantities of tubing. The more complex system adds a diffuser corporation cock, antisiphon backflow preventer, a safety housing enclosure, a flow pacing system, and a flow meter and signal.

Total capital costs for an electrically activated system range from USD 8,850 for the basic system to USD 25,960 for the most sophisticated system. The comparable range for the hydraulically activated systems is USD 12,200–36,565.

18.17.2 Operating and Maintenance Costs

As with solution-feed gas chlorinators, operating and maintenance costs for systems in the 9.5 m^3/d to 0.044 m^3/s (2,500 gpd to 1 MGD) size range are roughly the same. The annual estimated energy requirements for the diaphragm metering pump and the housing structure, assumed to be 58.1 m^2 (625 ft^2), are 570 and 2,560 kWh, respectively.

Table 18.14 Capital costs for liquid chlorinators in 2014 USD

	Electrically activated	Hydraulically activated
	2014 USD	
Equipment cost (basic system)	4,344	5,461
Installation	1,241	2,482
Site work	620	620
Contractor's overhead and profit (20%)	1,765	2,420
Engineering fees	881	1,216
Add ons:		
Alternate #1: add diffuser corporation cock and antisiphon backflow preventer	397	558
Alternate #2: add safety enclosure (housing)	16,718	16,718
Alternate #3: add flow pacing existing signal		3,586
Alternate #4: add flow meter signal, 8 in. (20.3 cm) or less		3,500
Total capital cost		
Basic system	8,850	12,200
Most sophisticated	25,960	36,565

Source: Updated US EPA.

Maintenance materials for minor component repairs are about USD 49 each year. Approximately 1 hour of labor is required each day to mix the sodium hypochlorite solution and check equipment. The total annual operation and maintenance cost for the sodium hypochlorite solution feed system is USD 9,490/yr.

18.17.3 Chemical Costs

Typically, sodium hypochlorite is available as a 15% (by weight) solution. Four-tenths of a kg (0.9 lb) of sodium hypochlorite solution is equivalent in oxidation potential to 0.45 kg (1 lb) of gaseous chlorine, and its cost is about three times that of gaseous chlorine.

EXAMPLE 18.8 LIQUID DISINFECTANT CONCENTRATION CALCULATION

Water pumped from a well is disinfected by a hypochlorinator. During a 7-day period, the water meter indicated that 1,000,000 gal (3,785,000 L) of water was pumped. A 2.2% sodium hypochlorite solution is stored in a 2.5-ft (0.763-m) diameter plastic tank. During this 7-day period, the level of hypochlorite in the tank dropped 1.5 ft (0.4572 m). What was the chlorine dose in mg/L?

Solution 1 (US Customary System):

$$(V1)(C1) = (V2)(C2) \tag{18.29}$$

$$(Q1)(C1) = (Q2)(C2) \tag{18.30}$$

where

$V1$ = first volume of liquid disinfectant, gal or ft^3
$C1$ = first concentration of liquid disinfectant, %
$V2$ = second volume of liquid disinfectant, gal or ft^3
$C2$ = second concentration of liquid disinfectant, %
$Q1$ = first flow of liquid disinfectant, gpm or ft^3/min
$Q2$ = second flow of liquid disinfectant, gpm or ft^3/min

Given:

$V1 = (2.5 \times 2.5 \times 0.785 \times 1.5\ \text{ft}^3)\ (7.48\ \text{gal/ft}^3) = 55.05$ gal
$C1 = 2.2\%$
$V2 = 1{,}000{,}000$ gal
$C2 = ?$

Results:

$(55.05\ \text{gal})(2.2\%) = (1{,}000{,}000\ \text{gal})(C2)$

$C2 = (55.05\ \text{gal})(2.2\%)\ /(1{,}000{,}000\ \text{gal})$

$= 0.0001211\%$

$= \mathbf{1.2\ mg/L.}$

Solution 2 (SI System):

$$(V1)(C1) = (V2)(C2) \tag{18.29}$$

where

$V1$ = first volume of liquid disinfectant, L

$C1$ = first concentration of liquid disinfectant, %

$V2$ = second volume of liquid disinfectant, L

$C2$ = second concentration of liquid disinfectant, %

Given:

$V1 = (0.763 \times 0.763 \times 0.785 \times 0.4572\ \text{m}^3)\ (1{,}000\ \text{liters/m}^3) = 208.94\ \text{L}$

$C1 = 2.2\%$

$V2 = 3{,}785{,}000\ \text{L}$

$C2 = ?$

Results:

$(208.94\ \text{L})(2.2\%) = (3{,}785{,}000\ \text{L})(C2)$

$C2 = (208.94\ \text{L})(2.2\%)\ /\ (3{,}785{,}000\ \text{L})$

$= 0.0001211\%$

$= \mathbf{1.2\ mg/L.}$

EXAMPLE 18.9 DISINFECTANT WEIGHT CALCULATION

Calculate the weight (in both kg and lb) of chlorine used to disinfect water, if 567.75 L (150 gal) of hypochlorite as a 2.6% chlorine solution was used.

Solution 1 (US Customary System):

$$W(\text{lb}) = V(\text{gal})(8.34\ \text{lb/gal})(\text{SG})(P/100) \tag{18.31}$$

where

W = weight of disinfectant, lb

V = volume of liquid disinfectant, gal

SG = specific gravity, normally 1

P = percent concentration of disinfectant (%)

Given:

$V = 567.75\ \text{L} = 150\ \text{gal}.$

SG = 1 (assumed)

8.34 lb/gal = English system conversion factor (1 gal of water weighs 8.34 lb)

P = percent concentration of disinfectant = 2.6%

Results:

W (lb) = V (gal) (8.34 lb/gal)(SG)(P/100)

= (150 gal) (8.34 lb/gal)(1)(2.6/100)

= 32.52 lb of chlorine.

Solution 2 (SI System):

$$W(\text{kg}) = V(\text{L})(1\ \text{kg/L})(\text{SG})(P/100) \quad \textbf{(18.32)}$$

where

W = weight of disinfectant, kg

V = volume of liquid disinfectant, L

SG = specific gravity, normally 1

P = percent concentration of disinfectant (%)

Given:

V = 567.75 L

SG = 1 (assumed)

1 kg/L = metric system conversion factor (1 L of water weighs 1 kg)

P = percent concentration of disinfectant = 2.6%

Results:

W (kg) = V (L) (1 kg/L)(SG)(P/100)

= (567.75 L) (1 kg/L)(1)(2.6/100)

= 14.76 kg of chlorine.

18.18 OPERATIONAL TECHNOLOGY OF CALCIUM HYPOCHLORINATION

Calcium hypochlorite is a white solid that can be purchased in granular, powdered, or tablet form. It contains 65% available chlorine and is readily soluble in water. The chemical is available in 0.9, 2.3, 3.6, and 15.9 kg (2, 5, 8, and 35 lb) cans and 362.9 kg (800 lb) drums, which are usually resealable. Calcium hypochlorite is a corrosive material with a strong odor and requires proper handling.

When packaged, calcium hypochlorite is very stable; therefore, an annual supply can be purchased in a single procurement. However, it is hygroscopic (readily absorbs moisture) and reacts slowly with atmospheric moisture to form chlorine gas. Therefore, shipping containers must be emptied completely or carefully resealed. Bulk handling systems cannot be used.

Typically, the entire contents of a calcium hypochlorite container are emptied into a mixing tank where they are readily and completely dissolved in water. The resulting corrosive solution is stored in and fed from a stock solution vessel constructed of corrosion-resistant materials such as plastic, ceramic, glass, or rubber lined steel. Solutions of 1% or 2% available chlorine can be delivered by a diaphragm type, chemical feed/metering pump.

Equipment, operating, and maintenance costs for calcium hypochlorite solution feed systems are similar to those for sodium hypochlorite feed systems. The equipment needed to mix the solution and inject it into the water being treated is the same.

A 9.5 m^3/d (2,500 gpd) treatment plant using a 5 mg/L dosage of chlorine needs 0.104 lb chlorine/d. Because solid calcium hypochlorite contains 65% available chlorine, 15.95 kg (0.16 lb)/d is required. In 2014, 1 kg of calcium hypochlorite costs USD 3.96 (USD 1.80/lb). For a 9.5 m^3/d (2,500 gpd) facility, 26.5 kg (58.4 lb) costing USD 105.35 is needed for 1 year. A 0.044 m^3/s (1 MGD) facility, using 400 times that amount of chlorine, would spend USD 42,140 annually.

18.19 OPERATIONAL TECHNOLOGY OF CHLORINE DIOXIDE DISINFECTION

For drinking water treatment, chlorine dioxide is generated from solutions of sodium chlorite ($NaClO_2$), which is usually purchased as a 25% aqueous solution or as a solid (80%

sodium chlorite). Historically, chlorine dioxide has been produced by treating sodium chlorite with chlorine gas, sodium hypochlorite solution and mineral acid, or mineral acid alone. In all three cases, the appropriate aqueous solutions of reactants are metered into and mixed in a chlorine dioxide reactor, which is a cylinder containing flow distributing packings, such as Raschig rings, glass beads, or hollow glass cylinders. Residence time of the solutions in a properly sized reactor is only a few seconds. The resulting yellow solution is pumped directly into the water to be treated.

In this manner, chlorine dioxide solutions are generated as the material is required and used immediately. Reactor operations are automated with appropriate metering and instrumentation that controls the addition of chlorine dioxide according to the flow rate of the water being treated.

The three historical techniques for generating chlorine dioxide are discussed in detail by US EPA. However, some of these procedures can result in excess free chlorine being present. Free chlorine can oxidize chlorine dioxide to form chlorate ions, which are difficult to remove from solution. Consequently, the current recommended approach to chlorine dioxide generation is to maximize its yield while minimizing the presence of free chlorine (thus minimizing the formation of chlorate ion). Slootmaekers et al., 1989 discusses generation techniques to meet these objectives.

For water disinfection, chlorine dioxide can be generated using several reaction schemes (see Figs. 18.10 and 18.11), such as the reaction of aqueous hypochlorous acid with dissolved chlorite ion:

$$2NaClO_2 + HOCl \rightarrow NaCl + NaOH + 2ClO_2 \quad \textbf{(18.33)}$$

Chlorine dioxide can also be generated by the reaction of solid sodium chlorite in solution with mineral acid, with chlorine or with hypochlorous acid. The reaction for chlorine and/or hypochlorous acid with chlorite ion is

$$2ClO_2^- + Cl_2(g) \rightarrow 2ClO_2(g) + 2Cl^- \quad \textbf{(18.34)}$$

$$2ClO_2^- + HOCl \rightarrow 2ClO_2(g) + Cl^- + OH^- \quad \textbf{(18.35)}$$

These reactions involve the formation of the unsymmetrical intermediate, Cl_2O_2:

$$Cl_2 + ClO_2 \rightarrow Cl_2O_2 + Cl^- \quad \textbf{(18.36)}$$

At high concentrations of both reactants, the intermediate is formed very rapidly. Elemental chlorine formed by Eq. (18.37) is recycled by means of Eq. (18.36). Thus, primarily chlorine dioxide is produced as a result:

$$2Cl_2O_2 \rightarrow 2ClO_2 + Cl_2 \quad \textbf{(18.37)}$$

or

$$Cl_2O_2 + ClO_2 \rightarrow 2ClO_2 + Cl^- \quad \textbf{(18.38)}$$

On the other hand, at low initial reactant concentrations, or in the presence of excess hypochlorous acid, primarily chlorate ion is formed, due to the following reactions:

$$Cl_2O_2 + H_2O \rightarrow ClO_3^- + Cl^- + 2H^+ \quad \textbf{(18.39)}$$

and

$$Cl_2O_2 + HOCl \rightarrow ClO_3^- + Cl^- + H^+ \quad \textbf{(18.40)}$$

Thus, high concentrations of excess chlorite ion favor the second-order reactions (Eqs. 18.37 and 18.38) and chlorine dioxide is formed. At low concentrations, the second-order disproportionation process becomes unimportant, and reactions in Eqs. (18.37) and (18.38) produce chlorate ion rather than chlorine dioxide. The reasons for the production of chlorate ion are related to the presence of high concentrations of free chlorine and the rapid formation of the Cl_2O_2 intermediate, which, in turn, reacts with the excess hypochlorous acid to form the unwanted chlorate ion.

The stoichiometry of the undesirable reactions which forms chlorate ion is

$$Cl_2O_2^- + HOCl \rightarrow ClO_3^- + Cl^- + H^+ \quad \textbf{(18.41)}$$

$$ClO_2^- + Cl_2 + H_2O \rightarrow ClO_3^- + 2Cl^- + 2H^+ \quad \textbf{(18.42)}$$

Therefore, the most effective way to minimize chlorate ion formation is to avoid conditions that result in low reaction rates (e.g., high pH values and/or low initial reactant concentrations, and the presence of free hypochlorous acid). Clearly, the reaction forming chlorate ion (Eq. 18.33) will be more troublesome in dilute solutions. On the other hand, whenever treatment by chlorine dioxide (which forms chlorite ion in the process) is followed by the addition of free chlorine (HOCl with a pH of 5–8), the unwanted chlorate ion will also be formed.

Establishing a chlorine dioxide residual: Laboratory studies have shown that about 70% of the chlorine dioxide added to drinking water is converted to chlorite ion. Therefore, 1.2–1.4 mg/L chlorine dioxide is the maximum practical dosage to meet the currently recommended maximum total oxidant residual of 1 mg/L. However, Slootmaekers et al., 1989 report that nearly the entire chlorine dioxide ion added as a primary oxidant/disinfectant is converted to chlorite ion. Because of differences in the nature of water constituents that exert demand for chlorine dioxide, this ratio should be individually determined for each water supply.

18.20 OPERATIONAL TECHNOLOGY OF OZONATION

While ozone is widely used for disinfection and oxidation in other parts of the world, it is relatively new in the United States.

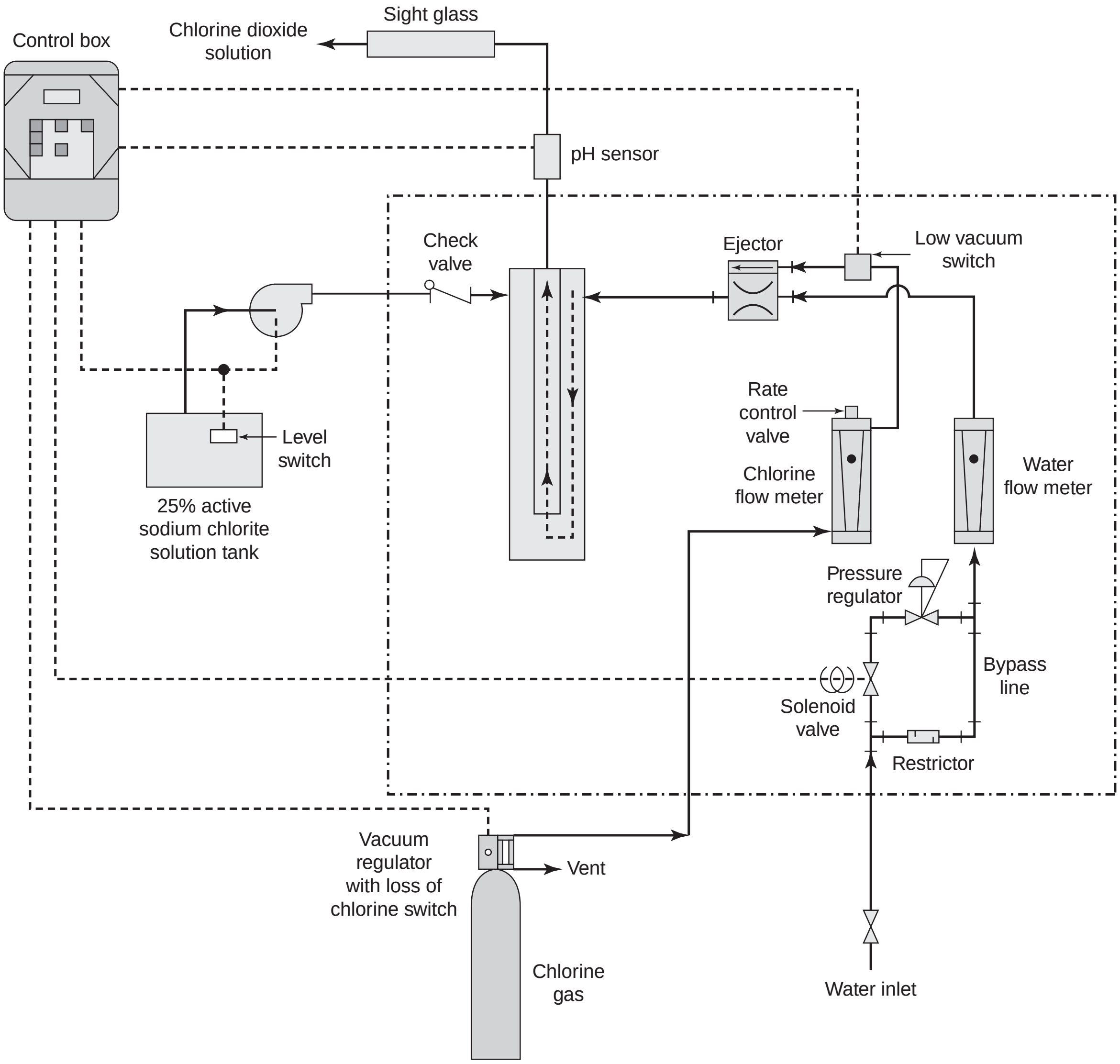


Figure 18.10 Conventional chlorine dioxide generation using chlorine–chlorite method (After Demers and Renner, 1992).

18.20.1 Process Description

Ozone (O_3) is a powerful oxidizing agent, second only to elemental fluorine among readily available chemical supplies. Because it is such a strong oxidant, ozone is also a powerful disinfectant. Unlike chlorine, ozone does not react with water to produce a disinfecting species. Instead, when exposed to a neutral or alkaline environment (pH above 6), UV light, or hydrogen peroxide, it decomposes in water to more reactive hydroxyl free radicals as shown in the equation below:

$$O_3 + H_2O \rightarrow O_2 + 2(OH) \qquad \textbf{(18.43)}$$

This reaction is accelerated at pH values above 8.

In water, ozone reacts as the ozone molecule, the hydroxyl free radical, or as a mixture of both. For primary disinfection, *Ct* values for ozone have been developed for molecular ozone, not for hydroxyl free radicals. These free radical species are more effective oxidizing agents than molecular ozone. However, they have extremely short half-lives (microseconds) and consequently may not be good disinfectants.

Since ozone is unstable at ambient temperatures and pressures, it must be generated onsite and used quickly. Ozone is generated by applying energy to oxygen (pure oxygen or dried air). A high-energy electrical field (see Fig. 18.12) causes oxygen to dissociate according to the equation below:

$$O_2 + e- \rightarrow 2[O] \qquad \textbf{(18.44)}$$

These oxygen “fragments”, [O], are highly reactive and combine rapidly with molecular oxygen to form the triatomic molecule, ozone:

$$2[O] + 2O_2 \rightarrow 2O_3 \qquad \textbf{(18.45)}$$

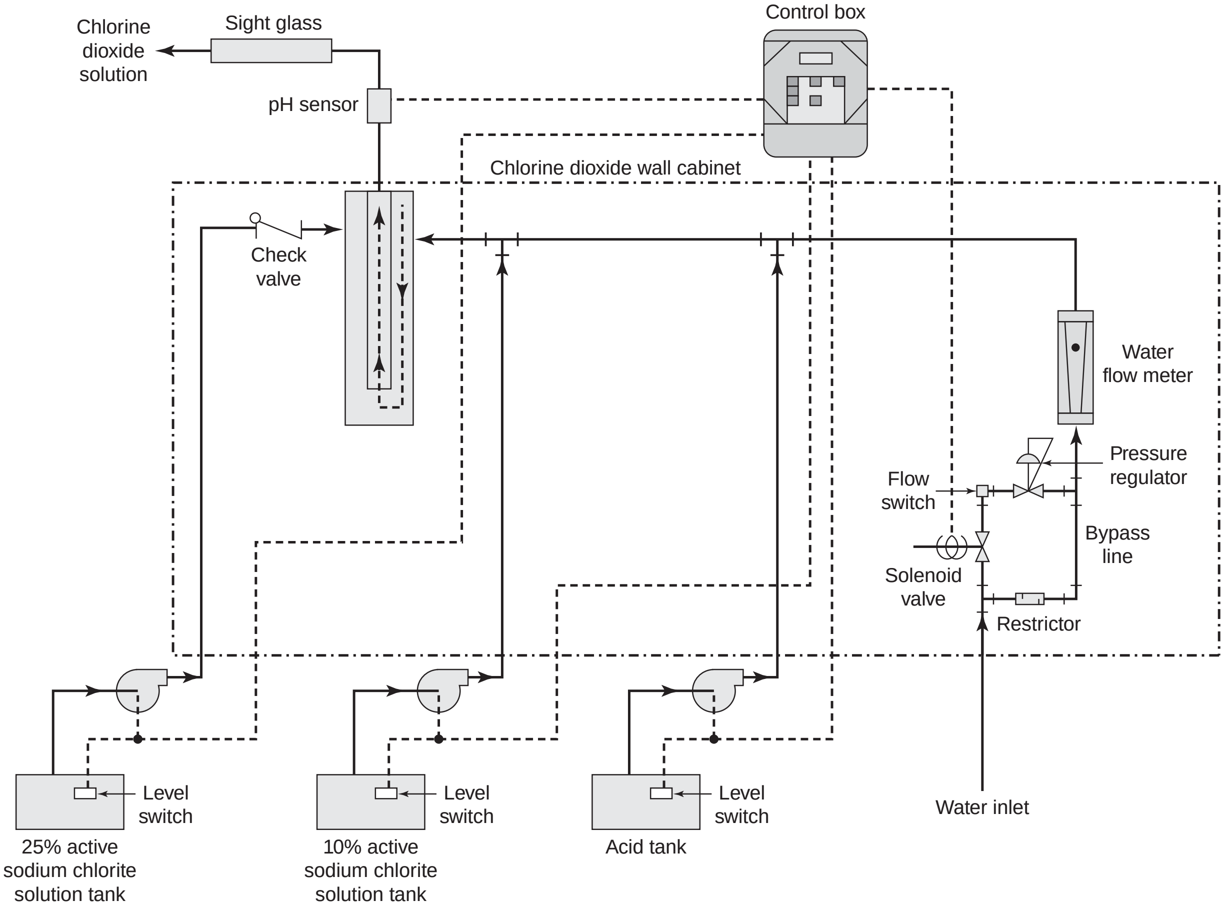


Figure 18.11 Manual feed equipment arrangement for generating chlorine dioxide from sodium hypochlorite solution and mineral acid. (After Demers and Renner, 1992).

The overall reaction that produces ozone is the sum of the above reactions:

$$3O_2 + e- \rightarrow 2O_3 \tag{18.46}$$

This reaction is reversible; once formed, ozone decomposes to oxygen. This reverse reaction occurs quite rapidly above 35°C. Because the reactions that convert oxygen to ozone also produce a considerable amount of heat, ozone generators have cooling components to minimize ozone losses by thermal decomposition.

Ozone has a characteristic odor that is detectable even at low concentrations (0.01–0.02 ppmv). Higher levels may cause olfactory and other reaction fatigue, and much higher levels are acutely toxic in some instances. The longer the exposure to ozone, the less noticeable is the odor.

Ozone is only slightly soluble in water, about 2–10 times more soluble than oxygen, depending on the temperature and its concentration as it enters the ozone contactor (see Fig. 18.13). The higher the concentration of ozone generated, the more soluble it is in water. Increasing pressure in the ozone contactor system also increases its solubility. Ozone's half-life in water ranges from 8 min to 14 h, depending on the level of ozone-demanding contaminants in the water.

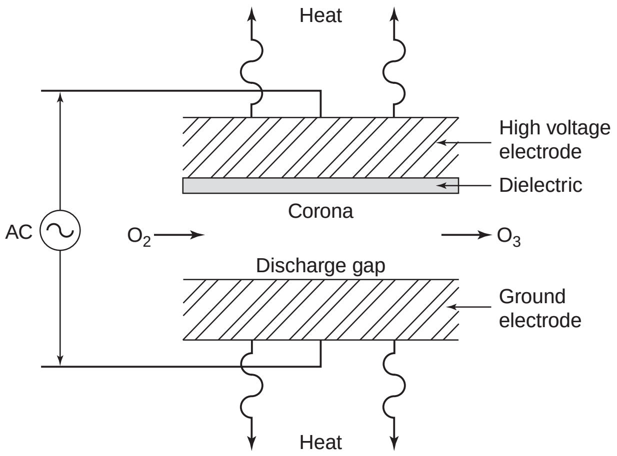
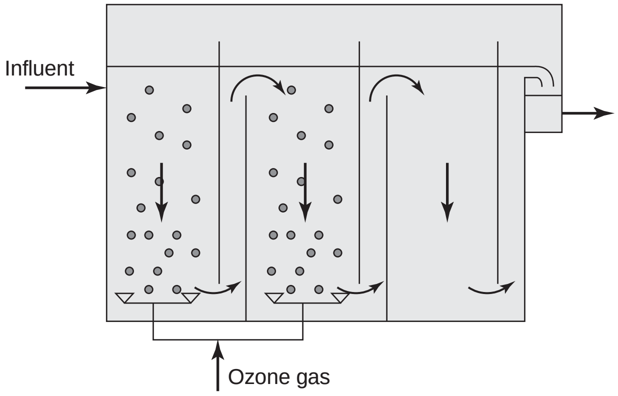


Figure 18.12 Typical ozone generating system (*Source*: U.S. EPA, 1999).

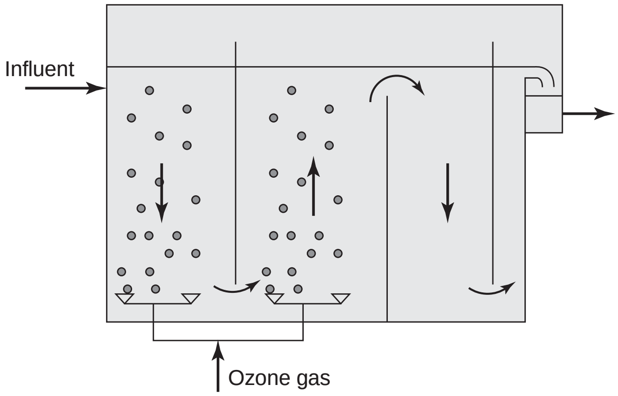
18.20.2 System Design Considerations

The five major elements of an ozonation system are

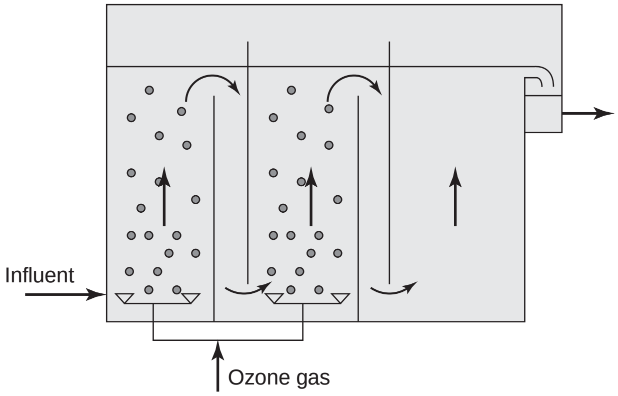
1. Air preparation or oxygen feed
2. Electrical power supply



(*a*) Counter current contactor



(*b*) Counter and cocurrent contactor



(*c*) Cocurrent contactor

Figure 18.13 Ozone bubble contactors (*Source*: U.S. EPA, 1999).

3. Ozone generation
4. Ozone contacting
5. Ozone contactor exhaust gas destruction

18.21 OPERATIONAL TECHNOLOGY OF UV DISINFECTION

UV radiation (254 nm) disinfection of bacteria and viruses has several important advantages: (1) it is readily available; (2) it produces no toxic residuals; (3) the required contact times are relatively short; and (4) the equipment is easy to operate and maintain, although maintenance must be performed on a regular basis to prevent fouling of certain components.

UV radiation disinfection is inappropriate for

1. Inactivation of *Giardia lamblia* cysts
2. Water containing high suspended solids concentrations, color, and turbidity
3. Water with high concentrations of soluble organic matter that can react with or absorb the UV radiation, thus reducing the disinfectant's performance

Since UV radiation is ineffective against *Giardia* cysts, but effective against viruses and bacteria, it is a good candidate for disinfecting groundwater not directly influenced by surface water. If the amount of radiation received by a target organism is not a lethal dose, however, reconstitution of the organism and reinfection of the water can occur.

Since UV radiation disinfection provides no disinfecting residual, a secondary disinfectant is needed. Very little oxidation of organic materials occurs with typical UV radiation systems used for drinking water disinfection; consequently if oxidation is required (for iron, manganese, sulfide, nitrate, etc.), a strong oxidizing agent may be necessary and can serve as a primary disinfectant as well. However, higher energy intensities and lower UV wavelengths (184.9 nm) can produce oxidation reactions.

UV bulbs that produce radiation at 184.9 nm generate some quantities of ozone which, in turn, can provide some oxidation of organic materials. The combination of UV radiation and ozone produces the hydroxyl free radical, which is a more powerful oxidizing agent than is ozone itself.

The 184.9 wavelength radiation is not as effective for UV disinfection as the 254 nm wavelength, except by the amount of ozone generated, which will provide some *Ct* value.

18.21.1 UV Disinfection System Design Considerations

The basic design considerations for a UV system are

1. Satisfying the UV demand of the water
2. Maximizing the use of UV energy delivered by the lamps
3. Maintaining the conditions that encourage plug flow

UV lamps are usually submerged in the water, perpendicular or parallel to the water flow. Submerged lamps are inserted into a quartz sleeve to minimize the water's *fouling* effects. The further the distance between the water and the lamp, the weaker the radiation dosage delivered because the energy dissipates or becomes dilute as the space it occupies increases in volume. The UV demand of other contaminants in the water also consumes radiation.

Specific design parameters to consider are:

1. *Residence time distribution* (*RTD*)—This describes the detention time of the water in the reactor and should be determined for several flow conditions.
2. *Plug flow*—The ability to maintain plug flow in the reactor is influenced by the inlet and exit designs. Disturbances at the inlet and exit planes of the lamp battery should be minimized and necessary changes in the flow direction should be made outside the lamp battery.
3. *Dispersion number*—A key goal is to minimize the dispersion number, d (cm^2/s). As a design goal, d should be between 0.02 and 0.05. This number represents a plug-flow reactor with low to moderate dispersion. This value is attained by increasing the product of the velocity (cm/s) of and distance traveled (cm) by the water as it flows through the reactor while under direct exposure to UV radiation. However, extended lengths and higher velocities cause higher head losses; therefore, adjusting the dispersion number may be necessary to meet specific criteria for either full-scale modules or pilot units. Head loss is determined over a wide velocity range and excludes entrance and exit losses.
4. *Effective volume*—The inlet and outlet designs should achieve equivalent water velocities at all points entering and exiting the lamp battery. This maximizes the lamp battery use and improves cost-effectiveness. Stilling walls (perforated baffles) and weirs in the reactor design assist in controlling water velocities.

18.21.2 UV Lamp Designs

Lamps used in UV disinfection systems typically have arc lengths of approximately 0.8 and 1.5 m (2.5 and 4.9 ft) and full lengths of 0.9 and 1.6 m (3 and 5.3 ft), respectively. The arc length describes the active, light-emitting portion of the lamp. Lamp diameters typically are 1.52 and 2.0 cm (0.6 and 0.8 in.). A sleeve made of fused quartz or another material that is highly transparent to UV light, such as Vycor, protects lamps that are submerged. Nonsubmerged lamps are placed near the wall of the water conduit, which is made of a UV light-translucent material.

UV dose [milliwatt-seconds per square centimeter (mW-s/cm^2) or millijoules per square centimeter (mJ/cm^2)] is the most important operational parameter in the UV disinfection. It can be calculated by the UV intensity (mW/cm^2) multiplied by the UV exposure time (s). If the UV intensity is independent of time, the UV dose can be calculated by the following equation:

$$D = (I)(t) \tag{18.47}$$

where

D = UV dosage, mJ/cm^2 or mW-s/cm^2

I = UV light intensity in the bulk solution, mW/cm^2

t = exposure time, s

However, if the UV is dependent on time, the UV dose can then be determined by integration of the UV light intensity (I) over the exposure time (t):

$$D = \int_0^t (I)(\mathrm{d}t) \tag{18.48}$$

EXAMPLE 18.10 UV EXPOSURE TIME AND UV DOSAGE

Assuming (a) the UV lamps have been used for less than 1,000 h and are still emitting UV light at constant intensity; (b) water flow rate is 40,780.80 m^3/d (1,429,539 ft^3/d; 10.77 MGD); (c) UV reactor volume is 0.236 m^3 (8.33 ft^3); and (d) the UV light intensity in the water solution is 100 mW/cm^2, determine both UV exposure time and UV dosage, and discuss the factors which will affect the UV intensity.

Solution 1 (US Customary System):

The UV exposure time can be calculated by the following equation:

$$t = V/Q \tag{18.49}$$

where V = UV reactor volume, ft^3, and Q = water flow, ft^3/s.
Given:

$V = 0.236 \times 35.314\ \text{ft}^3 = 8.33\ \text{ft}^3$.

$Q = 10.77\ \text{MGD} = 1{,}429{,}539\ \text{ft}^3/\text{day} = 16.55\ \text{ft}^3/\text{s}$.

Accordingly,

$t = (8.33\ \text{ft}^3)/(16.55\ \text{ft}^3/\text{s})$

$= \mathbf{0.5\ s.}$

Given:

$t = 0.5$ s and $I = 100$ mW/cm^2.

Accordingly,

$$\begin{aligned} D &= (100 \text{ mW/cm}^2)(0.5 \text{ s}) \\ &= 50 \text{ mW-s/cm}^2 \\ &= \mathbf{50\ mJ/cm^2}. \end{aligned}$$

The UV intensity is mainly affected by water quality, equipment and UV lamp configuration, and equipment and lamp age and sleeve fouling.

Solution 2 (SI System):

The UV exposure time can be calculated by the following equation:

$$t = V/Q \qquad \textbf{(18.49)}$$

where V = UV reactor volume, m^3, and Q = water flow, m^3/s.

Given:

$V = 0.236$ m^3.

$Q = 40780.80$ m^3/d $= 0.472$ m^3/s.

Accordingly,

$$\begin{aligned} t &= (0.236 \text{ m}^3)/(0.472 \text{ m}^3\text{/s}) \\ &= \mathbf{0.5\ s}. \end{aligned}$$

Given:

$t = 0.5$ s and $I = 100$ mW/cm^2.

Accordingly,

$$\begin{aligned} D &= (100 \text{ mW/cm}^2)(0.5 \text{ s}) \\ &= 50 \text{ mW-s/cm}^2 \\ &= \mathbf{50\ mJ/cm^2}. \end{aligned}$$

The UV intensity is mainly affected by water quality, equipment and UV lamp configuration, and equipment and lamp age and sleeve fouling.

18.21.3 Factors Affecting the Design of the UV Disinfection System

Initial microorganism density, suspended solids (or turbidity), UV demand of the water at the disinfection point, and water flow rate all affect the size and performance of the UV disinfection system.

The performance of a UV disinfection unit relates directly to the initial density of the indicator organisms. The higher the initial density, the greater the dosage of radiation required. For this reason, microorganism density should be continually monitored. Turbidity directly affects the performance of the UV disinfection system as well. Particulates suspended in water block the UV radiation, thereby protecting bacteria and hindering disinfection. The UV demand of the water affects the radiation intensity in the reactor and, thus, affects the system size and the lamp placement that achieves the desired performance.

Water flow rate is another key factor in determining system size. Both the hydraulic load to the plant and the design of the processes preceding disinfection affect flow. The size of the UV system, however, should be based on peak flow rates and projected flows for the plant's design year rather than on average flows, which are used to predict operating and maintenance requirements.

EXAMPLE 18.11 FACTORS AFFECTING UV DISINFECTION

Discuss the factors that will affect the UV disinfection process.

Solution:

The factors affecting UV disinfection include UV dose, UV intensity, UV exposure time, and UV transmittance. The UV intensity, in turn, is affected by water quality, the configuration of UV equipment and lamps, the fouling of quartz sleeve, and the age of UV lamps.

Since UV dose, intensity, and exposure time have been discussed and illustrated previously in Example 18.2, only the remaining factors are discussed in below.

18.21.4 UV Transmittance

UV *Transmittance* (% *T*) is a measure of the ability of the water to transmit UV light. It is both a measure of water quality and an important design factor for sizing UV equipment:

$$\% T = (10^{-A})(100) \tag{18.50}$$

where

$\% T$ = UV transmittance, %
A = absorbance measured by an spectrophotometer at about 254 nm

Water with a higher UV Transmittance requires less UV energy to disinfect water. Reduced UV light transmittance in water results in a lower UV disinfection efficiency; this is sometimes referred to as UV demand.

Since UV dose = intensity × exposure time, a reduced intensity can be compensated for by increasing the exposure time (i.e., retention time) or the number of UV lamps.

UV transmittance is measured with a UV spectrophotometer set at a wavelength of about 254 nm. The transmittance of a sample in a 1 cm quartz path length cuvette is read as a percentage compared to deionized water (set at 100%).

Transmittance decreases in the presence of UV absorbing substances and particles that absorb UV light. This results in a reduction of available UV energy for disinfection.

Water transmittance depends on upstream treatment processes. In general, water processing equipment located upstream of the UV system, such as filters, can affect performance. Different treatment processes produce water with different UV transmittance characteristics and observed cleaning frequency patterns. The presence of dissolved organics will reduce transmittance.

18.21.4.1 Inorganic Compounds Water treatment processes may use metal salts (such as alum, polyaluminum chloride, ferric chloride) for enhanced solids removal, phosphate reduction, and odor control. Dissolved aluminum salts, for instance, have no effect on UV transmittance. Iron in water, however, absorbs UV light and can contribute to quartz sleeve fouling/coating.

18.21.4.2 Level of Total Dissolved Organics Specific organic compounds in the drinking water will absorb some energy, which is emitted in the germicidal wavelength region of the UV light spectrum.

18.21.4.3 Total Hardness in Water or Wastewater The presence of high levels of inorganic magnesium or calcium hardness in a water or wastewater may contribute to coating of the quartz sleeve.

18.21.5 UV Equipment Configuration

The UV reactor is configured to optimize the number of UV lamps required to provide a specific dose with the necessary hydraulic capacity. It is desirable to optimize UV equipment configuration that will minimize hydraulic head loss and produce adequate turbulent flow (mixing).

18.21.6 UV Lamp Age and Quartz Sleeve Fouling

UV intensity gradually decreases with time and use due to lamp aging and sleeve fouling. This is factored into the UV design, so that equipment will provide a specific UV dose at the end of lamp life.

An accumulation of inorganic and organic solids on the quartz sleeve decreases the intensity of UV light that enters the surrounding water. The fouling rate varies with the water quality and may be more rapid in the presence of high concentrations of iron, calcium, and magnesium ions. A modern UV system has an optional automatic wiping system that can significantly reduce the required operator maintenance time for cleaning.

18.21.7 UV System Operating and Maintenance Considerations

The intensity of radiation in the reactor depends on the lamp output and the reactor cleanliness. Therefore, monitoring lamp intensity and properly maintaining the reactor are essential to reliable system performance.

18.21.8 Operation and Maintenance of UV Lamps

Lamp output is influenced by lamp temperature, voltage potential across the lamp, and age of the lamp. Lamp temperature cannot be controlled in submerged lamp systems. In other systems, however, lamp temperatures are controlled by regulating ambient air temperatures using cooling fans or recirculating the heat generated by the lamp ballasts for warming.

Adjusting the voltage will vary the lamp output. Decreasing the voltage reduces current to the lamp and, therefore, lamp output. Voltage regulators improve system efficiency by reducing voltage and "dimming" lamps to conserve power during periods of low UV demand. Lamp intensity cannot be reduced to levels below 50%, however, without causing the lamps to flicker and eventually turn off.

Factors affecting deterioration of performance and aging of UV lamps include electrode failure; mercury plating or blackening in the lamp's glass tube; and tube solarization, which results in reduced energy transmission through the glass. These factors can reduce lamp output by 40–60%.

UV lamps used for disinfection are typically hot cathode lamps, which deteriorate progressively with each startup. Life expectancy is determined by the number of times the lamp is started. The lamp life cited by most manufacturers is 7,500 h, based on a burning cycle of 8 h; that is, the lamp will last 7,500 h assuming it is restarted every 8 h. The average UV output after 7,500 h is estimated to be 70% of the lamp output at 100 h.

Lamp intensity should be measured to monitor lamp condition and determine the need for maintenance. The monitoring procedure compares current lamp intensity to the intensity of new lamps. The operator first measures the intensity, at a fixed distance, of three to five new lamps that have burned for about 100 h (the first 100 h is considered a "burn-in" period). The average of these values is the benchmark from which to measure the relative output of the lamps. Each lamp is tagged so that it can be monitored individually.

A similar procedure is used to monitor the transmittance of a quartz sleeve. First, the intensity of a single lamp is measured with and without a new, clean quartz sleeve. Similar measurements are taken of the quartz unit in use and compared to the transmittance of the new quartz. Before being tested, the quartz is cleaned to assure that maximum transmittance is restored.

18.21.9 Operation and Maintenance of the Reactor

The most important operating factor for the UV reactor is the cleanliness of the surfaces through which radiation must pass. Surface fouling can result in inadequate performance, so a strict maintenance schedule is recommended.

An operator determines the need for reactor cleansing by draining and visually inspecting the surfaces. Open reactor systems are easily inspected. Systems with sealed vessels are inspected through portholes or manways in the reactor shell. Surfaces of submerged quartz systems become coated with an inorganic scale, very much like boiler scale. This is a particular problem in areas with hard water. Additionally, the inside surface of the quartz and the outer surfaces of the Teflon tubes eventually develop a grimy dust layer, primarily from airborne dirt and water vapor.

Fouling of the reactor's internal surfaces also is indicated by reduced performance and intensity measured by in-line probes. While these provide some indication of fouling, operators must still visually inspect the surfaces.

The fouled surfaces of lamps and quartz sleeves are cleaned manually with a mild soap solution and then swabbed with a rag soaked in isopropyl alcohol. The transmittance of the lamps and sleeves is measured after cleaning and those that have inadequate measurements are replaced. An inventory allows the plant operator to trace operation of individual components. Quartz sleeves should last between 4 and 7 years, but this varies by site.

In Teflon systems, the lamps are on removable racks and should be cleaned and monitored in the same manner as the quartz systems. The Teflon tubes should also be cleaned with mild soap and swabbed with alcohol. Each tube should be monitored for transmittance, just as with the quartz sleeves. Monitoring may not be as straightforward because of the limited accessibility to the tubes and problems in obtaining direct measurements with a UV radiometer/detector.

EXAMPLE 18.12 ULTRAVIOLET RADIATION FOR PRIMARY DISINFECTION AT FORT BENTON, MT

The City of Fort Benton, MT, obtains drinking water from the Missouri River. The then current filtration plant (40 years old) was in need of upgrading. This example illustrates an engineering embarked by the city using two new technologies: (a) natural river bank filtration process and (b) UV technology.

Solution:

Rather than building a new filtration plant, the city built a new 0.088 m^3/s (2 MGD) treatment facility for upgrading its water supply system in 1987.

a. Natural river bank filtration process:

Water is drawn through Ranney collectors installed 6–7.5 m (20–25 ft) below the river bed, a system that allows the river bed to naturally filter the raw water. Turbidities of water entering the treatment plant average 0.08 NTU. No *Giardia* cysts have been found in the water from the Ranney collectors.

b. Primary disinfection using UV and secondary disinfection using chlorination

The water is treated with UV radiation for primary disinfection, and then chlorinated for secondary disinfection. An applied chlorine dosage of only about 1 mg/L is necessary. The entire water treatment system is housed in a 2.97 m^2 (32 ft^2) building.

The primary UV disinfection system consists of six irradiation chambers, two control cabinets with alarms, chart recorders, relays, hour-run meters, lamp and power on-lights, six thermostats, electrical door interlocks, mimic diagrams, and six UV intensity monitors measuring total UV output. Each irradiation chamber contains one 2.5 kW mercury-vapor, medium-pressure arc tube generating UV radiation at 253.7 nm.

The initial UV dosage is 41 mW-s/cm^2 at maximum water flow of 104 L/s (1,650 gpm) through each irradiation unit. Expected UV lamp life is 4,500 operating hours, providing a minimum UV dosage of 25 mW-s/cm^2. These conditions are designed to reduce concentrations of *E. coli* organisms by a minimum of 5 logs (10^5 reduction).

The UV system is equipped with a telemetry control system and fully automated backup system. Each bank of three irradiation chambers has two units on line at all times, with the third unit serving as backup. In the event that the UV intensity drops below acceptable limits (20 mW-s/cm^2) in any of the chambers, the automatic butterfly valve will close, stopping flow through the chamber; at that time, the automatic butterfly valve on the standby unit will open. The alarm system also is activated if UV intensity drops below acceptable limits in any of the chambers. The UV alarm system is interfaced with the automatic dialer and alarm system.

In 1987, total equipment costs for the six-unit UV irradiation system with butterfly valves was only USD 74,587. The equivalent cost in 2014 is about USD 141,000.

The latest Draft Guidance Manual of the US Environmental Protection Agency (US EPA) for compliance with disinfection requirements contains "*Ct* values" for inactivation of viruses by UV radiation independent of temperature:

Log virus inactivation = 2.0 when Ct value of UV = 21 (mW-s/cm^2)

Log virus inactivation = 3.0 when Ct value of UV = 36 (mW-s/cm^2)

For the UV facility at Ft. Benton, the initial UV dosage of 41 mW-s/cm^2 provides well in excess of 3-log inactivation of viruses. However, after 4,500 hours of UV lamp operation, the anticipated decrease in UV dosage (to 25 mW-s/cm^2) will provide only 2 logs of viral inactivation.

EXAMPLE 18.13 DISINFECTANT COMPARISON AND POINTS OF APPLICATION

Briefly discuss various disinfectants in terms of their biocidal effectiveness, disinfection by-product (DBPs) production, and point of applications in water and wastewater treatment plants.

Solution:

a. Biocidal effectiveness versus DBP production Tables 18.2, 18.3, and 18.4 present an overview of biocidal effectiveness of common disinfectants (chlorine, chlorine dioxide, monochloramine, ozone, UV, and AOP). One of the most important considerations in assessing disinfectants is balancing inactivation or biocidal effectiveness with by-product production. The by-products of greatest current concern are trihalomethanes and other halogenated organic compounds; chlorine has the greatest potential for generating harmful by-products. The amount of these by-products produced by chlorine is affected by

(1) Chlorine dosage
(2) Types and concentrations of organic material in the influent
(3) Influent temperature
(4) Influent pH
(5) Contact time for free chlorine
(6) Nature of residual (free chlorine vs. combined chlorine)
(7) Presence of bromide ion

If chlorine produces an unsatisfactory level of by-products, then other disinfectants are potential alternatives. Chlorine dioxide is effective, but the total levels of chlorine dioxide and its oxidation/reduction products may limit its applicability. Ozone and UV radiation are very effective primary disinfectants, but require the use of secondary disinfectants. Ozone will produce harmful halogenated by-products with influent containing bromide ions. It will also produce harmful oxidation products in the presence of certain synthetic organics such as heptachlor. Chlorine dioxide and monochloramines are effective secondary disinfectants, but require careful dosage and application management to avoid producing harmful by-products in finished water. There are three strategies for controlling DBPs:

(1) Removal of DBPs after they are formed—it is very difficult and expensive.
(2) Adoption of alternate disinfectants that do not produce undesirable DBPs—UV, ozone, and so on are alternate disinfectants.
(3) Reduction of the concentration of organics in the water or wastewater before oxidation or chlorination to minimize the formation of DBPs—selection of appropriate disinfectant application points.

b. Points of disinfectant application in water treatment plants

Table 18.15 introduces the points of various disinfectant applications in a conventional water treatment plant consisting of raw water intake, rapid mixing, flocculation, sedimentation, filtration, and GAC.

Table 18.15 Desired points of disinfectant application

Chlorine	Toward the end of the water treatment process to minimize THM formation and provide secondary disinfection
Ozone	Prior to the rapid mixing step in all treatment processes, except GAC and conventional treatment processes; prior to filtration for GAC; post-sedimentation for conventional treatment. In addition, sufficient time for biodegradation of the oxidation products of the ozonation of organic compounds is recommended prior to secondary disinfection
Ultraviolet radiation	Toward the end of the water treatment process to minimize presence of other contaminants that interfere with this disinfectant. UV is also suitable for primary disinfection
Chlorine dioxide	Prior to filtration; to assure low levels of ClO_2, ClO_2^-, and ClO_3, treat with GAC after disinfection
Monochloramines	Best applied towards the end of the process as a secondary disinfectant

Source: US EPA.

c. Points of disinfectant application in wastewater treatment plants

A typical wastewater treatment plant consists of preliminary screening, grit chamber, primary clarification (primary sedimentation or flotation), biological oxidation (activated sludge, trickling filter, rotating biological contactors, etc.), secondary clarification (secondary sedimentation or flotation), tertiary treatment (nitrification, denitrification, advanced treatment, etc.), disinfection, and dechlorination.

Therefore, disinfection is best applied toward the end of the wastewater treatment train where most of organic substances (in terms of COD, BOD, and TOC) or trihalomethane formation potential (THMPF) have been removed. The most common disinfection process used in wastewater treatment plants is chlorination. A final dechlorination process is used when chlorine residual is too high. Either chemical reducing agent (such as sulfur dioxide) or UV can be used for dechlorination. In case UV is used for final disinfection of wastewater, no additional dechlorination step is needed.

EXAMPLE 18.14 COMBINED FILTRATION AND DISINFECTION

The US SWTR requires all surface water systems and all systems using groundwater under the influence of surface water to achieve 3-log (99.9%) removal of *Giardia* and 4-log (99.99%) removal or inactivation of viruses. This level of treatment can be accomplished by disinfection alone if the source water is relatively free of turbidity and the system meets other specific conditions. Or, the treatment requirements can be met by a combination of filtration and disinfection. Tables 18.2, 18.16, and 18.17 introduce the technical capabilities of conventional filtration, direct filtration, slow sand filtration, package filtration plant, diatomaceous earth filtration, membrane filtration, and cartridge filtration for removal of microbial contamination. Table 18.18 presents the treatment removal credits and required disinfection efficiencies of conventional filtration, direct filtration, slow sand filtration, and diatomaceous earth filtration.

Table 18.16 Influent characteristics and capacities for filtration technologies

Filtration options	Turbidity (NTUs)	Color (in color units)	Coliform count (per 100 mL)	Typical capacity (MGD)
Conventional	No restrictions	<75	<20,000	>All Sizes
Direct	<14	<40	<500	>All Sizes
Slow sand	<5	<10	<800	<15
Package Plant		[Depends on processes utilized]		<6
Diatomaceous earth	<5	<5	<50	<100
Membrane	< 1	[Fouling index of <10]		<0.5
Cartridge	<2	NA	NA	<1.0

Source: US EPA.
NA = not available.
Conversion factor: 1 MGD = 3.785 MLD.

Table 18.17 Removal capacities of seven filter options (% removal)

Filtration options	Achievable *Giardia* cyst levels	Achievable virus levels
Conventional	99.9	99.0
Direct	99.9	99.0
Slow sand	99.99	99.9999
Package plant varies with manufacturer		
Diatomaceous earth	99.99	>99.95
Membrane	100	Very low
Cartridge	>99	Little data available

Source: US EPA.

Table 18.18 Removal credits and disinfection efficiencies for filtration

	Treatment removal credit (Log removals)		Required disinfection (Log inactivations[a])	
Type of filtration	*Giardia*	Viruses	*Giardia*	Viruses
Conventional	2.5	2.0	0.5	2.0
Direct	2.0	1.0	1.0	3.0
Slow sand	2.0	2.0	1.0	2.0
Diatomaceous earth	2.0	1.0	1.0	3.0

Source: US EPA.

[a] Assumed 3-log *Giardia* and 4-log virus removal and/or inactivation (can be increased based on sanitary hazards in watershed).

When both filtration and disinfection are used, it is first necessary to know the effectiveness of filtration in removing *Giardia* and viruses before it will be possible to determine the level of disinfection needed to reach the 3-log and 4-log treatment levels, as required by the US Environmental Protection Agency (US EPA). Determine the *Giardia*'s log removal efficiency of disinfection process for a water treatment plant using conventional filtration.

Solution:

According to Table 18.18, a well-operated conventional filtration system should be able to achieve 2.5-log removal of *Giardia* and 2.0-log removal/inactivation of viruses. Since conventional filtration achieves 2.5-log removal of *Giardia*, disinfection processes will need to achieve at least 0.5-log *Giardia* removal to meet the SWTR requirement of 3-log *Giardia* cysts removal.

EXAMPLE 18.15 DISINFECTION OF RECYCLED WASTEWATER FROM WATER TREATMENT PLANT

Disinfection can be a barrier to the recycling of pathogens from recycle streams in water treatment plants. The California Department of Health Services recommends that disinfection be applied for recycle streams. The main issues to be addressed when considering disinfection of recycle streams are

- **a.** The level of inactivation to be provided for specific organisms
- **b.** Whether disinfection is to be used alone or with a solids removal process
- **c.** The potential impacts of recycle stream disinfection on finished water quality, particularly the formation of DBPs

The most common disinfectant used by water treatment plants in the United States for disinfecting their recycle streams has been chlorine. Discuss (a) the advantages and disadvantages of chlorination process for disinfecting recycle streams and (b) the alternate disinfection processes which may eliminate or reduce the disadvantages.

Solution:

a. Discussion of advantages:
Pathogens are contaminants of concern in recycle streams. Depending on the type and amount of disinfectant used, *Cryptosporidium*, *Giardia*, and/or viruses can be inactivated. More advantages may be realized through disinfection of recycle streams as more studies are conducted on this practice.

b. Discussion of disadvantages:
Recycle stream disinfection should be examined for its potential effects on the main treatment train and finished water quality. Untreated recycle streams can have significant concentrations of TTHM precursors and TOC. If the recycle stream is treated with chlorine, then recycling may cause problems for the treatment plant in meeting DBP limits. The potential formation of DBPs through disinfection should be considered.

c. Alternate disinfection processes for treating recycle streams:

The oxidant demand of both potassium permanganate and chlorine dioxide was used for spent filter backwash samples from five participating water utilities. Overall, the potassium permanganate demands were approximately 5.5 times higher for spent filter backwash with particles than in samples without particles. Potassium permanganate disinfection at 2,400 mg/L-min (*Ct* value) with and without particles resulted in *Cryptosporidium* inactivations of 0.21 and 0.27 log, respectively. The presence of particles in spent filter backwash increased the chlorine dioxide demand by a factor of 4 when compared to samples without particles. Chlorine dioxide dosed at 115 mg/L-min (*Ct* value) produced 2.7- and 2.1-log inactivation of *Cryptosporidium* for spent filter backwash with and without particles, respectively. UV treatment was also examined for its effectiveness on *Cryptosporidium* in clarified spent filter backwash with turbidities between 10 and 14 NTU. UV doses as low as 3 mJ/cm^2 that were used in collimated beam experiments resulted in *Cryptosporidium* inactivation greater than 4 logs.

18.22 RECENT DEVELOPMENTS IN DISINFECTION MANAGEMENT—LOG REMOVAL/INACTIVATION CREDITS OF DRINKING WATER TREATMENT PROCESSES

18.22.1 Introduction

This section provides guidance for the assignment of *Cryptosporidium*, *Giardia*, and virus log removal/inactivation credits for various types of drinking water treatment processes. This technical information is primarily applicable to all community and noncommunity public water systems using surface water sources or *ground water sources directly influenced by surface water* (GWUDI). The virus inactivation tables included in this section are applied to water supply systems impacted by the US EPA *Ground Water Rule* (GWR).

It has been defined previously that *Ct* is the product of the chemical disinfectant residual concentration, *C*, in the water in mg/L at or before the first user, and the contact time, *t*, in min, that the water is in contact with the disinfectant. To be consistent with the US EPA tables, *T* is here being used to designate contact time:

$$CT = (C)(T)$$

The term "*removal*" used in disinfection management is the physical removal of microorganisms (such as of *Cryptosporidium*, *Giardia*, bacteria, and virus) usually through filtration.

The term "*inactivation*" used in disinfection management is the reproductive sterilization or the destruction of microorganisms through disinfection.

18.22.2 Surface Water Treatment Rule and Long Term 2 Enhanced Surface Water Treatment Rule

In the United States, the original SWTR that went into effect in 1993 required all public water supply systems using surface water or GWUDI sources to provide a 3-log or 99.9% *Giardia* removal/inactivation and a 4-log enteric virus removal/inactivation. US EPA recently promulgated the *Long Term 2 Enhanced Surface Water Treatment Rule* (LT2ESWTR). This LT2ESWTR requires all public water systems with surface water sources or GWUDI sources to provide treatment (filtration and/or disinfection) to adequately remove and/or inactivate various pathogenic organisms depending on the source water quality. Specifically, additional treatment may be required consisting of a 99.9% through 99.9995% removal/inactivation of *Cryptosporidium*. This is also defined as a 3-log through a 5.5-log removal/inactivation of *Cryptosporidium*. The additional level of treatment required under LT2ESWTR is dependent on the amount of *Cryptosporidium* in the source water and those requirements are detailed in the LT2ESWTR bin classification shown in Table 18.19. In addition to the *Cryptosporidium* treatment under LT2ESWTR, a 99.99% or 4-log removal/inactivation of enteric viruses continues to be required as it was under the original SWTR. Tables 18.20, 18.21, 18.22, 18.23, 18.24, and 18.25 show the efficiencies of various chemical (chlorine, chloramines, ozone, chlorine dioxide), physical (UV, filtration, membrane filtration), and natural (bank filtration) disinfection processes.

In the LT2ESWTR the target organism for removal and/or inactivation is *Cryptosporidium*. It is resistant to chemical disinfectants, remains viable in raw water for

Table 18.19 Long Term 2 Enhanced Surface Water Treatment Rule bin classifications

If source water *Cryptosporidium* concentration for filtered systems is in oocyst/L	And the system uses the following filtration treament, then additional treament requirements are…				
	Bin classification is…	Conventional filtration treatment	Direct filtration	Slow sand or diatomaceous earth	Alternative filtration technologies
<0.075	1	No additional treatment	No additional treatment	No additional treatment	No additional treatment
≥0.075 and <1.0	2	1.0-log treatment	1.5-log treatment	1.0-log treatment	At least 4.0 log
≥1.0 and <3.0	3	2.0-log treatment	2.5-log treatment	2.0-log treatment	At least 5.0 log
≥3.0	4	2.5-log treatment	3.0-log treatment	2.5-log treatment	At least 5.5 log

Source: US EPA.

Table 18.20 Virus inactivation using chemical disinfectants

CT values for inactivation of viruses by free chlorine, pH 6–9						
Log inactivation	1°C	5°C	10°C	15°C	20°C	25°C
2	5.8	4.0	3.0	2.0	1.0	1.0
3	8.7	6.0	4.0	3.0	2.0	1.0
3.5	10.2	7.0	5.0	3.5	2.5	1.5
4	11.6	8.0	6.0	4.0	3.0	2.0
CT values for inactivation of viruses by chloramine, pH 6–9						
Log inactivation	1°C	5°C	10°C	15°C	20°C	25°C
2	1,243	857	643	428	321	214
3	2,036	1,423	1,067	712	534	356
4	2,883	1,988	1,491	994	746	497
CT values for inactivation of viruses by chlorine dioxide, pH 6–9						
Log inactivation	1°C	5°C	10°C	15°C	20°C	25°C
2	8.4	5.6	4.2	2.8	2.1	1.4
3	25.6	17.1	12.8	8.6	6.4	4.3
4	50.1	33.4	25.1	16.7	12.5	8.4
CT values for inactivation of viruses by ozone, pH 6–9						
Log inactivation	1°C	5°C	10°C	15°C	20°C	25°C
2	0.9	0.6	0.5	0.3	0.25	0.15
3	1.4	0.9	0.8	0.5	0.40	0.25
4	1.8	1.2	1.0	0.6	0.50	0.30

Source: US EPA.

Table 18.21 *Cryptosporidium*, *Giardia*, and virus log inactivation values for UV

Log credit	*Cryptosporidium* dose (mJ/cm^2)	*Giardia* dose (mJ/cm^2)	Virus dose (mJ/cm^2)
0.5	1.6	1.5	39
1.0	2.5	2.1	58
1.5	3.9	3.0	79
2.0	5.8	5.2	100
2.5	8.5	7.7	121
3.0	12	11	143
3.5	15	15	163
4.0	22	22	186

Source: US EPA.

Table 18.22 *CT* values ($CT_{99.9}$) for 99.9% *Giardia* inactivation using chlorine dioxide and ozone as disinfectants[a]

Disinfectant	Temperature, °C					
	<= 1	5	10	15	20	>= 25
Chlorine dioxide	63	26	23	19	15	11
Ozone	2.9	1.9	1.4	0.95	0.72	0.48

Source: US EPA.

[a] These CT values achieve greater than a 99.99% inactivation of viruses. CT values between the indicated pH values may be determined by linear interpolation. If no interpolation is used, use the $CT_{99.9}$ value at the lower temperature for determining $CT_{99.9}$ values between indicated temperatures.

extended periods of time, and has been identified as a leading cause of waterborne disease outbreaks in the United States in recent decades. Since *Cryptosporidium* is smaller than *Giardia* and more resistant to chemical disinfectants than *Giardia* it is assumed that if the required *Cryptosporidium* level of treatment is achieved then the required *Giardia* level of treatment is also achieved. *Giardia*, at 5–15 microns is physically larger than *Cryptosporidium* with a size of approximately 3–5 microns and *Giardia* can be inactivated with chemical disinfectants. Under the LT2ESWTR, in addition to the *Cryptosporidium* treatment requirement, viruses are also required to be removed and/or inactivated to a level of 99.99% or a 4-log level.

Table 18.21 shows that ultraviolet light (UV) disinfection is effective for virus inactivation at very high dose and is effective for *Cryptosporidium* and *Giardia* inactivation at moderate doses. In the mid 1990s research studies showed that although *Cryptosporidium* remained alive in UV-treated water it is damaged at the DNA level and cannot replicate and therefore cannot cause illness. Further studies substantiated this initial work and other studies showed that *Giardia* is also inactivated. In 2003, the State of New York became the first state in the United States to accept UV disinfection as primary disinfection for surface and GWUDI sources of water. Ultimately the entire United States and other countries followed in allowing UV disinfection as primary disinfection for surface and GWUDI sources.

Table 18.23 Baffling factors

Baffling condition	*T10/T*	Baffling description
Unbaffled	0.1	None, agitated basin, very low length to width ratio, high inlet and outlet flow velocities. Can be approximately achieved in flash mix tank
Poor	0.3	Single or multiple unbaffled inlets and outlets, no intra-basin baffles
Average	0.5	Baffled inlet or outlet with some intra-basin baffles
Superior	0.7	Perforated inlet baffle, serpentine or perforated intra-basin baffles, outlet weir or perforated launders
Perfect	1.0	Very high length to width ratio (pipeline flow), perforated inlet, outlet, and intra-basin baffles

Source: US EPA.

*T*10 = Detention time (min) of segment * Baffling fact.

When using any chemical disinfectant for *Cryptosporidium*, *Giardia*, and virus treatment (see Tables 18.20, 18.22, and 18.24), baffle factors must be considered when determining available volume for contact time and *CT* calculations (see Table 18.23). In addition, removal efficiencies must be proven either on site or by manufacturer's data for alternative filtration technologies that is, bags, cartridges, and membranes. UV units must be validated for dose in accordance with the US EPA, the Austrian, or the German Standards.

Polychromatic UV light has a broader spectrum of disinfection capability. Research results indicate that a 4-log inactivation of viruses may occur at approximately 100 mJ/cm^2. It is uncertain if or when the US EPA may accept a lower dose than 186 mJ/cm^2. Even though 100 mJ/cm^2 is an achievable UV dose the normal required design and delivered dose for UV in New York State is 40 mJ/cm^2 and most units are validated for that dose. Based on Table 18.21, a dose of 40 mJ/cm^2 may be credited with a 0.5-log inactivation of viruses leaving a 3.5-log inactivation to be provided by other treatment. In consideration of these changes when ultraviolet light disinfection (UV) is designed for use as the primary disinfectant, a chemical disinfectant may also be required due to the resistance of Adenovirus to UV. The State of New York has always required a residual disinfectant such as chlorine to be used in addition to UV when a distribution system is for multiple buildings or service laterals. Single-service systems such as restaurants, convenience stores, or any other similar system with a single building have been approved in the past without a residual disinfectant.

The LT2ESWTR also requires that systems sample source water for *Cryptosporidium* or *E. coli* depending on the system size to determine the appropriate bin classification (Table 18.19) and subsequent additional treatment that may be required above and beyond that required by the original SWTR.

Systems may avoid *Cryptosporidium* or *E. coli* sampling required under the LT2ESWTR if they provide or intend to provide treatment meeting a minimum of 5.5-log removal/inactivation of *Cryptosporidium*.

One example for avoiding the need to sample under the LT2ESWTR would be a surface source or GWUDI source providing treatment with conventional filtration (credited up to 2.5-log removal of *Cryptosporidium*) plus UV disinfection (credited up to 4-log inactivation of *Cryptosporidium*). In this case they could be credited with the full 5.5-log removal/inactivation for *Cryptosporidium*. Even though the system could be theoretically credited with 6.5-log

Table 18.24 $CT_{99.9}$ values in min-mg/L for 3-log inactivation of *Giardia* cysts by free chlorine

	Temperature = 0.5°C							Temperature = 5°C							Temperature = 10°C						
	pH							pH							pH						
Chlorine concentration (mg/L)	<= 6.0	6.5	7.0	7.5	8.0	8.5	9.0	<= 6.0	6.5	7.0	7.5	8.0	8.5	9.0	<= 6.0	6.5	7.0	7.5	8.0	8.5	9.0
<= 0.4	137	163	195	237	277	329	390	97	117	139	166	198	236	279	73	88	104	125	149	177	209
0.6	141	168	200	239	286	342	407	100	120	143	171	204	244	291	75	90	107	126	153	183	218
0.8	145	172	205	246	295	354	422	103	122	146	175	210	252	301	78	92	110	131	158	189	226
1.0	148	176	210	253	304	365	437	105	125	149	179	216	260	312	79	94	112	134	162	195	234
1.2	152	180	215	259	313	376	451	107	127	152	183	221	267	320	80	95	114	137	160	200	240
1.4	155	184	221	266	321	387	464	109	130	155	187	227	274	329	82	98	116	140	170	206	247
1.6	157	189	226	273	329	397	477	111	132	158	192	232	281	337	83	99	119	144	174	211	253
1.8	162	193	231	279	338	407	489	114	135	162	196	236	287	345	86	101	122	147	179	215	259
2.0	165	197	236	286	346	417	500	116	138	165	200	243	294	353	87	104	124	150	182	221	265
2.2	169	201	242	297	353	426	511	118	140	169	204	248	300	361	89	105	127	153	186	225	271
2.4	172	205	247	298	361	435	522	120	143	172	209	253	306	368	90	107	129	157	190	230	276
2.6	175	209	252	304	368	444	533	122	146	175	213	258	312	375	92	110	131	160	194	234	251
2.8	178	213	257	310	375	452	543	124	148	178	217	263	318	382	93	111	134	163	197	239	287
3.0	181	217	261	316	382	460	552	126	151	182	221	268	324	389	95	113	137	166	201	243	292

	Temperature = 15°C							Temperature = 20°C							Temperature = 25°C						
	pH							pH							pH						
Chlorine concentration (mg/L)	<= 6.0	6.5	7.0	7.5	8.0	8.5	9.0	<= 6.0	6.5	7.0	7.5	8.0	8.5	9.0	<= 6.0	6.5	7.0	7.5	8.0	8.5	9.0
<= 0.4	49	59	70	83	99	118	140	36	44	52	62	74	89	105	24	29	35	42	50	59	70
0.6	50	60	72	86	102	122	146	38	45	54	64	77	92	109	25	30	36	43	51	61	73
0.8	52	61	73	88	105	126	151	39	46	55	66	79	95	113	26	31	37	44	53	63	75
1.0	53	63	75	90	108	130	156	39	47	56	67	81	98	117	26	31	37	45	54	65	76
1.2	54	64	76	92	111	134	160	40	48	57	69	83	100	120	27	32	38	46	55	67	80
1.4	55	65	78	94	114	137	165	41	49	58	70	85	103	123	27	33	39	47	57	69	82
1.6	56	66	79	96	116	141	169	42	50	59	72	87	105	126	28	33	40	48	58	70	84
1.8	57	68	81	98	119	144	173	43	51	61	74	89	108	129	29	34	41	49	60	72	86
2.0	58	69	83	100	122	147	177	44	52	62	76	91	110	132	29	35	41	50	61	74	88
2.2	59	70	85	102	124	150	181	44	53	63	77	93	113	135	30	35	42	51	62	75	90
2.4	60	72	86	105	127	153	184	45	54	65	78	95	115	136	30	36	43	52	63	77	92
2.6	61	73	88	107	129	156	188	46	55	66	80	97	117	141	31	37	44	53	65	78	94
2.8	62	74	89	109	132	159	191	47	56	67	81	99	119	143	31	37	45	54	66	80	96
3.0	63	76	91	111	134	162	195	47	57	68	83	101	122	146	32	38	46	55	67	81	97

Source: US EPA.

Note: If no interpolation is used, use the $CT_{99.9}$ value at the lower temperature, and at the higher pH.

Table 18.25 *CT* values for inactivation of *Giardia* cysts by free chlorine at 10°C inactivation

Chlorine concentration	pH <= 6 Log inactivations						pH = 6.5 Log inactivations						pH = 7.0 Log inactivations						pH = 7.5 Log inactivations					
(mg/L)	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	12	24	37	49	61	73	15	29	44	59	73	88	17	35	52	69	87	104	21	42	63	83	104	125
0.6	13	25	38	50	63	75	15	30	45	60	75	90	18	36	54	71	89	107	21	43	64	85	107	128
0.8	13	26	39	52	65	78	15	31	46	61	77	92	18	37	55	73	92	110	22	44	66	87	109	131
1	13	26	40	53	66	79	16	31	47	63	78	94	19	37	56	75	93	112	22	45	67	89	112	134
1.2	13	27	40	53	67	80	16	32	48	63	79	95	19	38	57	76	95	114	23	46	69	91	114	137
1.4	14	27	41	55	68	82	16	33	49	65	82	98	19	39	58	77	97	116	23	47	70	93	117	140
1.6	14	28	42	55	69	83	17	33	50	66	83	99	20	40	60	79	99	119	24	48	72	96	120	144
1.8	14	29	43	57	72	86	17	34	51	67	84	101	20	41	61	81	102	122	25	49	74	98	123	147
2	15	29	44	58	73	87	17	35	52	69	87	104	21	41	62	83	103	124	25	50	75	100	125	150
2.2	15	30	45	59	74	89	18	35	53	70	88	105	21	42	64	85	106	127	26	51	77	102	128	153
2.4	15	30	45	60	75	90	18	36	54	71	89	107	22	43	65	86	108	129	26	52	79	105	131	157
2.6	15	31	46	61	77	92	18	37	55	73	92	110	22	44	66	87	109	131	27	53	80	107	133	160
2.8	16	31	47	62	78	93	19	37	56	74	93	111	22	45	67	89	112	134	27	54	82	109	136	163
3	16	32	48	63	79	95	19	38	57	75	94	113	23	46	69	91	114	137	28	55	83	111	138	166

Chlorine concentration	pH = 8.0 Log inactivations						pH = 8.5 Log inactivations						pH <= 9.0 Log inactivations					
(mg/L)	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
<= 0 4	25	50	75	99	124	149	30	59	89	118	148	177	35	70	105	139	174	209
0.6	26	51	77	102	128	153	31	61	92	122	153	183	36	73	109	145	182	218
0.8	26	53	79	105	132	158	32	63	95	126	158	189	38	75	113	151	188	226
1	27	54	81	108	135	162	33	65	98	130	163	195	39	78	117	156	195	234
1.2	28	55	83	111	138	166	33	67	100	133	167	200	40	80	120	160	200	240
1.4	28	57	85	113	142	170	34	69	103	137	172	206	41	82	124	165	206	247
1.6	29	58	87	116	145	174	35	70	106	141	176	211	42	84	127	169	211	253
1.8	30	60	90	119	149	179	36	72	108	143	179	215	43	86	130	173	216	259
2	30	61	91	121	152	182	37	74	111	147	184	221	44	88	133	177	221	265
2.2	31	62	93	124	155	186	38	75	113	150	188	225	45	90	136	181	226	271
2.4	32	63	95	127	158	190	38	77	115	153	192	230	46	92	138	184	230	276
2.6	32	65	97	129	162	194	39	78	117	156	195	234	47	94	141	187	234	281
2.8	33	66	99	131	164	197	40	80	120	159	199	239	48	96	144	191	239	287
3	34	67	101	134	168	201	41	81	122	162	203	243	49	97	146	195	243	292

Source: US EPA.

Notes: $CT_{99.9}$ = CT for 3-log inactivation; $CT_{99.0}$ = CT for 2-log inactivation; $CT_{90.0}$ = CT for 1-log inactivation.

removal/inactivation of *Cryptosporidium* it has been the policy of the State of New York to only provide log credit to systems up to what they are required to have when they exceed that required level.

Another example is membrane filtration with a pore size small enough to provide nanofiltration (NF). This may be allowed by the US EPA as standalone treatment for *Cryptosporidium*, *Giardia*, and viruses, as shown in Table 18.26.

Table 18.26 Microbial Toolbox summary table for options, water treatment credits, and criteria

Toolbox option	*Cryptosporidium treatment credit with design and implementation criteria*
Source Protection and Management Toolbox Options	
(1) Watershed control program	0.5-log credit for state-approved program comprising required elements, annual program status report to state, and regular watershed survey. Unfiltered systems are not eligible for credit.
(2) Alternative source/intake management	No prescribed credit. Systems may conduct simultaneous monitoring for treatment bin classification at alternative intake locations or under alternative intake management strategies.
Pre-filtration Toolbox Options	
(3) Presedimentation basin with coagulation	0.5-log credit during any month that presedimentation basins achieve a monthly mean reduction of 0.5 log or greater in turbidity or alternative state-approved performance criteria. To be eligible, basins must be operated continuously with coagulant addition and all plant flow must pass through basins.
(4) Two-stage lime softening	0.5-log credit for two-stage softening where chemical addition and hardness precipitation occur in both stages. All plant flow must pass through both stages. Single-stage softening is credited as equivalent to conventional treatment.
(5) Bank filtration	0.5-log credit for 25 foot (7.62 m) setback; 1.0-log credit for 50 foot (15.24 m) setback; aquifer must be unconsolidated sand containing at least 10% fines; average turbidity in wells must be less than 1 NTU. Systems using wells followed by filtration when conducting source water monitoring must sample the well to determine bin classification and are not eligible for additional credit.
Treatment Performance Toolbox Options	
(6) Combined filter performance	0.5-log credit for combined filter effluent turbidity less than or equal to 0.15 NTU in at least 95% of measurements each month.
(7) Individual filter performance	0.5-log credit (in addition to 0.5-log combined filler performance credit) if individual filter effluent turbidity is less than or equal to 0.15 NTU in at least 95% of samples each month in each filter and is never greater than 0.3 NTU in two consecutive measurements in any filter.
(8) Demonstration of performance	Credit awarded to unit process or treatment train based on a demonstration to the state with a state-approved protocol.
Additional Filtration Toolbox Options	
(9) Bag or cartridge filters (individual filters)	Up to 2-log credit based on the removal efficiency demonstrated during challenge testing with a 1.0-log factor of safety.
(10) Bag or cartridge filters (in series)	Up to 2.5-log credit based on the removal efficiency demonstrated during challenge testing with a 0.5-log factor of safety.
(11) Membrane filtration	Log credit equivalent to removal efficiency demonstrated in challenge test for device if supported by direct integrity testing.
(12) Second stage filtration	0.5-log credit for second separate granular media filtration stage if treatment train includes coagulation prior to first filter.
(13) Slow sand filters	2.5-log credit as a secondary filtration step; 3.0-log credit as a primary filtration process. No prior chlorination for either option.
Inactivation Toolbox Options	
(14) Chlorine dioxide	Log credit based on measured CT in relation to CT table.
(15) Ozone	Log credit based on measured CT in relation to CT table.
(16) UV	Log credit based on validated UV dose in relation to UV dose table; reactor validation testing required to establish UV dose and associated operating conditions.

Source: US EPA.

18.22.3 Ground Water Rule and Total Coliform Rule

Groundwater sources not influenced by surface water have historically been required by the State of New York to have 15 min of chlorine contact time when using chlorine for disinfection. In addition, UV treatment has been acceptable as far back as the 1980s. Between the 1980s and 2001 the required UV design dose was 16 mJ/cm^2 for groundwater sources. In 2001 the State of New York revised the required dose for all new UV installations regardless of the source water to a dose of 40 mJ/cm^2. As with surface and GWUDI sources, all groundwater sources on systems with a distribution system (more than one building) need to provide a disinfection residual in the distribution system. In October 2006, the US EPA GWR was promulgated and that rule may add additional treatment requirements for certain susceptible groundwater sources.

Specifically the US EPA promulgated the final GWR in October 2006 to reduce the risk of exposure to fecal contamination that may be present in public water systems that use groundwater sources. US EPA proposed the GWR on May 10, 2000. The rule establishes a risk-targeted strategy to identify groundwater systems that are at high risk for fecal contamination. The GWR also specifies when corrective action (which may include disinfection) is required to protect consumers who receive water from groundwater systems from bacteria and viruses.

The GWR in the United States addresses risks through a risk-targeting approach that relies on four major components:

1. Periodic sanitary surveys of ground water systems that require the evaluation of eight critical elements and the identification of significant deficiencies (e.g., a well located near a leaking septic system). States must complete the initial survey by December 31, 2012, for most *community water systems* (CWS) and by December 31, 2014, for CWS with outstanding performance and for all noncommunity water systems.
2. Source water monitoring to test for the presence of *E. coli*, *enterococci*, or *coliphage* in the sample. There are two monitoring provisions: (a) triggered monitoring for systems that do not already provide treatment that achieves at least 99.99% (4-log) inactivation or removal of viruses and that have a total coliform-positive routine sample under *Total Coliform Rule* (TCR) sampling in the distribution system; and (b) assessment monitoring—as a complement to triggered monitoring, a state has the option to require systems, at any time, to conduct source water assessment monitoring to help identify high-risk systems.
3. Corrective actions required for any system with a significant deficiency or source water fecal contamination. The system must implement one or more of the following correction action options: (a) correct all significant deficiencies, (b) eliminate the source of contamination, (c) provide an alternate source of water, or (d) provide treatment, which reliably achieves 99.99% (4-log) inactivation or removal of viruses.
4. Compliance monitoring to ensure that treatment technology installed to treat drinking water reliably achieves at least 99.99% (4-log) inactivation or removal of viruses.

With the promulgation of the US EPA's groundwater rule, virus inactivation has come to the forefront of groundwater source treatment design. The State of New York historically required groundwater sources that were not GWUDI be designed with a minimum of 15 minutes of contact time for chlorine disinfection. It is understood that in the past baffle factors may not have been considered for groundwater system chlorine contact time calculations and these existing systems will only need to be reevaluated if they have a trigger event under the groundwater rule. One reason that they need not be reevaluated is that the required *CT* values (See Table 18.20) at groundwater temperature would likely result in a contact time that is less than the 15 min provided. Another and more important reason is that they are not required to meet the 4-log virus inactivation unless a trigger event under the groundwater rule occurs.

EXAMPLE 18.16

At 10°C, Table 18.20 shows that a *CT* of 6 is required for a 4-log inactivation of viruses using chlorine. What will be the required chlorine contact time if the residual chlorine is 0.5 mg/L at 10°C? What will be the required chlorine contact time if the residual chlorine is 1 mg/L at 10°C?

Solution:

Assuming a residual of 0.5 mg/L and the *CT* of 6 from Table 18.20 would result in a required contact time of 12 min (before a baffling factor is applied).

Assuming a residual of 1 mg/L and the *CT* of 6 from Table 18.20 would result in a required contact time of 6 min (before a baffling factor is applied).

Therefore at either residual the historical 15 min should be an adequate disinfection time in the State of New York.

All new groundwater systems should be designed for a minimum of 4-log virus inactivation based on the *CT* values in Table 18.20 when using a chemical disinfectant including the consideration of baffling factors. The continued use of the historical 15 min of contact time in addition to baffling factor consideration for design is strongly recommended by the State of New York. Although the *CT* virus inactivation in Table 18.20 may allow us to accept designs with contact times less than 15 min and in certain cases it may be desirable to allow for less time, the 15 min provides an additional factor of safety. If a new properly designed, sited, constructed, and tested groundwater system wishes to use UV without a chemical disinfectant (assuming no distribution system) it should understand that if it were ever triggered into the groundwater rule treatment requirements then it may have to provide additional treatment to meet the 4-log inactivation of viruses.

In conclusion, any water supply systems using surface sources without filtration avoidance or GWUDI sources must provide a minimum of a 3-log removal/inactivation treatment of *Giardia* and a 4-log inactivation of viruses. These systems may also be required to provide additional treatment for *Cryptosporidium* depending on the bin classification they fall into or if they do not wish to sample under the LT2ESWTR. All new groundwater sources should be designed to provide a 4-log removal/inactivation of viruses but may not need to. If 4-log removal/inactivation treatment for viruses is not provided, the system may be required to provide it under the groundwater rule if they have a trigger event under that rule in the future.

When considering the design of water treatment on any public water supply system the appropriate tables in this section should be used to determine the minimum level of treatment depending on the source treatment requirements. Table 18.26 includes general Microbial Toolbox log assignments from LT2ESWTR guidance documents and these used along with Tables 18.19, 18.20, 18.21, 18.22, 18.23, 18.24, and 18.25 provide specific log assignments for various treatments.

PROBLEMS/QUESTONS

18.1 The following material balance equations are used for calculation of the concentrations and/or flows of the two solutions to be mixed together:

$$(Q1)(C1) + (Q2)(C2) = (Q\text{mix})(C\text{mix})$$

$$(Q1 + Q2) = Q\text{mix}$$

$$(Q1)(C1) + (Q2)(C2) = (Q1 + Q2)(C\text{mix})$$

where

$Q1$ = flow rate of the first solution, gpd, or gpm, or m^3/min
$Q2$ = flow rate of the second solution, gpd, or gpm, or m^3/min
$Q\text{mix}$ = flow rate of the mixed solution, gpd, or gpm, or m^3/min
$C1$ = concentration of the first solution, mg/L or %
$C2$ = concentration of the second solution, mg/L or %
$C\text{mix}$ = concentration of the mixed solution, mg/L or %

Estimate the desired strength (as % chlorine) of a hypochlorite solution being pumped by a hypochlorinator that delivers at a fixed rate of 80 gpd (302.8 L/d). 1.2 MG (4.54 ML) of water is being treated with 1 mg/L of chlorine dosage.

18.2 The following material balance equations should be used for calculation of the concentrations and/or volumes of the two solutions to be mixed together:

$$(V1)(C1) + (V2)(C2) = (V\text{mix})\,(C\text{mix})$$

$$(V1 + V2) = V\text{mix}$$

$$(V1)(C1) + (V2)(C2) = (V1 + V2)\,(C\text{mix})$$

where

$V1$ = volume of the first solution, gal, L, ft^3, or m^3
$V2$ = volume of the second solution, gal, L, ft^3, or m^3
$V\text{mix}$ = volume of the mixed solution, gal, L, ft^3, or m^3
$C1$ = concentration of the first solution, mg/L or %
$C2$ = concentration of the second solution, mg/L or %
$C\text{mix}$ = concentration of the mixed solution, mg/L or %

How many liters of water must be added to 10 L of a 5% sodium hypochlorite solution to produce a 1% hypochlorite solution?

18.3 How many lb/d (kg/d) of chlorine do you need to treat 4 MGD (15.1 ML/d) of water at a dosage of 2 mg/L?

18.4 The water chlorine demand is 12 mg/L, the target chlorine residual is 1.2 mg/L, and the plant flow is 5.6 MGD (21.2 ML/d). How many lb/d of 65% hypochlorite solution will be required?

18.5 Estimate the water flow (gpd and L/d) pumped by a sodium permanganate feeder if the sodium permanganate solution is in a container with a diameter of 2.5 ft (0.76 m) and the permanganate solution level drops 14 in. (0.36 m) during a 9-hour period. The permanganate feeder operated continuously during the 9-hour period.

18.6 Estimate the desired strength (as % chlorine) of a sodium hypochlorite solution which is pumped by a hypochlorinator that delivers 115 gpd (435.3 L/d). The water being treated requires a chlorine dose of 12 lb/d (5.45 kg/d) of chlorine.

18.7 A treatment plant uses 30 lb/week (13.62 kg/week) of gas chlorine from 150 lb cylinders (68.1 kg cylinders). How many 150 lb cylinders (68.1 kg cylinders) will be used per month (assume 30 days per month)?

18.8 How many gallons of 5% sodium hypochlorite will be needed to disinfect a well with a 16-inch (406.40 mm) diameter casing and well screen? The well is 180 feet (54.86 m) deep and there is 76 ft (23.16 m) of water in the well. Assume that an initial chlorine dose of 100 mg/L is to be used.

18.9 Chlorination is used for disinfection of water mains, water storage tanks, and so on before they are placed online for service. Normally continuous-feed chlorination method is preferred. Initial chlorine concentration, minimum contact time, minimum residual chlorine concentration, and coliform count of the disinfected water are 25 mg/L, 24 h, not less than 10 mg/L, and zero coliform count, respectively. If the minimum residual chlorine concentration of 10 mg/L and zero coliform count cannot be reached after 24 h of contact, the disinfection process must be repeated until the minimum residual chlorine concentration of 10 mg/L and zero coliform are both satisfied.

A new 12-inch (304.80 mm) diameter water main 700 ft (213.36 m) long needs to be disinfected. An initial chlorine dose of 25 mg/L is expected to maintain a chlorine residual of over 10 mg/L during the 24-h disinfection period. How many gallons of 5.25% sodium hypochlorite solution will be needed?

18.10 To meet the needs of situations requiring reduced disinfection contact times, the slug-feed chlorination method can be used for disinfection of water mains, water storage tanks, and other water treatment facilities. The slug-feed method allows only a 3-h contact time or higher, but requires a 100 mg/L initial chlorine dosage, not less than 50 mg/L residual chlorine concentration at any time, and zero coliform count after 3 h. Otherwise the disinfection procedure must be repeated.

A water storage tank has been taken out of service for inspection and repairs. The tank needs to be disinfected before being placed back on line. The reservoir is 50 ft (15.24 m) in diameter and 12 ft deep (3.66 m). An initial chlorine dose of 100 mg/L is expected to maintain a chlorine residual of over 50 mg/L during the 3-h disinfection period. How many gallons of 15% sodium hypochlorite solution will be needed?

18.11 Calculate the actual chlorine dosage in mg/L if 250 gallons (946 L) of a 2.5% sodium hypochlorite solution were used to treat 1.5 MG (5.68 ML) of water.

18.12 A deep-well turbine pump delivers 250 gpm (946 L/min) against normal operating heads. If the desired chlorine dose is 2 mg/L, what should be the chlorine feed rate in lb/day?

18.13 Environmental Rules and Regulations of federal, state (provincial), and local governments are different and changing from time to time. Assuming you are an environmental engineer in the United States, visit the web site of the US Environmental Protection Agency (www.epa.org) and report the current US EPA's (a) maximum contaminant level goals (MCLG) and maximum contaminant levels (MCL) for total trihalomethanes (TTHMs) and five haloacetic acids (HAA5), bromate, chlorite; and (b) maximum disinfectant residual level goals (MDRLG) and maximum disinfectant residual level (MDRL) for chlorine, chloramine, and chlorine dioxide.

18.14 A public water supply system collected and analyzed eight samples from a water distribution system on the same day for TTHMs. The laboratory results in μg/L (microgram per liter) are recorded below:

90, 100, 110, 100, 120, 110, 110, and 100

What was the average TTHM for the day?

18.15 The running annual average of the quarterly TTHM measurements can be determined using the following equation:

$$\text{AR-TTHM} = (Q_1\text{-TTHM} + Q_2\text{-TTHM} + Q_3\text{-TTHM} + Q_4\text{-TTHM})/4$$

where

AR-TTHM = average running TTHM, μg/L
Q_1-TTHM = average first quarter TTHM, μg/L
Q_2-TTHM = average second quarter TTHM, μg/L
Q_3-TTHM = average third quarter TTHM, μg/L
Q_4-TTHM = average fourth or current quarter TTHM, μg/L

The results of the quarterly average TTHM measurements for 2 years are recorded below. Calculate the running annual average of the four quarterly measurements in μg/L.

Quarter	1	2	3	4	1	2	3	4
Ave quarterly TTHM (μg/L)	87	98	122	105	93	97	119	99

18.16 A 20 MGD (75.7 MLD) direct filtration plant applies free chlorine as a disinfectant. The chlorine has a contact time of 25 min under peak flow conditions of 20 MGD (75.7 MLD). The pH of water is 7.5 and the water temperature is 10°C. The free chlorine residual is 2 mg/L. Table 18.18 indicates that various filtration processes are capable of achieving certain removal of *Giardia* cysts and viruses.

Determine (a) the log removal of *Giardia* cysts inactivation that must be achieved by chlorination, (b) the *CT* value provided by the contact time of 25 min and chlorine residual of 2 mg/L; and (c) the required *CT* value for the water treatment system.

18.17 The same 20 MGD (75.7 MLD) water treatment plant described in Problem 18.4 will be operated at 50% of maximum flow (i.e., 10 MGD; 37.85 MLD). Please properly operate the plant by the following: (a) determine the new detention time of the water treatment; (b) suggest a new free chlorine residual for water treatment but still provide an adequate *CT* value; (c) determine the actual provided *CT* value; and (d) determine the required *CT* value under the reduced flow condition. It is important to note that the detention time is a function of flow and volume of the water treatment facility.

$$Q = V/T$$

where

Q = water flow, gpm (L/min)
V = volume of water treatment facility, gal (L)
T = detention time of water treatment facility, min

18.18 A 20 MGD (75.7 MLD) conventional filtration plant applies free chlorine as a disinfectant. The chlorine has a contact time of 25 min under peak flow conditions of 20 MGD (75.7 MLD). The pH of the water is 7.5 and the water temperature is 10°C (50°C). Determine (a) the log removal of *Giardia* cysts inactivation that must be achieved by chlorination, (b) the *CT* value provided by the contact time of 25 minutes under an assumed chlorine residual of 1 mg/L; and (c) the required *CT* value for the water treatment system.

18.19 If prechlorination was not practiced in a groundwater treatment plant, what free chlorine residual would be necessary for the clearwell alone to provide adequate disinfection assuming (a) the required *CT* value for disinfection is 11.6 mg/L-min for 4-log removal of virus, (b) the gross detention time for the clearwell is 60 minutes; and (c) the clearwell has a baffled inlet and outlet with some intra-basin baffles, or an average baffling factor of 0.5.

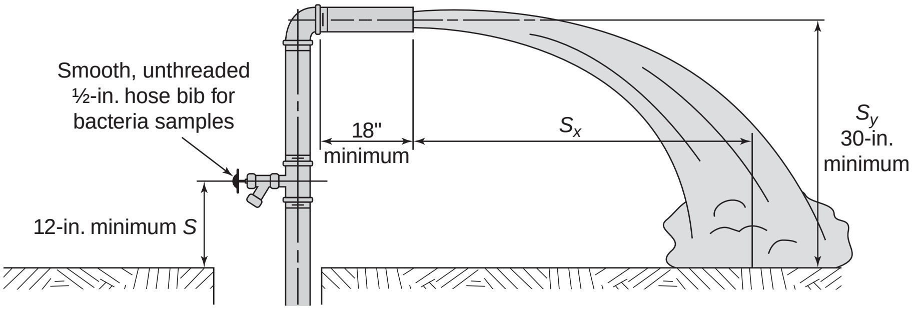


Figure 18.14 Figure for Problem 18.25. Conversion factors: 1 in. = 1″ = 2.54 cm

18.20 The log removal of a disinfection process can be calculated using the following equation:

$$\text{LR} = \text{Log}\,(\text{PCi}) - \text{Log}\,(\text{PCe}) = \text{Log}\,[(\text{PCi})/(\text{PCe})]$$

where

LR = log removal of a disinfection process, dimensionless
PCi = particle of influent, #/mL
PCe = particle of effluent, #/mL

Calculate the log removal of 5–15 micron particles per milliliter (#/mL) if the influent particle count to a water filtration plant reported 2,500 particles in the 5–15 microns range per milliliter of water and the filtration plant effluent reported 20 particles in the 5–15 microns range per milliliter of filtered water.

18.21 The percent reduction and the log removal can be determined by the following two equations:

$$\text{PR} = 100\,(\text{Ci} - \text{Ce})/\text{Ci}$$

where

PR = percent reduction of microorganisms, %
Ci = influent concentration of microorganisms, #/L
Ce = effluent concentration of microorganisms, #/L

$$\text{LR} = \text{Log}\,(\text{Ci}) - \text{Log}\,(\text{Ce}) = \text{Log}\,[(\text{Ci})/(\text{Ce})]$$

where

LR = log removal of a process, dimensionless.

In a nanofiltration (NF) demonstration project, the water treatment plant (WTP) that adopts the NF system for testing successfully reduces *Cryptosporidium* from 2,358,000 #/L to 10 #/L. Determine the percent reduction and log removal of the NF system.

18.22 Prove that

(a) 90% reduction = 1-log removal,
(b) 99% reduction = 2-log removal,
(c) 99.9% reduction = 3-log removal,
(d) 99.99% reduction = 4-log removal, and
(e) 99.999% reduction = 5-log removal.

18.23 Estimate the chlorine demand for water in mg/L if the chlorine dosage is 2.8 mg/L and the chlorine residual is 0.6 mg/L.

18.24 Estimate the feed flow rate of a hypochlorinator in gpm (L/min), if the sodium hypochlorite solution is in a container with a diameter of 3.5 ft (1.07 m) and the hypochlorite level drops 2 ft (0.61 m) during a 120-min time period.

18.25 Figure 18.14 shows an AWWA suggested blowoff and sampling tap for disinfection of water mains using the continuous-feed chlorination method. It is also a very useful method for determination of the hydrant discharge flow rate during hydrant flushing or water distribution system investigation. The formula for estimating the rate of discharge can be estimated using the following equation:

$$Q = 2.83\ \text{d}^2\ (Sx)\ /(Sy)^{0.5} \qquad \text{(US customary units)}$$
$$Q = 0.00329\ d^2\ (Sx)\ /(Sy)^{0.5} \qquad \text{(SI units)}$$

where

Q = discharge in gpm (L/min)
d = inside diameter of discharge pipe, in. (mm)
Sx = horizontal distance of water jet, in. (mm)
Sy = height of water jet, in. (mm)

Estimate the flow from the hydrant in gpm (L/min) during a water main disinfection operation, if the inside diameter of discharge pipe = 2 in. (50 mm), the horizontal distance of water jet = 62 in. (1,575 mm), and the height of water jet = 38 in. (965 mm).

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Chapter 19

Chemical Precipitation and Water Softening

19.1 CHEMICAL PRECIPITATION

Natural waters and wastewaters acquire their chemical characteristics either within the hydrological cycle or during domestic and industrial use. Some of the acquired solids, liquids, and gases are taken into solution and maintain their integrity; others react with water or with one another in water to form new chemical species. Some solids may be left in suspension, some liquids may be immiscible and nonreactive, and some gases may be entrapped only mechanically.

Precipitation is essentially the opposite of dissolution. Understandably, therefore, precipitation, like solution, can often be based on the *law of mass action* and available *equilibrium constants*. However, conformance of this kind does not necessarily assure either stoichiometric completion of the precipitation reaction or the adequacy of added stoichiometric amounts. Amounts greater than those calculated may have to be called on to attain wanted performance. As a rule, precipitants should be chosen from among:

1. Chemicals normally occurring in natural waters—hydroxide and carbonate, for example
2. Relatively insoluble chemicals—iron(III) and aluminum(III), for instance; and, in a general sense
3. Safe chemicals that do not produce toxic residues—for instance, avoidance of Ba^{2+}, because it is toxic and would be left in solution if sulfate were precipitated from drinking water as $BaSO_4$.

Most multivalent cations in water can be precipitated in predictable amounts as carbonates or hydroxides. This is fortunate, because their solubility governs important treatment processes, including chemical stabilization and the precipitation of Ca^{2+} and Mg^{2+}, as well as other cations in water softening. However, knowing the solubility product is not enough. Thus the dissolving species, when $CaCO_3(s)$ is added to pure water, are Ca^{2+} and CO_3^{2-}, and because CO_3^{2-} is a base its reaction products with water are predominantly $HCO_3{}^-$ and OH^-. In more general terms, $CaCO_3$ reacts with hydrogen ions, or an acid, as follows:

$$CaCO_3(s) + H^+ = Ca^{2+} + HCO_3^{2-} \qquad \textbf{(19.1)}$$

It is seen that lowering the pH by adding an acid increases the solubility of $CaCO_3$, whereas raising the pH by adding a base decreases it. Raising the pH, indeed, governs removal of Ca^{2+} by the lime-soda softening process. Systemic and quantitative determinations of $CaCO_3$ solubility relations can be based on the equilibria of Appendix 22.

19.2 DESCRIPTION OF PRECIPITATION PROCESS

Chemical precipitation is a widely used, proven technology for the removal of metals and other inorganics, suspended solids, fats, oils, greases, and some other organic substances (including organophosphates) from wastewater. Generally speaking, precipitation is a method of causing contaminants that are either dissolved or suspended in solution to settle out of solution as a solid *precipitate*, which can then be separated from the liquid portion by sedimentation, flotation, and/or filtration. A voluminous precipitate can capture ions and particles during formation and settling, in effect "*sweeping*" ions and particles from the wastewater. Precipitation is assisted through the use of a coagulant, an agent which causes smaller particles suspended in solution to gather into larger aggregates. Frequently, polymers are used as coagulants. The long-chain polymer molecules can be either positively or negatively charged (cationic or anionic) or neutral (nonionic). Since water chemistry typically involves the interaction of ions and other charged particles in solution, these electrical qualities allow the polymers to act as bridges between particles suspended in solution or to neutralize particles in solution. The specific approach used for precipitation will depend on the contaminants to be removed, as described below.

19.2.1 Metals Removal

Water *hardness* is caused primarily by the dissolution of calcium and magnesium carbonate and bicarbonate compounds in water and, to a lesser extent, the sulfates, chlorides, and silicates of these metals. The removal of these dissolved compounds, called *water softening*, often proceeds by chemical

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Water Engineering: Hydraulics, Distribution and Treatment, First Edition. Nazih K. Shammas and Lawrence K. Wang.

precipitation (see Section 19.11). Lime (calcium oxide), when added to hard water, reacts to form calcium carbonate, which itself can act as a coagulant, sweeping ions out of solution in formation and settling. To do this with lime alone, a great deal of lime is typically needed to work effectively; for this reason, the lime is often added in conjunction with ferrous sulfate, producing insoluble ferric hydroxide. The combination of lime and ferrous sulfate is only effective in the presence of dissolved oxygen, however. Alum, when added to water containing calcium and magnesium bicarbonate alkalinity, reacts with the alkalinity to form an insoluble aluminum hydroxide precipitate.

Soluble heavy metal ions can be converted into insoluble metal hydroxides or carbonates through the addition of hydroxide compounds. Additionally, insoluble metal sulfides can be formed with the addition of ferrous sulfate and lime. Once rendered insoluble, these compounds will tend to precipitate and settle. The solubility of the metal compounds thus formed is pH dependent; most tend to be least soluble in alkaline solutions. Since the optimal pH for precipitation depends both on the metal to be removed and on the counter ion used (hydroxide, carbonate, or sulfide), the best treatment procedure must be determined on a case-by-case basis (see iron and manganese oxidation for their removal in Section 13.12). Metal solubility data are available in Benefield and Morgan, as well as in many other sources.

Once the optimal pH for precipitation is established, the settling process is often accelerated by addition of a polymer coagulant, which gathers the insoluble metal compound particles into a coarse floc that can settle rapidly by gravity.

19.2.2 Removal of Fats, Oils, and Greases

Fats, oils, and greases are typically organic substances which tend to bead together or form “slicks” on the surface of aqueous solutions. They behave in this way because these organic, *nonpolar* substances are typically insoluble in water, which is inorganic and polar. Because they tend to be less dense than water, they float to the surface rather than settling to the bottom. In situations where the oily substance is free floating in slicks, skimming the surface of the solution is often the best way to remove most of the material. However, oils, fats, and greases can become *emulsified* in aqueous solution, meaning that small globules of the oily product can become suspended throughout the water. These globules are localized, particle-like aggregations of compatibly charged molecules existing in an incompatible aqueous medium—which is to say that these molecules are *hydrophobic* (“water-fearing”). Often times, other substances (especially products like soaps and detergents) in solution can act as aids to making hydrophobic substances soluble in water.

To remove emulsified oils and greases, the emulsion must be broken up by destabilizing the electrical charge attractions that keep the localized clusters of oily molecules stable in solution. This can be done with the addition of a polymer designed for charge neutralization. In this way, the charge attraction of the oily particles is disrupted, allowing them to separate from the aqueous solution.

19.2.3 Phosphorus Removal

Metal salts (most commonly ferric chloride or aluminum sulfate, also called alum) or lime has been used for the removal of phosphate compounds from water. When lime is used, a sufficient amount of lime must be added to increase the pH of the solution to at least 10, creating an environment in which excess calcium ions can react with the phosphate to produce an insoluble precipitate (hydroxylapatite). Lime is an effective phosphate removal agent, but results in a large sludge volume. When ferric chloride or alum is used, the iron or aluminum ions in solution will react with phosphate to produce insoluble metal phosphates. The degree of insolubility for these compounds is pH dependent. Moreover, many competing chemical reactions can take place alongside these, meaning that the amount of metal salt to add to the solution cannot simply be calculated on the basis of the phosphate concentration, but must be determined in the laboratory for each case.

Wastewater effluents, even though they may have been biologically treated, will still contain substantial amounts of phosphate, ammonia, and nitrate. All three are nutrients for the plankton and for littoral aquatic weeds. Heavy fertilization (*eutrophication*) may stimulate nuisance growths, especially in lakes, ponds, and similar waters of suitable depth. When this is so, it may pay to remove phosphates from effluents by chemical precipitation. Multivalent metal ions will do this, Fe(III), A1(III), and Ca(II) being candidate additives. All of them are precipitants of phosphate rather than coagulants. Under favorable pH conditions the quantity of metal ions needed for precipitation obeys the stoichiometry of the reaction

$$\mathrm{Al}^{3+} + \mathrm{H}_n\mathrm{PO}_4^{3-n} = \mathrm{AlPO}_4(\mathrm{s}) + n\mathrm{H}^+ \qquad \textbf{(19.2)}$$

Hence 1 mole of A1(III) or Fe(III) is necessary to precipitate 1 mole of phosphorus. The pH dependence of the metal ion and phosphate interaction is quantitatively in accord with the solubility relationships of the metal phosphate and the metal hydroxide, the hydrolysis of the metal ion, and the acid–base equilibria of the phosphate ions (Appendix 21). Figure 19.1 charts the solubility of $\mathrm{FePO_4(s)}$ and $\mathrm{AlPO_4(s)}$ from the equilibrium constants. The total concentration of soluble phosphorus in equilibrium with both solid $\mathrm{FePO_4}$ and $\mathrm{AlPO_4}$ is plotted as a function of pH.

Under neutral and alkaline conditions, $\mathrm{FePO_4}$ and $\mathrm{AlPO_4}$ are quite soluble; much hydroxide is co-precipitated with phosphate, and the stoichiometric efficiency of phosphate removal decreases with increasing pH. Because the acidity of Al^{3+} is less than that of Fe^{3+} (i.e., Al^{3+} hydrolyzes

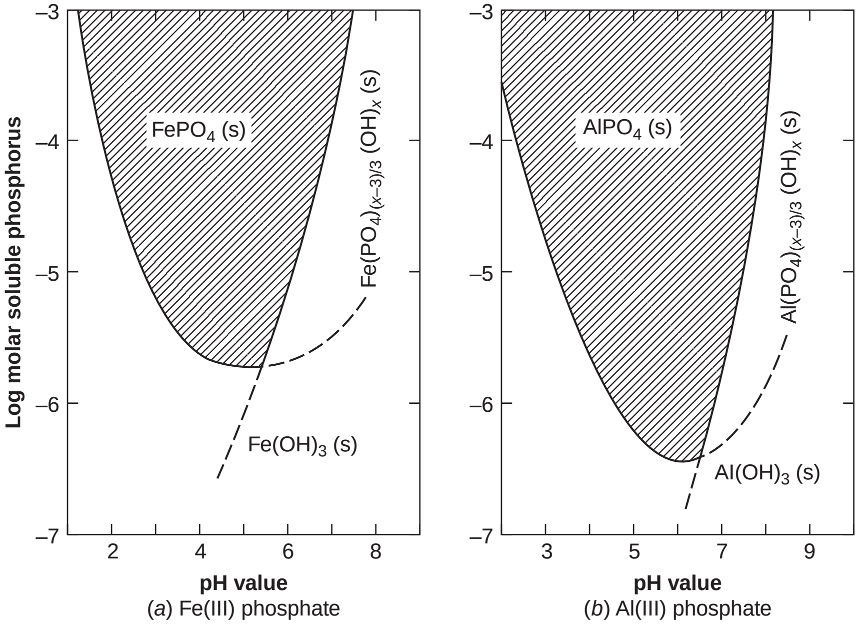

Figure 19.1 Solubility of iron(III) and aluminum(III) phosphates. The solid lines trace the concentration of residual soluble phosphorus after precipitation by Fe(III) or Al(III), respectively, in concentrations equimolar to the original phosphorus concentration. Inside the shaded area pure metal phosphates are precipitated. Outside, toward higher pH values, mixed hydroxo-phosphato metal precipitates are formed (After Fair et al., 1971).

at a higher pH than does Fe^{3+}), the difference in pH values at minimum $A1PO_4$ and $FePO_4$ solubility is about +1 unit. For the same reason, the difference in accompanying minimum solubilities of $A1PO_4$ and $FePO_4$ is about –1 unit, even though $A1PO_4$ has a slightly larger solubility product. Accordingly, proper pH adjustment can achieve a high degree of removal control. Polymeric phosphates and organic phosphates are also precipitated by Fe(III) and Al(III).

In the alkaline pH range, Ca^{2+} ions can precipitate phosphate. The solubility of phosphate is then controlled by the solubility equilibrium of hydroxylapatite, or

$$Ca_{10}(PO_4)_6(OH)_2(s) = 10Ca^{2+} + 6PO_4^{3-} + 2OH^- \quad (\log K \approx -90; 25°C) \qquad \textbf{(19.3)}$$

The so-called calcium phosphate, $Ca_3(PO_4)_2(s)$, is not formed under such conditions. The magnitude of the equilibrium constant for Eq. (19.3) indicates that the addition of Ca^{2+} and proper pH adjustment could render the residual concentration of soluble phosphorus insignificant. Because $[Ca^{2+}] \gg [P]$ in most waters, any base producing a pH greater than 10 will exceed the solubility product of hydroxylapatite. For economic reasons, lime is generally the chemical of choice, even though the efficiency of phosphate removal by calcium at a given pH falls below the prediction based on solubility equilibria. Reasons for this discrepancy are that (1) precipitation of $Ca_{10}(PO_4)_6(OH)_2(s)$ is slow and solubility equilibrium is not reached within economically justifiable detention times; (2) particles of precipitated hydroxylapatite may behave as colloids and not settle out readily; (3) certain polymeric phosphates are capable of forming soluble polyphosphato-calcium complexes; and (4) within certain pH ranges, some of the Ca^{2+} in the water is used up in $CaCO_3$ precipitation.

19.2.4 Removal of Suspended Solids

Finely divided particles suspended in solution can elude filtration and other similar removal processes. Their small size allows them to remain suspended over extended periods of time. More often than not, the particles populating water and wastewater are negatively charged. For this reason, cationic polymers are commonly added to the solution, both to reduce the surface charge of the particles and also to form bridges between the particles, thus causing particle coagulation and settling.

Alternatively, lime can be used as a clarifying agent for removal of particulate matter. The calcium hydroxide reacts in the wastewater solution to form calcium carbonate, which itself acts as a coagulant, sweeping particles out of solution.

19.2.5 Additional Considerations

The chemical agents most frequently used for chemical precipitation are shown in Table 19.1. The amount of chemicals required for treatment depends on the pH and alkalinity of the wastewater, the phosphate level, and the point of injection and mixing modes, among other factors. Competing reactions often make it difficult to calculate the quantities of additives necessary for chemical precipitation. Accurate doses should be determined by jar tests and confirmed by field evaluations. Chemicals are usually added via a chemical feed system that can be completely enclosed and may also include storage space for unused chemicals. Choosing the most effective precipitant and coagulant depends on jar test results, ease of storage, ease of transportation, and consideration of the operation and maintenance costs for associated equipment.

Although chemical precipitation is a well-established treatment method, research continues to enhance its

Table 19.1 Chemicals used in precipitation

Lime—calcium oxide, CaO and calcium hydroxide $Ca(OH)_2$

Produces calcium carbonate in wastewater which acts as a coagulant for hardness and particulate matter. Often used in conjunction with other coagulants, since (1) by itself, large quantities of lime are required for effectiveness and (2) lime typically generates more sludge than other coagulants.

Ferric sulfate—$Fe_2(SO_4)_3$

Typically used with lime to soften water. The chemical combination forms calcium sulfate and ferric hydroxide. Wastewater must contain dissolved oxygen for reaction to proceed successfully.

Alum or filter alum—$Al_2(SO_4)_3 \cdot 14\ H_2O$

Used for water softening and phosphate removal. Reacts with available alkalinity (carbonate, bicarbonate, and hydroxide) or phosphate to form insoluble aluminum salts.

Ferric chloride—$FeCl_3$

Reacts with alkalinity or phosphates to form insoluble iron salts.

Polymer

High molecular weight compounds (usually synthetic) which can be anionic, cationic, or nonionic. When added to water, can be used for charge neutralization for emulsion-breaking or as bridge-making coagulants, or both. Can also be used as filter aids and sludge conditioners.

Source: US EPA modified.

effectiveness. Much recent research concentrates on combining chemical precipitation with other treatment methods such as photochemical oxidation, reverse osmosis, and biological methods to optimize performance. Wang and Wu (1982) have demonstrated the use of magnesium carbonate in a dissolved air flotation (DAF) and recarbonation softening system for removal of calcium hardness (417 mg/L as $CaCO_3$) from groundwater.

19.3 APPLICABILITY

Chemical precipitation can be used to remove contaminants from water and wastewater. It can be used for water softening, heavy metal removal from metal plating wastes, oil and grease removal from emulsified solutions, and phosphate removal from wash-waters and other wastewater. It is an effective tool for wastewater polishing and removal of particulate matter.

19.4 ADVANTAGES AND DISADVANTAGES

Before deciding whether chemical precipitation meets the treatment needs, it is important to understand the advantages and disadvantages of this methodology. The *advantages* include the following:

1. Chemical precipitation is a well-established technology with ready availability of equipment and many chemicals.
2. Some treatment chemicals, especially lime, are very inexpensive.
3. Completely enclosed systems are often conveniently self-operating and low maintenance, requiring only replenishment of the chemicals used. Oftentimes, a sophisticated operator is not needed.

The disadvantages are as follows:

1. Competing reactions, varying levels of alkalinity, and other factors typically make calculation of proper chemical dosages impossible. Therefore, frequent jar tests are necessary for confirmation of optimal treatment conditions. Overdosing can diminish the effectiveness of the treatment.
2. Chemical precipitation may require working with corrosive chemicals, increasing operator safety concerns.
3. The addition of treatment chemicals, especially lime, may increase the volume of waste sludge up to 50%.
4. Large amounts of chemicals may need to be transported to the treatment location.
5. Polymers can be expensive.

Table 19.2 provides a summary of properties and considerations appropriate to chemicals commonly used for precipitation.

19.5 DESIGN CRITERIA

Chemical precipitation is normally carried out through a chemical feed system, most often a totally automated system providing for automatic chemical feeding, monitoring, and control. Full automation reduces manpower requirements, allows for less sophisticated operator oversight, and increases efficiency through continuous operation.

An automatic feed system may consist of storage tanks, feed tanks, metering pumps (although pumpless systems do exist), overflow containment basins, mixers, aging tanks, injection quills, shot feeders, piping, fittings, and valves. Figure 19.2 shows a simplified flow chart of a chemical feed system.

Chemical feed system storage tanks should have sufficient capacity to run for some time without running out and causing downtime. At least a 1-month supply of chemical storage capacity is recommended, though lesser quantities may be justified when a reliable supplier is located nearby, thus alleviating the need for maintaining substantial storage space. Additive chemicals come in liquid and dry form (see Table 19.2).

When working with dry chemicals, a volumetric feeder or a gravity feeder can be used to measure the amount of chemical to be dissolved in water. Gravimetric feeders measure the chemical as a weight per unit time; volumetric feeders, by contrast, measure the chemical volume per unit time. While gravimetric feeders are more expensive than

Table 19.2 Commercial forms of precipitation chemicals

Chemical	Commercial characteristics
Alum	Alum is an off-white crystal which, when dissolved in water, produces acidic conditions. As a solid, alum may be supplied in lumps, but is available in ground, rice, or powdered form Shipments range from small 100 lb (45.4 kg) bags, to bulk quantities of 4,000 lb (1816 kg). In liquid form, alum is commonly supplied as a 50% solution delivered in minimum loads of 4,000 gal (15,140 L) The choice between liquid and dry alum depends on the availability of storage space, the method of feeding, and economics
$FeCl_3$	Ferric chloride, or $FeCl_3$, is available in either dry (hydrate or anhydrous) or liquid form. The liquid form is usually 35–45% $FeCl_3$. Because higher concentrations of $FeCl_3$ have higher freezing points, lower concentrations are supplied during the winter. It is highly corrosive
Lime	Lime can be purchased in many forms, with quicklime (CaO) and hydrated lime ($Ca(OH)_2$) being the most prevalent forms. In either case, lime is usually purchased in the dry state, in bags, or in bulk.
Polymer	Polymers may be supplied as a prepared stock solution ready for addition to the treatment process or as a dry powder. Many competing polymer formulations with differing characteristics are available, requiring somewhat differing handling procedures. Manufacturers should be consulted for recommended practices and use.

Source: US EPA.

volumetric ones, they are also more accurate. Even so, volumetric feeding systems are more commonly used. In either case, the type of feeding mechanism required depends on the feed rate anticipated. Table 19.3 summarizes the types of feeding mechanisms available with associated feed rates.

In choosing a feed system, one must be certain that the materials used to build the system are chemically compatible with the chemicals to be used. Equipment manufacturers' chemical resistance charts should be used in selecting appropriate construction materials.

Table 19.3 Types of feed mechanisms within volumetric and gravimetric feeders

Dry feed mechanism	Feed rate, lb/h (kg/h)
Rotating disk	10 (4.54)
Oscillating	10–100 (4.54–45.4)
Rotary gate	200–500 (90.8–227)
Belt	500–20,000 (227–9,080)
Screw[a]	10–24,000 (4.54–10,896)

Source: US EPA.

[a] Typically for volumetric feeders.

19.6 PERFORMANCE—JAR TESTING

For any given water or wastewater, the optimal treatment strategy should be determined by jar testing. Commercial chemical vendors provide testing guidelines to determine the most appropriate chemical(s) and the most effective dosage. Laboratory bench-scale jar testing apparatuses typically allow for six samples, each 1 liter in size, to be tested simultaneously. One central control operates the mixing of all jars, hence one variable (e.g., polymer dosage) can be manipulated in a test group while all other factors, including mixing rates and times, can be kept constant. Inconclusive and incorrect interpretation of the results may be the result of using too small a sample for stock solution, adding chemicals inconsistently, erroneous data recording, using old chemicals, choosing improper flocculation and settling conditions (time, duration), using different people to perform tests, and choosing too narrow a dosage range.

19.7 OPERATION AND MAINTENANCE

A routine O&M schedule should be developed and implemented for any type of bulk chemical feed/handling system. Many systems are now completely enclosed, factory mounted/piped/wired systems. All manufacturer O&M recommendations should be followed, including testing and calibration. Regular O&M includes the following:

1. Occasional flushing of the system, if this is not provided automatically

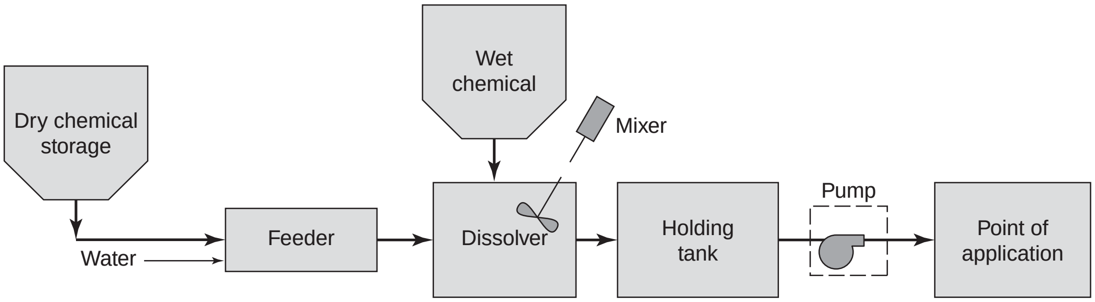

Figure 19.2 Flow diagram of a chemical feed system (US EPA).

2. Inspecting and replacing pump seals, bags, dust filters, pH- and ion-specific electrodes, and other components
3. Periodically lubricating bearings, motor, and other moving parts
4. Developing an emergency response plan for onsite storage of chemicals

19.8 COSTS

The overall cost of chemical precipitation depends on many variables, including the characteristics of the water, the chemicals and dosages to be used, the volume of water to be treated, and the level of water purity desired. Moreover, chemical costs can vary widely depending on the form and quantity of material to be procured. Material prices fluctuate according to the region of the country. Chemicals provided in bags or measured batches are more expensive than when purchased in bulk quantities. Table 19.4 summarizes some chemical prices as of January 2000, reported in terms of 2014 USD using Appendix 16.

It should be noted, however, that estimation of treatment cost cannot be determined solely on the price of the chemicals. For example, one study found that while it was less expensive to purchase alum than ferric sulfate, overall treatment costs were less using ferric sulfate.

Generally speaking, lime is readily available and the least expensive of common treatment options. A completely enclosed lime chemical handling system costs between 2014 USD 153,000 and USD 188,000, with the lower prices reflecting gravity versus pump systems. The more expensive lime systems include a slaker to convert calcium oxide to calcium hydroxide. These systems can consume up to 2000 lb (907 kg) of lime/h and include storage for up to 60,000 lb (27,216 kg) of lime. The trade-off in using lime, however, is in the large sludge volumes that result from its use. For this reason, lime is often used in tandem with more expensive additives. These trade-offs must be kept in mind when performing jar tests and developing the optimal treatment strategy for each situation.

Table 19.4 Costs of selected chemicals

	2014 Cost
Product description	USD/ton (USD/metric ton)
Aluminum sulfate, liquid, in tanks, iron free	386 (425)
Aluminum sulfate, liquid, in tanks, NOT iron free	218 (240)
Aluminum sulfate, dry, 100 lb (45.4 kg) bags, iron free	360 (396)
Aluminum sulfate, dry, 100 lb (45.4 kg) bags, NOT iron free	353–402 (388–443)
Ferric chloride, technical grade, in tanks	366–431 (388–473)
Ferrous sulfate, monohydrate, granulated, bulk	351 (385)
Lime, chemical, hydrated, bulk	100 (110)

Source: Updated US EPA.

19.9 PRECIPITATION OF HARDNESS AND CARBONATES—WATER SOFTENING

Calcium and magnesium ions are the principal hardness-forming constituents of water. The adverse effect of hard water is an economic one:

1. Consumption of soap
2. Rings and stains in sanitary fixtures
3. Deterioration of fabric
4. Deposition in boilers

In the lime-soda process of water softening, Ca^{2+} is precipitated as $CaCO_3$ and Mg^{2+} as $Mg(OH)_2$. However, it is important to recognize that other carbonic constituents are affected at the same time. Thus the solubility relations of $CaCO_3$ and $Mg(OH)_2$ shown in Figs. 19.3a and 19.3b suggest that removal of hardness and carbonate constituents is only achieved to best advantage (1) when the pH of the final solution is sufficiently high to depress the solubility of $CaCO_3$ and $Mg(OH)_2$ and (2) when $[Ca^{2+}]_{\text{final}} = C_{\text{T final}}]$ (C_T = total dissolved carbon species: $[H_2CO_3] + [HCO_3^-] + [CO_3^{2-}]$), a prerequisite for simultaneous precipitation of Ca^{2+} and carbonate with equal efficiency. The solubility of $[Mg^{2+}]$ is governed by the solubility product of $Mg(OH)_2$ (Appendix 21).

The pH is raised conveniently and economically with lime (CaO) or $Ca^{2+} + 2OH^-$ after addition to water. To equalize the final stoichiometric concentrations of Ca^{2+} and C_T, a carbonate-bearing base such as Na_2CO_3 (soda ash) is added. The following condition must be met after lime and soda have been added:

$$[Ca^{2+}]_{\text{original}} + [Ca^{2+}]_{\text{added lime}} = C_{\text{T original}} + [C]_{\text{added } Na_2CO_3} \tag{19.4}$$

Requisite amounts of chemicals are dictated by (1) Eq. (19.4) and (2) the desired degree of softening. Concentrations of residual Ca^{2+}, C_T, and Mg^{2+} are given by the final pH of the solution, provided that solubility equilibria have been reached (Figs. 19.3a and 19.3b). Therefore, the chemical dosage necessary to reach the pH value that produces the selected residual concentration levels of soluble Ca^{2+}, C_T, and Mg^{2+} can either be calculated or determined experimentally. Experimentally, the conditions laid down in

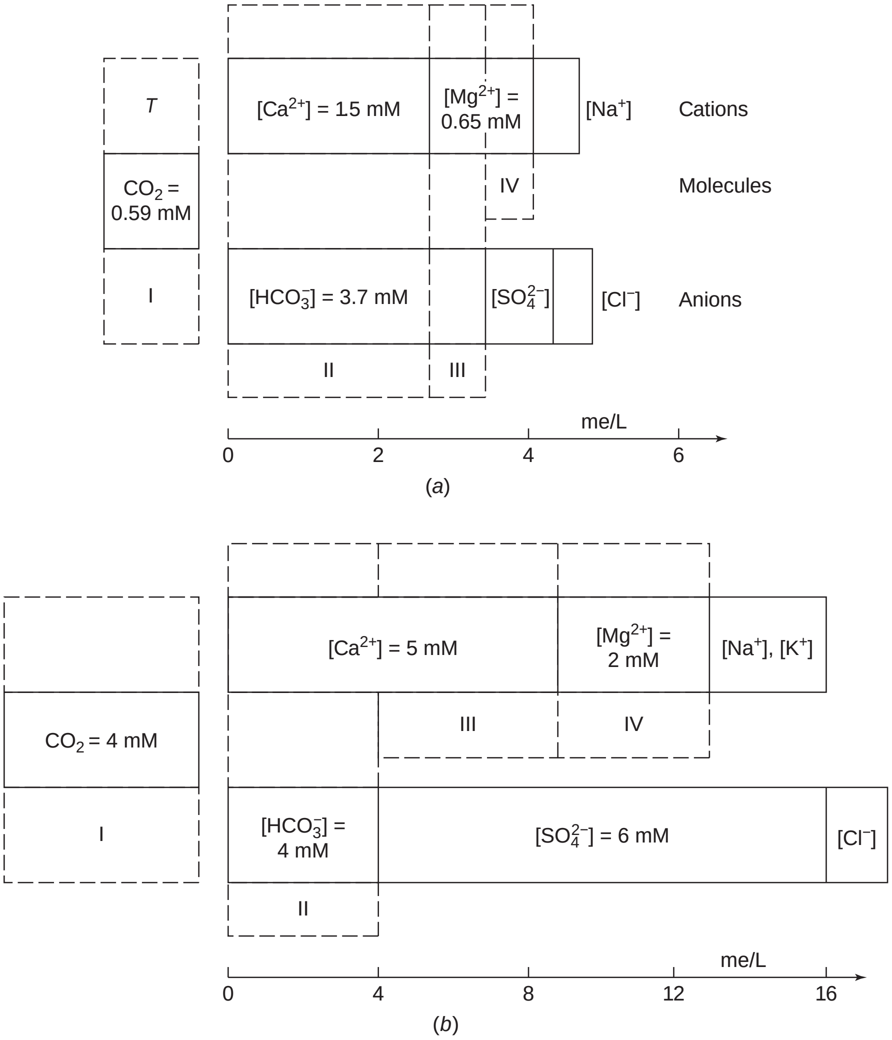

Figure 19.3 Bar diagram for cation–anion balance of waters to be softened by lime-soda process (Examples 19.1 and 19.2) (After Fair et al., 1971).

Eq. (19.4) are met by first adding lime or soda ash to adjust $[Ca^{2+}]_{\text{original}}$ to $C_{T\ \text{original}}$, followed by simultaneous titration of a sample of the water to be softened from two burettes dispensing equimolar lime and soda solutions. Small and volumetrically equal increments of both solutions are added until the pH approximates the desired value.

An approximate evaluation of requisite chemicals can be obtained from the stoichiometry of the following reactions:

$$H_2CO_3 + Ca(OH)_2 \rightarrow CaCO_3(s) + 2H_2O \quad \textbf{(19.5)}$$

$$Ca^{2+} + 2HCO_3^- + Ca(OH)_2 \rightarrow 2CaCO_3(s) + 2H_2O \quad \textbf{(19.6)}$$

$$Ca^{2+} + Na_2CO_3 \rightarrow CaCO_3(s) + 2Na^+ \quad \textbf{(19.7)}$$

$$HCO_3^- + Ca(OH)_2 \rightarrow CaCO_3(s) + OH^- + H_2O \quad \textbf{(19.8)}$$

$$\begin{aligned} Mg^{2+} + 2HCO_3^- + 2Ca(OH)_2 &\rightarrow 2CaCO_3(s) \\ &+ Mg(OH)_2(s) + 2H_2O \quad \textbf{(19.9)} \end{aligned}$$

$$\begin{aligned} Mg^{2+} + Ca(OH)_2 + Na_2CO_3 &\rightarrow CaCO_3(s) \\ &+ Mg(OH)_2(s) + 2Na^+ \quad \textbf{(19.10)} \end{aligned}$$

The reactions are assumed to go to completion.

In summary there are four variations of the lime-soda process:

1. *Single-stage lime process*: Application to water containing only carbonate hardness, high calcium and low magnesium
2. *Excess lime process*: Application to water containing only carbonate hardness, high calcium and high magnesium
3. *Single-stage lime-soda-ash process*: Application to water containing carbonate hardness, high calcium and low magnesium and some noncarbonate hardness (NCH)
4. *Excess lime-soda process*: Application to water containing carbonate hardness, high calcium and high magnesium and some NCH

EXAMPLE 19.1 CALCULATIONS OF LIME AND SODA ASH CONCENTRATIONS REQUIRED FOR WATER SOFTENING

Calculate how much lime and soda ash are required to soften a raw water with a total hardness = 215 mg/L (as $CaCO_3$) or 2.15×10^{-3} M; magnesium = 15.8 mg/L (as Mg^{2+}) or 0.65×10^{-3} M; Na^+ = 8 mg/L or 0.35×10^{-3} M; SO_4^{2-} = 28.6 mg/L or 0.3×10^{-3} M; Cl^- = 10 mg/L or 0.285×10^{-3} M; alkalinity = 185 mg/L (as $CaCO_3$) or 3.7×10^{-3} eq/L; carbon dioxide = 25.8 mg/L (as CO_2) or 0.59×10^{-3} M; pH = 6.7.

Solution:

Strike an electroneutrality balance as in Fig. 19.3a, recognizing that, at pH 6.7, $[HCO_3^-] \gg [CO_3^{2-}]$, a condition applying to virtually all hard and high-alkalinity waters. Subdivide the diagram into individual blocks after computing the required dosages as in Table 19.5.

Verify that Eq. (19.4) is fulfilled, that is, that

$$[Ca^{2+}] + [\text{lime}] = C_T + [\text{soda}].$$

By substitution, find that this is so:

$$1.5 \times 10^{-5} + (3.09 + 0.35) \times 10^{-3} = (0.59 + 3.7) \times 10^{-3} + 0.30 \times 10^{-3}$$

$$1.5 \times 10^{-3} + 3.44 \times 10^{-3} = 4.29 \times 10^{-3} + 0.30 \times 10^{-3} = 4.59 \times 10^{-3}.$$

Table 19.5 Computed electroneutrality balance (Example 19.1).

Block no.	Applicable equation	Requisite CaO, mM[a]	Requisite Na_2CO_3, mM
I	16–12	0.59	—
II	16–13	1.50	…
III	16–16	0.70	…
IV	16–17	0.30	0.30
Total to be applied		3.09	0.30

After Fair et al. (1971).

[a] mM = millimolar.

EXAMPLE 19.2 CALCULATIONS OF LIME AND SODA ASH CONCENTRATIONS REQUIRED FOR WATER SOFTENING

Calculate the amounts of lime and soda ash required to soften a water of the following composition: $[Ca^{2+}] = 5 \times 10^{-3}$ M; $Mg^{2+} = 2 \times 10^{-3}$ M; $[HCO_3^-] = 4 \times 10^{-3}$ M; $H_2CO_3 = 4 \times 10^{-3}$ M; $SO_4^{2-} = 6 \times 10^{-3}$ M; $C1^- = 0.8 \times 10^{-3}$ M; pH = 6.3.

Solution:

To strike the electroneutrality balance shown in Fig. 19.3b, calculate the requisite amounts of chemicals shown in Table 19.6. To hasten and complete precipitation, add lime and soda in excess of calculated values.

Table 19.6 Computed electroneutrality balance (Example 19.2)

Block no.	Applicable equation	Requisite CaO, mM	Requisite Na_2CO_3, mM
I	16–12	4	—
II	16–13	2	…
III	16–14	…	3
IV	16–17	2	2
Total		8	5
Excess[a]		1	1
To be applied		9	6

After Fair et al. (1971).

[a] In excess lime treatment, carbonate equivalent to the excess of lime must be added. Otherwise the calcium ions in excess would defeat the purpose of softening. Carbon dioxide is often added in lieu of Na_2CO_3.

Examples 19.1 and 19.2, together with Eqs. (19.7) and (19.10), show that the amount of soda ash needed for efficient softening is a function of the so-called *noncarbonate hardness* (*NCH*). NCH is the total hardness not compensated by HCO_3^- or CO_3^{2-} in a charge-balance equation, that is,

$$\mathrm{NCH(eq/L)} = 2[\mathrm{Ca}^{2+}] + 2[\mathrm{Mg}^{2+}] - [\mathrm{HCO}_3^-] - 2[\mathrm{CO}_3^{2-}] \tag{19.11}$$

The NCH has also been called *permanent hardness*, because it is not precipitated when water is heated to the boiling point. Because soda ash is relatively expensive in comparison with lime, some of the NCH is purposely not removed for the sake of economy. Instead it is left behind as the residual hardness of the finished water. When lime softening is combined with ion exchange softening (Chapter 20), the lime serves the purpose of precipitating most of the carbonate hardness and the ion exchange eliminates the remaining carbonate hardness as well as the NCH.

The residual hardness of lime-soda-softened waters is normally higher than its calculated value, because detention times can seldom be made long enough to ensure full precipitation. To speed precipitation, lime and soda ash are added to the raw water in excess of stoichiometric requirements. Thus the treated water will usually contain the following amount of hardness:

Soluble $CaCO_3$ = 40 mg/L as $CaCO_3$.
Soluble $Mg(OH)_2$ = 10 mg/L as $CaCO_3$.
Excess lime = 60 mg/L as $CaCO_3$.
Hence total hardness in effluent water = 40 + 10 + 60 = 110 mg/L as $CaCO_3$.

To shorten the settling time and remove precipitates more efficiently, coagulants—alum, activated silica, or polyelectrolytes, for instance—may be added. The solubilities of $CaCO_3$ and $Mg(OH)_2$ decrease, and their rates of precipitation increase, with rising temperatures. Therefore, hot-process softening is commonly used in boiler-water treatment, and sometimes also to obtain waters of very low residual hardness.

EXAMPLE 19.3 COMPUTATION OF DAILY CHEMICAL DOSAGES FOR WATER SOFTENING

A water treatment plant has a flow of 12 MGD (45.42 MLD). Raw water analysis shows the presence of the following compounds:

NaCl	400 mg/L
CO_2	8 mg/L
$Ca(HCO_3)_2$	200 mg/L
$MgSO_4$	50 mg/L

How many lb (kg) of lime (as CaO) and/or soda ash (Na_2CO_3) are required per day to soften the water?

Solution 1 (US Customary System):

Molecular weights from Appendix 23:

$CO_2 = 12 + 2 \times 16 = 44$.
CO_2 in water = $H_2CO_3 = 2 \times 1 + 12 + 3 \times 16 = 62$.
$Ca(HCO_3)_2 = 40 + 2(1 + 12 + 3 \times 16) = 40 + 2 \times 61 = 162$.
$CaO = 40 + 16 = 56$.
CaO in water = $Ca(OH)_2 = 40 + 2(16 + 1) = 74$.
$MgSO_4 = 24.3 + 32 + 4 \times 16 = 120.3$.
$Na_2CO_3 = 2 \times 23 + 12 + 3 \times 16 = 106$.

$$H_2CO_3 + Ca(OH)_2 \rightarrow CaCO_3(s) + 2H_2O \tag{19.12}$$

44 as CO_2	56 as CaO
8	?

mg/L CaO = 8 × 56/44 = 10.18 mg/L.
lb/day CaO = 12 MGD × 10.18 mg/L × 8.34 lb/gal = 1,019 lb/day of lime.

$$Ca(HCO_3)_2 + Ca(OH)_2 \rightarrow 2CaCO_3(s) + 2H_2O \tag{19.13}$$

162	56 (as CaO)
200	?

CaO = 200 × 56/162 = 69.14 mg/L CaO.
lb/day CaO = 12 MGD × 69.14 mg/L × 8.34 lb/gal = 6,920 lb CaO/day.

$$MgSO_4 + Ca(OH)_2 + Na_2CO_3 \rightarrow CaCO_3(s) + Mg(OH)_2(s) + Na_2SO_4 \tag{19.14}$$

120.3	56 (as CaO)	106
50	?	?

CaO = 50 × 56/120.3 = 23.28 mg/L lime.
Na_2CO_3 = 50 × 106/120.3 = 44.06 mg/L soda ash.
lb/day CaO = 12 MGD × 23.28 mg/L × 8.34 lb/gal = 2,330 lb/day of lime.
lb/day Na_2CO_3 = 12 MGD × 44.06 mg/L × 8.34 lb/day = **4,410 lb/day of soda ash.**
lb/day total CaO = 1,019 + 6,920 + 2,330 = **10,269 lb/day of lime.**

Solution 2 (SI System):

The computation of mg/L dosages of lime and soda ash are the same as in Example 19.3:

Lime = 10.18 mg/L + 69.14 mg/L + 23.28 mg/L = mg/L CaO.
Soda ash = 44.06 mg/L Na_2CO_3.
kg/day CaO = 102.60 mg/L × 45 × 10^6 L/d/(10^6 mg/kg) = **4,617 kg CaO/day.**
kg/day Na_2CO_3 = 44.06 mg/L × 45 × 10^6 L/d/(10^6 mg/kg) = **1,983 kg Na_2CO_3/day.**

19.10 RECARBONATION AFTER WATER SOFTENING

Waters softened by the lime-soda process are generally supersaturated with $CaCO_3$ and $Mg(OH)_2$. They can be *stabilized* by blowing CO_2 into them to lower the pH from around 11 down to 8.5.

Normally this is done before filtration. The so-called *secondary carbonation* or *recarbonation* relieves supersaturation and reduces precipitation of $CaCO_3$ on sand grains and in pipelines. Small amounts of polyphosphates or meta-phosphates are also helpful. Although phosphates form soluble complexes with Ca^{2+}, too little phosphate is normally added to account for the observed retardation of precipitation. Moreover, polyphosphates do not prevent the precipitation of $Mg(OH)_2$.

Reduction in the concentrations of fluorides and silica is an interesting side reaction of the excess lime treatment of high-magnesium waters. Co-precipitation of F^- and silicate with $Mg(OH)_2$ forms nonstoichiometric fluoro- and silicato-hydroxo precipitates of Mg^{2+}.

19.11 RECOVERING LIME AFTER WATER SOFTENING

Numerous ways have been proposed for recovering lime from the sludge produced by softening. Precipitated $CaCO_3$ can be converted into CaO by calcining, that is, by heating the dried solids to drive off CO_2. However, the calcined sludge must not contain much magnesium if it is to be recycled. The Hoover and the Lykken–Estabrook processes are examples of effective recovery. The Hoover process does so by two-stage treatment. In the first stage only enough lime is added to precipitate the Ca^{2+} ions. In the second stage further amounts of lime precipitate the Mg^{2+} ions. The first-stage sludge is recovered; the second-stage sludge is wasted. The carbon dioxide produced during calcining can be used to recarbonate the softened water. In the Lykken–Estabrook process all the recovered precipitate is added to about 12% of the water being softened. At the resulting high pH both Ca^{2+} and Mg^{2+} and most of the $Mg(OH)_2$ in the added chemical are precipitated. Because of this the second-round sludge is wasted. Subsequently the overtreated

(12%) portion of water is mixed with the mainstream and softens it.

PROBLEMS/QUESTIONS

19.1 What causes hardness in water?

19.2 What is water softening?

19.3 How are soluble heavy metals removed from water?

19.4 What do we mean by 'Oils, fats, and greases can become *emulsified* in aqueous solution'?

19.5 How is emulsified oil removed from solution?

19.6 How are phosphates removed from water?

19.7 What are the factors that one usually takes into consideration in choosing the most effective precipitant and coagulant?

19.8 What can chemical precipitation remove from water and wastewater?

19.9 What are the advantages and disadvantages of chemical precipitation?

19.10 The overall cost of chemical precipitation depends on many variables. What are these variables?

19.11 In water softening by chemical precipitation

1. What are the adverse effects (disadvantages) of using hard water?
2. Name the four water softening processes and explain the particular conditions under which each process is used.
3. Recarbonation is often applied after water softening. Give reasons and explain why.

19.12 Groundwater was analyzed and found to have the following composition, with all concentrations as $CaCO_3$:

H_2CO_3 = 195 mg/L (carbon dioxide in water)
Ca^{2+} = 320 mg/L
Mg^{2+} = 90 mg/L
Na^{+} = 90 mg/L
HCO_3^- = 240 mg/L (bicarbonate alkalinity)
Cl^- = 80 mg/L
SO_4^{2-} = 180 mg/L

1. Draw a bar diagram of the untreated water.
2. Establish the hardness distribution.
3. Calculate the chemicals requirements (chemical dosage) for softening the water.
4. Estimate the hardness of the treated water.

19.13 A water treatment plant has a flow of 3 MGD. How many pounds of lime (CaO) and soda ash (Na_2CO_3) will be required per day to soften the water, knowing that it contains 100 mg/L Ca $(HCO_3)_2$ and 50 mg/L $CaSO_4$.

19.14 (Metric) A water treatment plant has a flow of 11 ML/day. How many kg of lime (CaO) and soda ash (Na_2CO_3) will be required per day to soften the water, knowing that it contains 100 mg/L Ca $(HCO_3)_2$ and 50 mg/L $CaSO_4$.

19.15 (Metric) A water softening plant has a flow of 76 ML/day of hard water. The water has the following chemical content

CO_2	6 mg/L
$Ca(HCO_3)_2$	150 mg/L
$MgCl_2$	100 mg/L

How many kg of lime (as CaO) and/or soda ash (Na_2CO_3) are required per day to soften the water?

19.16 A water softening plant has a flow of 20 MGD of hard water. The water has the following chemical content

CO_2	6 mg/L
$Ca(HCO_3)_2$	150 mg/L
$MgCl_2$	100 mg/L

How many lb of lime (as CaO) and/or soda ash (Na_2CO_3) are required per day to soften the water?

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Chapter 20

Adsorption and Ion Exchange

Solid–liquid adsorption plays an important part in a variety of water and wastewater treatment processes. In most of these, activated carbon is, so far, the *adsorbent of choice*. Adsorption is the accumulation of atoms or molecules on the surface of a material. This process creates a film of the *adsorbate* where the molecules, ions, or atoms are accumulated on the *adsorbent*'s surface. It should not be mixed with *absorption*, in which a substance diffuses into a liquid or solid. The term *sorption* includes both processes, while desorption is the reverse process.

Adsorption is present in many natural physical, biological, and chemical systems, and is widely used in industrial applications such as activated charcoal, synthetic resins, and water purification. Adsorption and ion exchange are sorption processes in which certain adsorbates are selectively transferred from the fluid phase to the surface of insoluble, rigid particles suspended in a vessel or packed in a column.

Ion exchange and membrane processes are becoming used extensively in water and wastewater treatment. Ion exchange is primarily used for the removal of hardness ions, such as magnesium and calcium, and for water demineralization. Reverse osmosis (RO) and electrodialysis, both membrane processes, remove dissolved solids from water using membranes.

Adsorption is a consequence of surface energy. The exact nature of the bonding depends on the details of the species involved, but the adsorption process is generally classified as physical sorption that is a characteristic of weak van der Waals forces or chemical sorption, which is characteristic of covalent bonding.

20.1 ADSORPTION PROCESSES

The accumulation of a substance as an adsorbate at an interface is a vectorial function of the forces of attraction and repulsion of the substance to and from the solution phase or from and to the solid phase, respectively. The affinity of adsorbents for adsorbates varies with the force fields at (or built up at) the liquid–solid interface. Affinity to the solution phase—often repulsion rather than attraction—cannot be modified. Substances repelled from solution are surface active; that is, they decrease surface or interfacial tension, and tend to accumulate at an interface. This they do because of the dual constitution of their responding molecules or ions. Their hydrophilic part is attracted relatively more to the water phase than to the solid phase, and their hydrophobic part is repelled relatively more from the water phase than from the solid phase. Detergents are examples of surface-active substances, but most organic constituents of wastewaters exhibit some surface or interfacial activity. By contrast, simple ions tend to increase surface tension and are desorbed from interfaces. The failure of activated carbon to remove ions such as Ca^{2+}, Cl^-, and F^- from solution is an example. Understandably, the interfacial adsorption of organic molecules increases with their surface activity and their size. Understandably, too, interfacial adsorption depends on the structure of the adsorbent and not only on the relative magnitude of its interfacial area. Other surface properties being equal, adsorbents with large pore openings, for instance, should adsorb large molecules and colloidal particles preferentially. Macromolecular dyes are indeed removed from solution selectively by wide-pore adsorbents.

20.2 ADSORPTION KINETICS AND EQUILIBRIA

Ordinary, or physical, adsorption is usually rapid. It is reversible, and a condition of equilibrium between adsorbed and dissolved *adsorbate* is reached soon after contact with the *adsorbent*. However, when the adsorbent is a porous solid, like granular rather than powdered activated carbon (PAC), or when the concentration of the adsorbate is small, full contact may be limited by diffusional or other transport processes. The attainment of equilibrium is then delayed correspondingly.

20.2.1 Adsorption Kinetics

Adsorption by activated carbon involves the accumulation or concentration of substances at a surface or interface. Adsorption is a process in which matter is extracted from one phase and concentrated at the surface of another, and is therefore

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Water Engineering: Hydraulics, Distribution and Treatment, First Edition. Nazih K. Shammas and Lawrence K. Wang.

termed a surface phenomenon. Adsorption from water onto activated carbon can occur as a result of two separate properties of the water-activated carbon system, or some combination of the two: (1) the low solubility of a particular solute in the water and (2) a high affinity of a particular solute in the water for the activated carbon. According to the most generally accepted concepts of adsorption, this latter surface phenomenon may be predominantly one of electrical attraction of the solute to the carbon, of van der Wads attraction, or of a chemical nature.

There are essentially four consecutive steps in the adsorption of dissolved materials in water and wastewater by the adsorbent (e.g., granular activated carbon (GAC)). The first step is the *transport* of the solute from the bulk solution to the liquid film surrounding the adsorbent. The second step is the diffusion of the solute through the liquid film to the exterior of the adsorbent—*film diffusion*. The third step is the diffusion of the solute within the pores of the adsorbent—*pore diffusion*. The fourth and final step is *adsorption* of the solute on the interior surfaces bounding the pore and capillary spaces of the adsorbent.

There are several factors which can influence adsorption, including

1. The nature of the adsorbent itself
2. The nature of the material to be adsorbed, including its molecular size and polarity
3. The nature of the solution, including its pH and temperature
4. The contacting system and its mode of operation

Only when power input is low does transport of the adsorbate to the adsorbent become a controlling factor. With powdered carbon < 300 mesh (<0.08 mm) in size of separation, solution transport seems to be the rate-determining step at most reasonable stirring speeds. By contrast, intraparticle transport is normally the slowest and, therefore, rate-determining step in the uptake of organic substances, even in rapidly stirred systems.

A study of the observational data included in Fig. 20.1 suggests that the rate of adsorptive removal of responsive impurities is closely approximated by Eq. (20.1):

$$y/y_0 = 1 - (1 + nk_0 t)^{-1/n} \tag{20.1}$$

where y/y_0 is the proportion of adsorbate removed in t days; k_0 is the initial, or starting, rate of reaction or reaction velocity in day^{-1}; and n is the response coefficient of the substances to be removed or adsorbed. In this case, the rate process is closely approximated by the above purification equation as follows:

$$y/y_0 = 1 - (1 + 3.6t)^{-1/4} \tag{20.2}$$

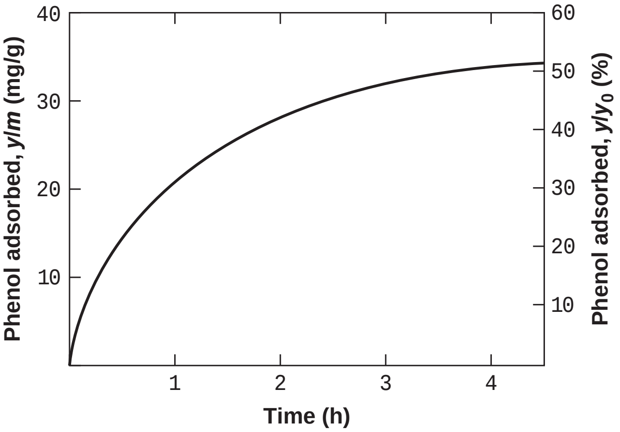


Figure 20.1 Rate of adsorption of phenol by granular activated carbon [dosage 35 mg/L of 100/140 mesh (0.13 mm) granular carbon. Initial phenol concentration 2.3 mg/L; 4-h concentration 1.15 mg/L] (*Source*: US EPA).

20.2.2 Conditions of Equilibrium

Positions of equilibrium in adsorption are given by adsorption isotherms, which relate the quantity adsorbed per unit of adsorbent to the concentration of adsorbate (Fig. 20.2). From an isotherm test, it can be determined whether or not a particular degree of organic removal can be effected by adsorption alone. It will also show the approximate adsorptive capacity of the carbon for the application. Isotherm tests also afford a convenient means of studying the effects of pH and temperature on adsorption. Isotherms put a large amount of data into concise form for ready evaluation and interpretation. Isotherms obtained under identical conditions using the same test solutions for two test carbons can be quickly and conveniently compared to reveal the relative merits of the carbons.

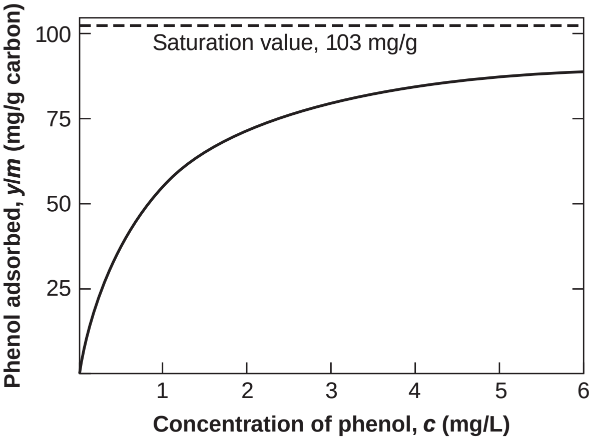


Figure 20.2 Equilibrium adsorption of phenol by granular activated carbon [room temperature, 50/60 mesh (0.273 mm) granular carbon] (*Source*: US EPA).

Three mathematical formulations of these isotherms are in common use for adsorption from aqueous solution. One, *the Freundlich isotherm* equation, is

$$\frac{y}{m} = Kc^{1/n} \tag{20.3}$$

in which y is the quantity adsorbed, m is the mass of the adsorbent, y/m is the quantity adsorbed by a unit weight of adsorbent, c is the equilibrium concentration of adsorbate in solution, and K and n are empirical constants for each adsorbent–adsorbate pair at a given temperature. Values of n are normally greater than unity, suggesting that adsorption is relatively more efficient at low concentrations. The logarithmic form of the Freundlich equation,

$$\ln\left(\frac{y}{m}\right) = \ln K + \frac{1}{n}\ln c \tag{20.4}$$

indicates a linear variation of ln (y/m) with ln c. Accordingly, logarithmic plots of adsorption data (Fig. 20.3a) can be used for correlation and interpolation and for evaluation of K and n.

The second equation, the *Langmuir isotherm*, is based on assumptions of a monomolecular layer of adsorbent, uniformity of adsorbent surface, and no interaction between adsorbate molecules. It has the form

$$\frac{y}{m} = \frac{abc}{1 + ac} \tag{20.5}$$

The constants a and b, although they have theoretical significance within the limits of the assumptions, are usually determined empirically. The equation predicts a linear variation of y/m with c (Eq. 20.6) at low concentrations when $ac \ll 1$:

$$y/m = abc \tag{20.6}$$

and a limiting adsorptive capacity (Eq. 20.7), when $ac \gg 1$ at high concentrations:

$$y/m = b \tag{20.7}$$

The Langmuir equation can be converted into linear form in a number of ways. A convenient representation for evaluating the limiting adsorptive capacity, b, is by Eq. (20.8). Through the line's slope and y-intercept (Fig. 20.3b) we can obtain b and a, which are constants for each adsorbent/adsorbate pair at a given temperature:

$$\frac{1}{y/m} = \frac{1}{b} + \frac{1}{ab}\left(\frac{1}{c}\right) = \frac{1}{b}\left[1 + \frac{1}{a}\left(\frac{1}{c}\right)\right] \tag{20.8}$$

The third isotherm is based on assumptions of a multilayer adsorbent, that is, some molecules are adsorbed on already adsorbed molecules and the Langmuir isotherm is no more valid. Stephan Brunauer, Paul Emmett, and Edward Teller developed the *BET model isotherm* that takes the multilayer possibility into account. Their theory is called BET theory, after the initials in their last names. The BET isotherm is represented by

$$\frac{y}{m} = \frac{Bcb}{(c_s - c)[1 + (\mathrm{B} - 1)(\mathrm{c}/c_s)]} \tag{20.9}$$

in which c_s is the saturation concentration of adsorbate in solution and B and b are constants. The BET equation (20.9) can be converted into linear form (Fig. 20.3c) to facilitate its application to experimental data as in Eq. (20.10):

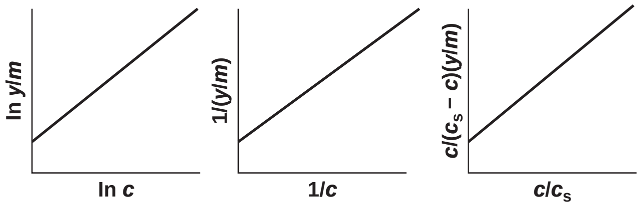
$$\frac{c}{(c_s - c)(\mathrm{y}/m)} = \frac{1}{Bb} + \left(\frac{B-1}{Bb}\right)\left(\frac{c}{c_s}\right) \tag{20.10}$$

A plot of this term $\frac{c}{(c_s-c)(y/m)}$ of Eq. (20.10) against $\left(\frac{c}{c_s}\right)$ yields a straight line having an intercept $\frac{1}{Bb}$ and a slope of $\left(\frac{B-1}{Bb}\right)$.

The extent of adsorption of a particular adsorbate on an adsorbent is also dependent on the temperature and on the presence of other adsorbates in the solution. Adsorption usually decreases with increasing temperature, but its extent is not readily predictable for adsorption from aqueous solutions and has not been found to be significant for the range of temperatures ordinarily encountered in water treatment. If other adsorbates are present, they tend to decrease adsorption of a particular adsorbate by competing for space on the surface.

As a unit operation, adsorption is generally not confined to the removal of single substances. However, the interactions at play may be competitive. Rapid adsorption of one substance conceivably reduces the number of sites remaining open to other substances. An example of mutual inhibition in the sorption of competing adsorbates is the less effective transport of phenolic odors from water that contains much organic color. The overall rate of removal of organic matter may drop off in such circumstances.

Generally speaking, power is dissipated usefully in treatment processes (1) for the purpose of increasing or decreasing the size or concentration of substances within a given phase or (2) for transferring substances from one phase to another: from the solid to the liquid phase or vice versa and from the liquid to the gas phase or vice versa. Because quantitative change is effected by conjunction contact and because transfer or phase change is promoted by interfacial contact, the creation, renewal, or energizing of needed conjunctions



(*a*) Freundlich (*b*) Langmuir (*c*) BET

Figure 20.3 Linear plots of adsorption isotherms (*Source*: US EPA): (a) Freundlich, (b) Langmuir, and (c) BET.

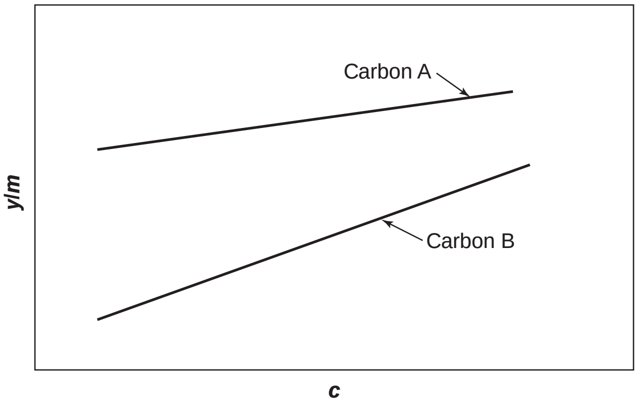


Figure 20.4 Adsorption isotherm, carbons A and B (*Source*: US EPA).

or interfaces has become a governing component of modern treatment procedures.

20.2.3 Interpretation of Adsorption Isotherms

Figures 20.4 and 20.5 are presented to illustrate the interpretation of adsorption isotherms. In Fig. 20.4, the isotherm for Carbon A is at a high level and has only a slight slope. This means that adsorption is large over the entire range of concentrations studied. The fact that the isotherm for Carbon B in Fig. 20.4 is at a lower level indicates proportionally less adsorption, although adsorption improves at higher concentrations over that at low concentrations. An isotherm having a steep slope indicates that adsorption is good at high concentrations, but much less at low concentration. In general, the steeper the slope of its isotherm, the greater the efficiency of a carbon in column operation. In Fig. 20.5, Carbon D is better suited to countercurrent column operation than Carbon C. It has a higher capacity at the influent concentration, or more reserve capacity. Carbon C in Fig. 20.5 would be better than Carbon D for batch treatment.

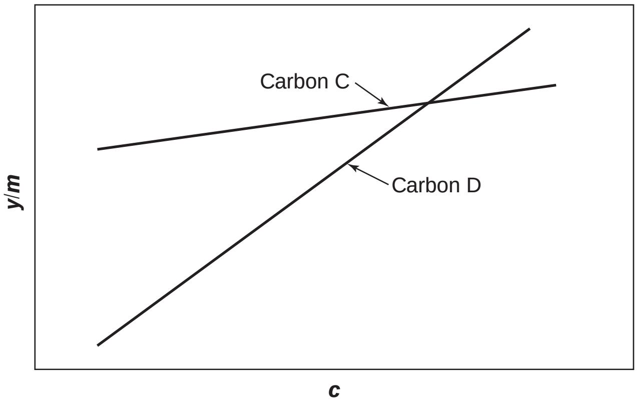


Figure 20.5 Adsorption isotherm, carbons C and D (*Source*: US EPA).

20.3 CHARACTERISTICS OF ADSORBENTS

Adsorbents are used usually in the form of spherical pellets, rods, moldings, or monoliths with hydrodynamic diameters between 0.5 and 10 mm. They must have high abrasion resistance, high thermal stability, and small pore diameters, which results in higher exposed surface area and hence high surface capacity for adsorption. The adsorbents must also have a distinct pore structure, which enables fast transport of solutes. Most industrial adsorbents fall into one of three classes:

1. Oxygen-containing compounds: These are typically hydrophilic and polar, including materials such as silica gel and zeolites.
2. Carbon-based compounds: These are typically hydrophobic and nonpolar, including materials such as activated carbon and graphite.
3. Polymer-based compounds: These are polar or nonpolar functional groups in a porous polymer matrix.

20.3.1 Silica Gel

Silica gel is a chemically inert, nontoxic, polar, and dimensionally stable (<400°C) amorphous form of SiO_2. It is prepared by the reaction between sodium silicate and sulfuric acid, which is followed by a series of after-treatment processes such as aging and pickling. These after-treatment methods results in various pore size distributions. Silica is used for drying of process air (e.g., oxygen, natural gas) and adsorption of heavy (polar) hydrocarbons.

20.3.2 Activated Carbon

Activated carbon removes organic contaminants from water by the process of adsorption or the attraction and accumulation of one substance on the surface of another. In general, high surface area and pore structure are the prime considerations in adsorption or organics from water; whereas, the chemical nature of the carbon surface is of relatively minor significance. GACs typically have surface areas of 500–1400 m^2/g. Activated carbon has a preference for organic compounds and, because of this selectivity, is particularly effective in removing organic compounds from aqueous solution.

Activated carbon is a highly porous, amorphous solid consisting of microcrystallites with a graphite lattice, usually prepared in small pellets or a powder. It is nonpolar and cheap. One of its main drawbacks is that it is combustible.

Activated carbon is manufactured by a process consisting of raw material dehydration and carbonization followed by activation. The starting material is dehydrated and carbonized by slowly heating in the absence of air, sometimes using a dehydrating agent such as zinc chloride or phosphoric acid. Excess water, including structural water, must be driven from the organic material. Carbonization converts this organic material to primary carbon, which is a mixture of ash (inert inorganics), tars, amorphous carbon, and crystalline carbon (elementary graphitic crystallites). Non-carbon elements (H_2 and O_2) are removed in gaseous form and the freed elementary carbon atoms are grouped into oxidized crystallographic formations. During carbonization, some decomposition products or tars will be deposited in the pores, but will be removed in the activation step.

Activation is essentially a two-phase process requiring burn-off of amorphous decomposition products (tars), plus enlargement of pores in the carbonized material. Burn-off frees the pore openings, increasing the number of pores, and activation enlarges these pore openings. Activated carbon can be manufactured by two different procedures: physical activation and chemical activation. Although both processes are widely used, physically activated carbons are utilized in water and wastewater treatment while chemically activated carbons are utilized elsewhere—such as the recovery of solvents.

Carbonization and activation are two separate processes. The methods of carbonization differ, and the method used will affect activation and the final quality of the carbon. The essential steps in carbonization are as follows:

1. Dry the raw material at temperatures up to 170°C.
2. Heat the dried material above 170°C causing degradation with evolution of CO, CO_2, and acetic acid.
3. Exothermally decompose the material at temperatures of 270–280°C with formation of considerable amounts of tar, methanol, and other by-products.
4. Complete the carbonization process at a temperature of 400–600°C, with a yield of approximately 80% primary carbon.

The carbonized intermediate product is then treated with an activating agent such as steam or carbon dioxide (steam is most widely used). Steam, at temperatures of 750–950°C, burns off the decomposition products exposing pore openings for subsequent enlargement. All pores are not plugged with amorphous carbon; therefore, some pores are exposed to the activating agent for longer periods of time. Exposure to the activating agent results in the widening of existing pores and development of the macroporous structure. Figure 11.8 in Chapter 11 shows a porous carbon particle after activation. An activated carbon particle has many macrochannels and microchannels, which provide a large surface area for adsorption.

20.3.3 Polymeric Adsorbents

A polymeric adsorbent is defined as a *macroporous* or *macroreticular* polymeric material that has similar properties to ion exchange resin, but has no functional ionic group. These polymeric adsorbents are hard, durable, insoluble spheres of high surface area and porosity. They are also available in a variety of polarities.

In general, adsorbents are solids that possess high specific surfaces, usually well above 5 m^2 of exposed surface area per gram of solid. Adsorbents fall into two major physical classes, porous and nonporous. The porous adsorbents consist of particles that are usually large (greater than 50 mesh) and the high surface area is a result of pores of varying diameters that "permeate" the particles. The diameters of these pores are larger than molecular distances. Nonporous adsorbents are usually finely divided solids (less than 10 μm), and the high surface area of such materials is due to the fine state of subdivision that is achieved by various techniques such as grinding and precipitation.

The terms microporous and macroporous, usually used in adsorption terminology, refer to those pores less than 20 Å and greater than 200 Å, respectively. Pores of diameters between 20 Å and 200 Å are referred to as *transitional pores* and *polymeric adsorbents*. This classification could, of course, also be applied to the macroreticular ion exchange resins; however, the terminology would not distinguish those pores that are part of the organic gel structure of the macroreticular ion exchange resins and polymeric adsorbents.

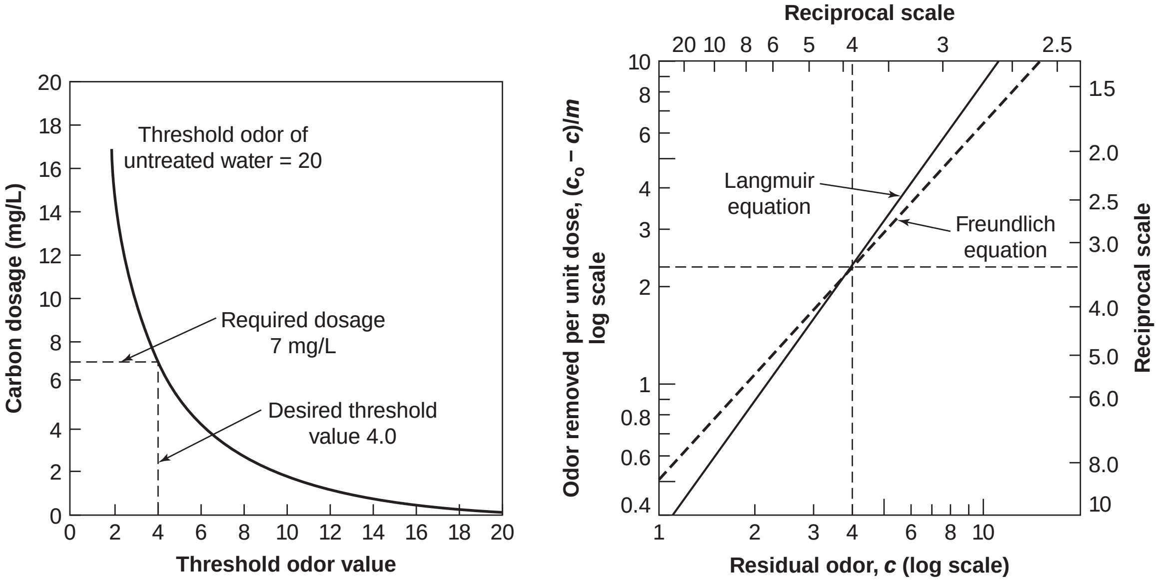
Polymeric adsorption, also referred to as *resin adsorption*, is an adsorption process involving the use of granular polymeric adsorbents (GPA) to extract and, in some cases, recover dissolved organic solutes from aqueous water and wastewater. A polymeric adsorption system is very similar to a GAC adsorption system in terms of adsorption theory, principles, and process equipment. The former uses GPA as the granular adsorbents, while the latter uses GAC.

A polymeric adsorption system is also very similar to an ion exchange process system in terms of process equipment and polymeric resins used in the process. However, the former (polymeric adsorption) is an adsorption process using nonionic polymeric resins to be the adsorbents for removing pollutants, while the latter (ion exchange) is an ion exchange process using ionic polymeric resins (cationic ion exchange resins or anionic exchange resins) to be the ion exchange agents for removing pollutants

20.3.4 Carbon Properties Relating to Adsorption

At one time the adsorptive capacity of different carbons for pure phenol was made the basis for their general comparison. However, the *phenol value* or amount of carbon in mg/L required to reduce 100 μg/L of phenol by 90% does not necessarily reflect the relative efficiency of a given carbon in removing substances other than phenol. Direct testing against the waters to be treated has therefore become the preferred procedure. Commercial carbons possess a phenol value of 15–30.

Although the Freundlich, Langmuir, and BET isotherms (Eqs. 20.4, 20.8, and 20.10) can be used in comparative tests to identify the magnitudes of the pertinent adsorption coefficients of a given adsorbent, in practice instead, experimental



(*a*) (*b*)

Figure 20.6 Determination of carbon dosage for odor control: (a) arithmetic plotting of experimental results; (b) double logarithmic plotting for Freundlich equation and reciprocal plotting for Langmuir equation (After Fair et al. 1971).

carbon dosage is plotted against observed residual concentrations of the sorbate, and required dosages are read from curves fitted by eye. Either arithmetic or double logarithmic plots are used (Fig. 20.6 and Example 20.1).

Because adsorption is a surface phenomenon, the ability of activated carbon to adsorb large quantities of organic molecules from solution stems from its highly porous structure, which provides a large surface area. Carbon has been activated with a surface area yield of some 2,500 m^2/g, but 1000 m^2/g is more typical.

Particle size is generally considered to affect adsorption rate, but not adsorptive capacity. The external surface constitutes a small percent of the total surface area of an activated carbon particle. Since adsorption capacity is related to surface area, a given weight of carbon gains little adsorptive capacity upon being crushed to smaller size.

Activated carbons are classified according to their form: for example, powdered or granular; and according to their use: for example, water or wastewater purification, sugar decolorizing, and liquid or gas phase solvent extraction. GACs are those which are larger than approximately US Sieve Series No. 50, while PACs are those which are smaller. Properties of several commercially available granular carbons are presented in Table 20.1.

PAC is normally crushed to such size that 50% will pass a 300 mesh sieve and 95% a 200 mesh sieve—that is, 50–75 μ, respectively. The adsorptive capacity is high because a pound of finely divided activated carbon contains some 10^{13} particles and a solid cubic foot of carbon particles presents a combined pore and external surface of about 10 mi^2 (29.5 km^2) to the water in which it is suspended.

PAC is an inexpensive treatment option (capital cost) that can typically be added to an existing treatment system's infrastructure. This flexibility makes PAC an attractive option for short-term treatment responses to poor water conditions. It is particularly useful to treat taste and color deficiencies.

PAC works quickly and efficiently but it is limited to lower removals than GAC and becomes expensive if it must be used on a continuous basis. When the process is complete the powdered carbon must be removed, usually by filtration.

Head loss in the carbon contactor is an important design consideration and is affected by the carbon particle size. The suspended solids concentration in the water or wastewater to be treated by the carbon will also affect the head loss and will thereby be a factor in selection of carbon particle size.

Like PAC, GAC also attracts not only known contaminants, but also mostly harmless, naturally dissolved organic matter. Therefore, careful monitoring is needed to ensure that enough carbon remains active to adsorb all contaminants. Particulates may also clog systems and compromise their effectiveness. GAC systems have a higher capital cost but are capable of accomplishing higher levels of removal, and their operating costs (mostly the cost of replacing spent GAC) are lower if removal is required on a continuous basis.

20.4 ADSORPTION OF ODORS AND TASTES

Within the range and intensity of odors and tastes generally encountered in water and wastewater, man's sensory responses to these stimuli obey the Weber–Fechner law. According to this law, discrimination between odor and taste stimuli is confined to threshold ratios, not to threshold differences. The common ratio of successive dilutions is approximately 1.4. It follows that man's discrimination between odor and taste intensities is relatively poor in comparison with his much more acute senses of sight and hearing.

Table 20.1 Properties of commercially available carbon

	ICI America Hydrodarco 3,000	Calgon Filtrasorb 300 (8×30)	Westvaco Nuchar WV-L (8×30)	Witco 517 (12×30)
Physical properties				
Surface area (m^2/g)	600–650	950–1,050	1,000	1,050
Apparent density (g/cm^3)	0.43	0.48	0.48	0.48
Density, backwashed and drained (lb/ft^3)	22	26	26	30
Real density (g/cm^3)	2.0	2.1	2.1	2.1
Particle density (g/cm^3)	1.4–1.5	1.3–1.4	1.4	0.92
Effective size (mm)	0.8–0.9	0.8–0.9	0.85–1.05	0.89
Uniformity coefficient	1.7	1.9 or less	1.8 or less	1.44
Pore volume ($cm^{3/}g$)	0.95	0.85	0.85	0.60
Mean particle diameter (mm)	1.6	1.5–1.7	1.5–1.7	1.2
Specifications				
Sieve size (US std. series)				
Larger than no. 8 (max. %)	8	8	8	–
Larger than no. 12 (max. %)	–	–	–	5
Smaller than no. 30 (max. %)	5	5	5	5
Smaller than no. 40 (max. %)	–	–	–	–
Iodine no.	650	900	950	1,000
Abrasion no., minimum	–	70	70	85
Ash (%)	–	8	7.5	0.5
Moisture as packed (max. %)	–	2	2	1

Source: US EPA.

The odors of municipal and other decomposable wastewaters are most pronounced when they, or the solids removed from them, become septic. However, there are industrial wastes with characteristic odors of their own that are not changed by septicity. Proper planning and management of water supplies and wastewater works will minimize odor and taste troubles.

To describe the adsorption of odors and tastes from water in terms of threshold values, the Freundlich and Langmuir equations (Eqs. 20.3–20.8) are normally translated into the following forms:

$$\frac{(c_o - c)}{m} = Kc^{1/n} \tag{20.11}$$

$$\ln\left[\frac{c_o - c}{m}\right] = \ln K + \frac{1}{n}\ln c \tag{20.12}$$

$$\frac{(c_o - c)}{m} = \frac{abc}{1 + ac} \tag{20.13}$$

$$\frac{1}{(c_o - c)/m} = \frac{1}{b} + \frac{1}{ab}\left(\frac{1}{c}\right) \tag{20.14}$$

Here c_o is the threshold odor or taste of the water to be treated, c is the residual threshold value, m is the concentration of the adsorbent, and K, n, a, and b are coefficients, b being the limiting adsorptive capacity of the adsorbent. Generalization of observed removals is important in research and in the identification of attainable adsorption.

Reduction of tastes and odors in water supplies by adsorption of the offending substances on activated carbon is probably the most important direct use of adsorption in water treatment. Columns or beds of GAC are employed (1) for concentrating organic pollutants from water for purposes of analysis or (2) for removal of the pollutants. Some of the removal of color-producing substances and other pollutants from water during coagulation may be the result of adsorption. Indeed, certain processes make use of this phenomenon to reduce silica, fluoride, or radioactive substances. Beds of GAC can be operated either as *fixed* or *packed beds* or as *fluidized* or expanded beds. Packed beds normally accumulate solids faster than do fluidized beds.

EXAMPLE 20.1 DETERMINATION OF CARBON DOSAGE FOR ODOR REDUCTION USING THE FREUNDLICH ISOTHERM

Figure 20.6a shows an arithmetic plot of carbon dosage, m, in mg/L against observed threshold odor values c. The threshold odor of the untreated water is 20 and the required carbon dosage is read from the plot as 7 mg/L to reduce the odor to a threshold value of 4. Calculate the carbon dosage based on the Freundlich isotherm in Fig. 20.6b.

Solution:

In Fig. 20.6b a logarithmic plot of Freundlich isotherm identifies the magnitudes of K and n by scales of $y = (c_o - c)/m$ and $x = c$. Thus K is read at the intercept with the y-axis at $c = 1$ of the straight line best fitting the observed points, namely, $K = 0.50$. The magnitude of $1/n$ is read as the slope:

$$1/n = (\log 6.3 - \log 0.50)/(\log 10 - \log 1) = 1.10, \text{ or } n = 0.91.$$

Therefore, the required dosage is

$$\frac{(c_o - c)}{m} = Kc^{1/n} \tag{20.15}$$

$$m = \frac{(c_o - c)}{Kc^{1/n}} \tag{20.16}$$

$$m = (20 - 4)/(0.5 \times 1^{1.10}) = \mathbf{7\ mg/L}.$$

EXAMPLE 20.2 DETERMINATION OF CARBON DOSAGE FOR ODOR REDUCTION USING THE LANGMUIR ISOTHERM

Calculate the carbon dosage in Example 20.1 based on the Langmuir isotherm in Fig. 20.6b.

Solution:

In Fig. 20.6b for the Langmuir isotherm a reciprocal plot of $(c_o - c)/m$ against c makes $(1/ab)$ the slope of the straight line of best fit:

$$(1/ab) = (1/2 - 1/10)/(1/3.5 - 1/15) = 1.82 \text{ or } ab = 0.55 \text{ and}$$

$1/b$ from the relationship is

$$1/b = 1/[(c_o - c)/m] - (1/ab)(1/c) = 0.16 - 1.82 \times 0.1 = -0.022, \text{ or}$$

$$b = -45 \text{ and}$$

$$a = 0.55/b = -1.2 \times 10^{-2}.$$

Therefore, the required dosage is

$$(20 - 4)(1 - 1.2 \times 10^{-2} \times 4)/(0.55 \times 4) = \mathbf{7\ mg/L}.$$

Because $ac \ll 1$, linear variation of $(c_o - c)/m$ at low concentrations is supported.

20.5 PILOT CARBON COLUMN TESTS

Although the treatability of a particular water or wastewater by carbon and the relative capacity of different types of carbon for treatment may be estimated from adsorption isotherms, carbon performance and design criteria are best determined by pilot tests. Adsorption isotherms are determined in a batch test and the treatment of wastewater by GAC most often is effected in a continuous system involving packed beds similar to filtering operations in water treatment. Pilot tests provide much more accurate estimates of the performance that can be expected in a full-scale unit.

Information which can be obtained from pilot tests includes

1. Type of carbon
2. Contact time
3. Bed depth
4. Pretreatment requirements
5. Carbon dosage in terms of pounds carbon per million gallons of flow or pounds organic material removed per pound of carbon.
6. Breakthrough characteristics
7. Affect of biological activity including possible extension of the carbon capacity as well as potential deleterious effects such as generation of hydrogen sulfide
8. Head loss characteristics

Pilot carbon column tests are performed for the purpose of obtaining design data for full-scale plant construction. Pilot column tests make it possible to do the following things:

1. Compare the performance of two or more carbons under the same dynamic flow conditions.
2. Determine the minimum contact time required to produce the desired quality of carbon column effluent, which is the most important of all design factors.

3. Check the manufacturer's data for head loss at various flow rates through different bed depths.
4. Check the backwash flow rate necessary to expand the carbon bed for cleaning purposes.
5. Establish the carbon dosage required, which will determine the necessary capacity of the carbon regeneration furnaces and auxiliaries.
6. If the overall process or plant flow sheet has not been firmly established, check the effect of various methods of pretreatment (influent water quality) upon carbon column performance, carbon dosage and overall plant costs.
7. Evaluate the practical advantages and disadvantages that cannot be evaluated by reading the experiences of others, for alternates such as use of upflow or downflow carbon columns or the particle size of carbon to be used.

Figures 20.7 and 20.8 are schematic flow diagrams of upflow and downflow pilot carbon columns.

20.6 BREAKTHROUGH CURVE

Since granular carbon columns are dynamic systems, not only are the equilibrium adsorption properties of the carbon important, but also the rates of adsorption illustrated by Fig. 20.9. Figure 20.9 shows *breakthrough curves* of a carbon as obtained by passing a fluid containing an adsorbable substance through a packed bed of carbon. The concentration

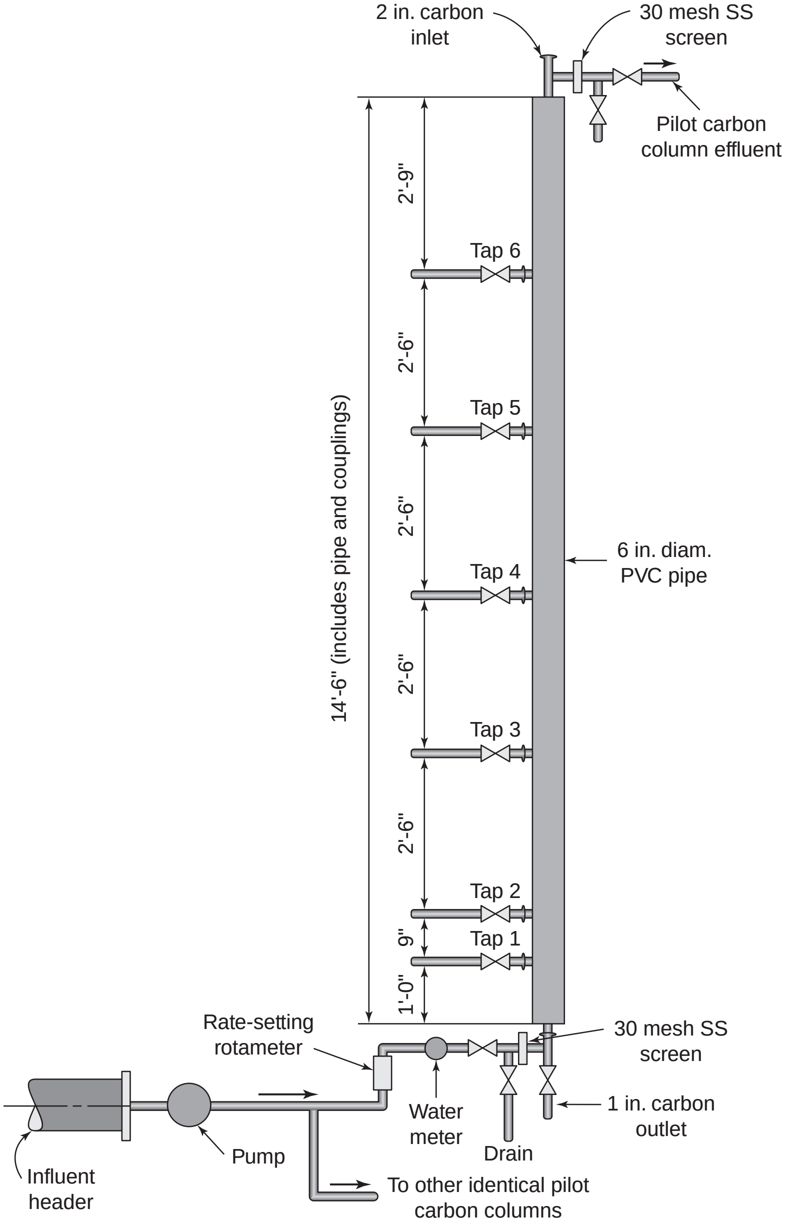


Figure 20.7 Upflow pilot carbon column (*Source*: US EPA). Conversion factors: $1' = 1$ ft = 0.3048 m; $1'' = 1$ in. = 25.4 mm.

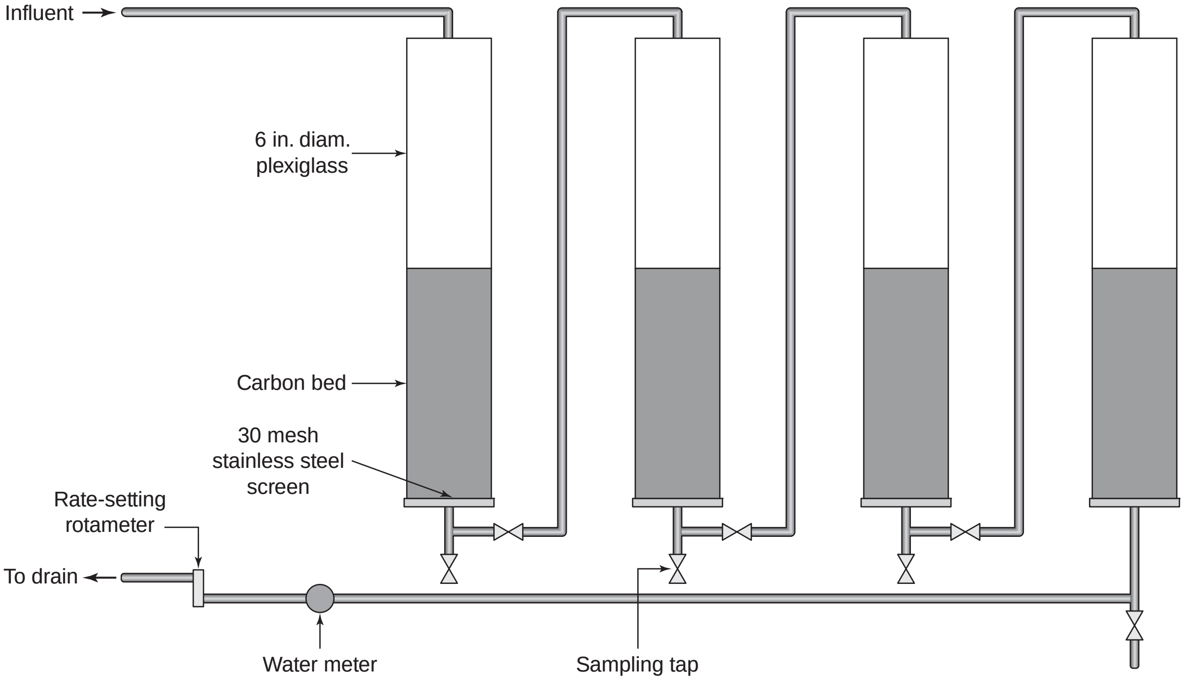


Figure 20.8 Downflow pilot carbon columns (*Source*: US EPA). Conversion factors: 1″ = 1 in. = 25.4 mm; 1 mesh = 1 opening/in. = 0.4 opening/cm.

of the adsorbable substance in the effluent stream from the bed is plotted against the volume of fluid passed through. In Fig. 20.9 the breakthrough curve for Carbon E is much steeper than that for Carbon F. This will occur when the rate of adsorption for Carbon E is much greater than that for Carbon F. Generally speaking, greater rates for adsorption are desired for maximum efficiency of the carbon. The carbon with the steepest breakthrough curve will, therefore, have the longest service life even though the capacity of the carbons at equilibrium may be the same or even higher. Thus, for selecting carbons and designing adsorption systems, the rate of adsorption as reflected in the breakthrough curves is an important consideration.

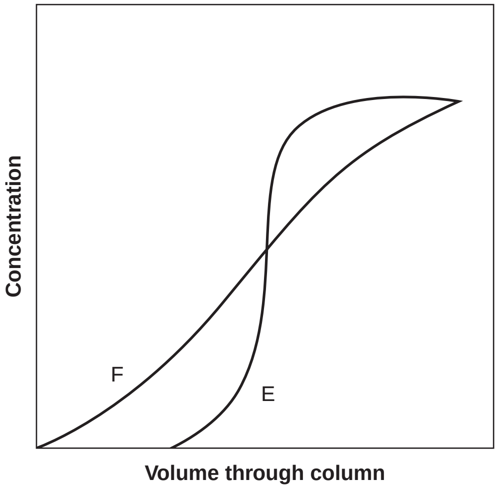


Figure 20.9 Typical breakthrough curve (*Source*: US EPA).

In actual use, carbon column operation is discontinued when the effluent concentration reaches the acceptable allowable value, c_a, usually at 5% of the original concentration, c_o that is, $c_a = 5\%$ of c_o. The carbon column is considered to be exhausted when the effluent concentration, c_e is equal to 95% of the original concentration $c_{o,}$ that is, $c_e = 95\%\ c_o$ (see Fig. 20.14 for Example 20.3).

20.7 PROCESS TECHNOLOGY

20.7.1 Types of Contact Beds

The alternatives for carbon contacting systems include

1. Downflow or upflow of water through the carbon bed
2. Series or parallel operation (single or multistage)
3. Pressure or gravity operation in downflow contactors
4. Packed or expanded bed operation in upflow contactors
5. Materials of construction and configuration of carbon vessel
   (a) Steel or concrete
   (b) Circular or rectangular cross-section

Upflow beds have an advantage over downflow beds in the efficiency of carbon use because they can more closely approach continuous countercurrent contact operation. Countercurrent operation results in the minimum use of carbon, or the lowest carbon dosage rate. Upflow beds may be designed to allow addition of fresh carbon and withdrawal of spent carbon while the column remains in operation. When these operations are conducted almost continuously, the bed may be referred to as a pulsed bed. A pulsed bed may be either an upflow packed bed or an upflow expanded bed. Upflow packed beds require a high clarity influent, which may be considered a disadvantage to their use. Upflow expanded beds have the advantages of being able to treat flows relatively high in suspended solids and of being able to use finer carbon (which reduces the required contact time) without excessive head losses. Where upflow packed beds typically use 8 × 30 mesh carbon, upflow expanded beds typically use 12 × 40 mesh.

Provision must be made to periodically and thoroughly backwash downflow beds to relieve the pressure drop associated with the accumulation of suspended solids. Continued operation of a downflow bed for several days without backwashing may compact or foul the bed sufficiently to make it more difficult to expand the bed during backwash without the use of an excessive quantity of backwash water, that is, more than 5% of the product water. Upflow beds may be flushed through a simple well screen inlet–outlet system. Downflow beds require a false bottom support system, backwash facilities, and controls similar to those used in waterworks practice for sand filters.

The use of pressure vessels for carbon contactors will increase the flexibility of operation since it will allow the system to be operated at higher pressure losses. This may allow the carbon contact system to operate during upsets or variations in the flow. Gravity contactors may be more economical since concrete and common wall construction may be utilized.

The *fixed-bed* operation (see Fig. 20.10), which is in essence countercurrent operation, is the most effective and efficient way of using carbon and other adsorbents. In the course of each operating cycle, entrant water then comes into contact with the adsorbent along a gradient of mounting residual activity until the most active carbon gives a final polish to the effluent water while the adsorption reaction is driven to ultimate saturation of the carbon.

GAC can replace other filtering materials in structures not unlike rapid filters. Beds of GAC can in fact be made to perform as both filters and adsorbents. However, activated carbon filters must be somewhat deeper than sand filters, even though they may be operated at higher rates of flow per unit area of bed. For adsorption, the rate of flow per unit volume rather than per unit area of bed is the important parameter in practice. Carbon grain sizes range from 0.8 to 3 mm or 14/40 mesh, and rates of 2.0–0.4 gpm/ft^3 (44–9 L/min/m^3) are obtained when beds 5–10 ft (1.5–3 m) deep filter water at rates of 10–4 gpm/ft^2 (400–160 L/min/m^2). Associated contact times are 2–8 min, and efficiencies of 50–90% collection of carbon-chloroform extractables (CCE) are obtained. Because of the many variables involved, pilot-plant scale tests are normally obligatory.

20.7.2 Design Criteria of Carbon Beds

The flow rate and bed depth necessary for optimum performance will depend upon the rate of adsorption. The general range of flow rates (or hydraulic loading) is 2–10 gpm/ft^2 (80–400 L/min/m^2) of cross-sectional area. Bed depths are usually 10–30 ft (3–9 m).

Both theoretical analysis and experimental data support the contention that there are critical velocities for liquids passing through porous beds which change the nature of the resistance to diffusion. At low velocities, the solute content of the stagnant film surrounding the adsorbent particles may become depleted more rapidly than the solute can be replaced by diffusion from the main body of the liquid. Thus, the diffusional resistance across the film is controlling. As the velocity is increased, the point will be reached where the controlling effect will be the inability of the adsorbent material to remove the solute from solution as rapidly as it is transported to the surface from the main body of the stream. Within the range of loadings of 2–10 gpm/ft^2 (80–400 L/min/m^2), several studies have found that velocity is not a limiting factor.

Hydraulic loading has an additional effect on carbon column operation. Increasing flow rates through the carbon will cause increasing head losses. Head loss is dependent on the flow rate and carbon particle size. This relationship for clean water passing through a bed of clean carbon is expressed in the formula:

$$\Delta P = \frac{KvVL_c}{D_p^2 D_c} \quad \text{(US customary system)} \qquad \textbf{(20.17a)}$$

where

ΔP = pressure drop, in.

K = constant

v = viscosity, centipoise

V = volume passing per unit time or flow rate, gpm

L_c = column length or bed depth, ft

D_p = mean particle diameter, mm

D_c = column diameter, in.

An equivalent metric equation for pressure drop calculation is presented below:

$$\Delta P = (560)\frac{KvVL_c}{D_p^2 D_c} \quad \text{(SI system)} \qquad \textbf{(20.17b)}$$

where

ΔP = pressure drop, mm

K = constant

v = viscosity, centipoise

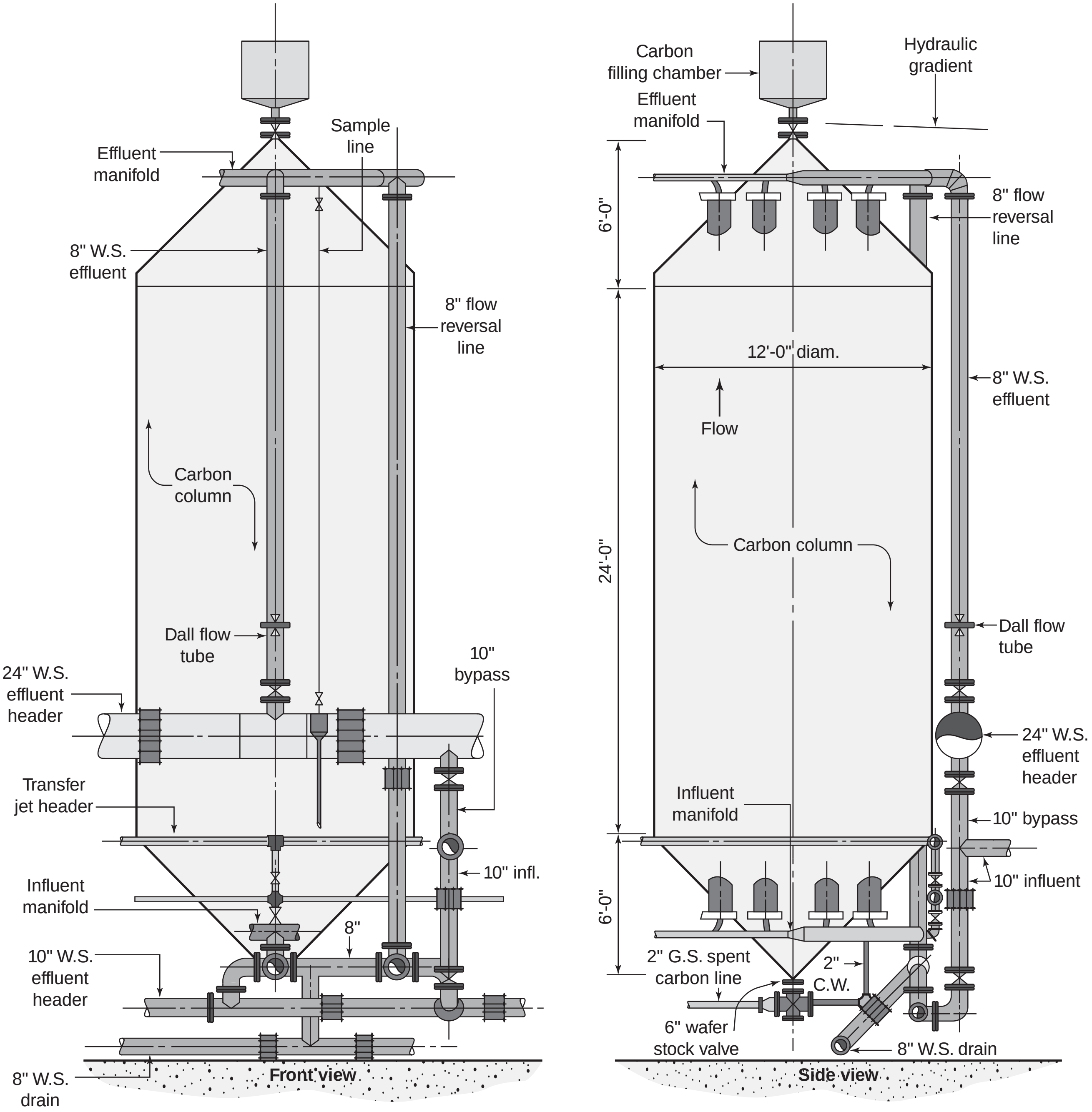


Figure 20.10 Upflow countercurrent carbon column (*Source*: US EPA). Conversion factors: 1″ = 1 in. = 25.4 mm; 1′ = 1 ft = 0.3048 m.

V = volume passing per unit time or flow rate, L/min

L_c = column length or bed depth, m

D_p = mean particle diameter, mm

D_c = column diameter, mm

Hydraulic head loss is then related directly to flow rate and inversely related to particle size. Figure 20.11 illustrates the increasing pressure drop with increasing hydraulic loading for different sized carbons from different manufacturers, operated in a downflow mode. Because of the more favorable head loss characteristics, 8 × 30 mesh carbon is often preferred for downflow beds while 12 × 40 mesh carbon may be preferable for upflow beds because a lower upflow velocity is required for expansion.

The head loss for a given hydraulic loading with wastewater feed must be determined by pilot testing. Since head loss development is such an important consideration in the design

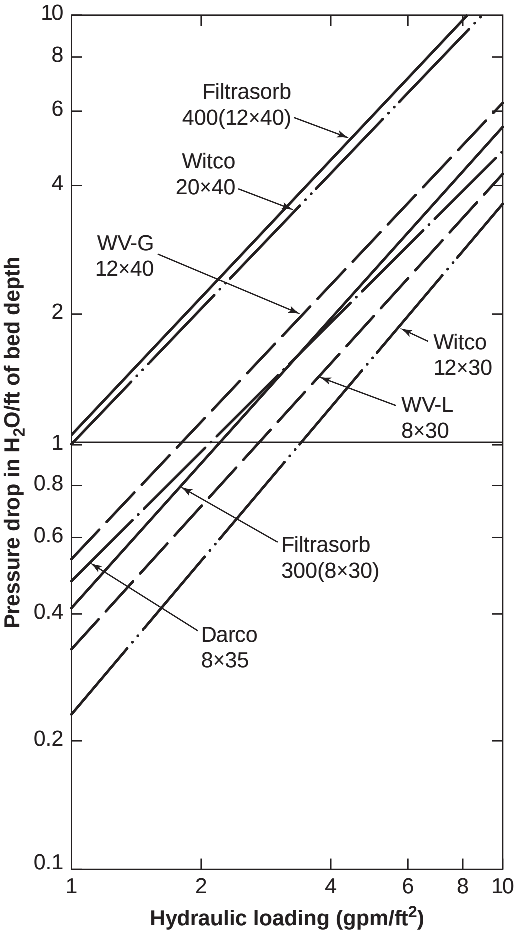


Figure 20.11 Pressure drop versus hydraulic loading (*Source*: US EPA). Conversion factors: 1 gpm/ft^2 = 40.7 L/min/m^2; 1 in./ft = 83.33 mm/m.

Table 20.2 General design parameters for carbon beds

Design parameter	Value
Influent suspended solids	<5 mg/L
Carbon requirement	200–400 lb carbon/MG
Carbon size	
Downflow	8 × 30 (0.8 × 3 mm)
Upflow	12 × 40 upflow (0.6 × 2 mm)
Hydraulic loading	2–10 gpm/ft^2
Contact time (empty bed basis)	10–50 min
Organics removal	0.2–0.8 lb COD/lb carbon
Backwash rate	10–20 gpm/ft^2
Backwash time	10–15 min
Backwash expansion	10–50%
Column depth	10–30 ft
Column diameter	3–12 ft
Contactor configuration	Gravity or pressure vessels Steel or concrete construction
Flow configuration	Upflow or downflow One stage or multistage

Source: US EPA.
Conversion factors: lb/MG = 1.72 kg/ML; 1 gpm/ft^2 = 40.7 L/min/m^2, 1 lb/lb = 1 kg/kg; 1 ft = 0.3048 m.

of a carbon bed, hydraulic loading cannot be discussed in isolation from several other design factors. If an excessive rate of head loss development (due to a high hydraulic loading) is anticipated, an upflow bed should be given consideration. The choice of gravity versus pressurized flow may also be influenced by the anticipated rate of head loss development. Very high hydraulic loadings are practical only in pressurized systems. Gravity flow in downflow beds is considered practical only at hydraulic loadings less than about 4 gpm/ft^2. The general range of design criteria used in carbon treatment systems is shown in Table 20.2.

Upflow expanded beds should be considered when high head loss is expected. At low flow rates, the particles are undisturbed and the bed remains fixed. As the flow rate is increased, however, a point is reached where all particles no longer remain in contact with one another, and the carbon bed is expanded in depth. The flow rate required for initial expansion of the bed is accompanied by a sizable increase in head loss. As the flow rate is increased, there is further expansion of the bed. Flow rates required for further expansion of the bed are accompanied by lesser increases in head losses.

20.7.3 Design of an Adsorption Bed

The approach to the design of adsorption beds is based on the kinetic approach suggested by Thomas. His derived kinetic expression is as follows:

$$\frac{c}{c_0} = \frac{1}{1 + e^{\frac{k_1}{Q}(q_0 M - c_0 V)}} \tag{20.18}$$

where

c = effluent solute concentration

c_0 = influent solute concentration

k_1 = rate constant, L/h-kg (gal/h-lb)

q_0 = maximum solid-phase concentration of the sorbed solute, kg/kg (lb/lb)

M = mass or weight of adsorbent, kg (lb)

V = throughput volume (total volume treated from breakthrough curve), L (gal)

Q = water or wastewater flow rate, L/h (gal/h)

Rearranging Thomas equation we get

$$\frac{c_0}{c} = 1 + e^{\frac{k_1}{Q}(q_0 M - c_0 V)} \tag{20.19}$$

$$\ln\left(\frac{c_0}{c} - 1\right) = \frac{k_1 q_0 M}{Q} - \frac{k_1 c_0}{Q} V \tag{20.20}$$

This equation plots, log $\left(\frac{c_0}{c}-1\right)$ versus V as a straight line (Fig. 20.12) with

$$\text{Intercept} = \frac{K_1 q_0 M}{Q} \qquad \textbf{(20.21)}$$

and

$$\text{Slope} = \frac{k_1 c_0}{Q} \qquad \textbf{(20.22)}$$

Note that the following three parameters should be the same for pilot and full-scale columns

1. The design reaction constant k_1
2. The design maximum solid-phase concentration q_0
3. The breakthrough time T_B

The data required for the solution of the Thomas equation can be obtained from the pilot granular carbon column test discussed earlier. The methodology is clarified in the following example.

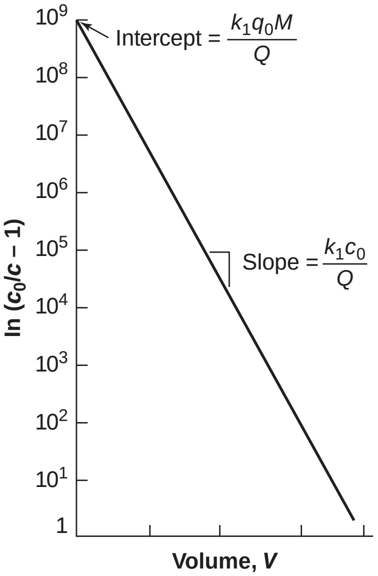


Figure 20.12 Semilog plot for the determination of intercept and slope of column pilot test.

EXAMPLE 20.3 DESIGN OF AN ADSORPTION CARBON BED

Water having a total organic carbon (TOC) of 200 mg/L is to be treated by a granular carbon column at a water flow, $Q = 150\ \text{m}^3$/day. The minimum TOC removal should not be less than 95%. The data obtained from a pilot column breakthrough test is as follows:

Total volume, V (L)	TOC concentration (mg/L)
400	9
1,000	11
1,300	8
1,900	9
2,300	30
2,500	100
2,700	165
2,930	190
3,120	200

The size and water flow rate for the pilot column are as follows:

Inside diameter, 9.50 cm
Height of carbon bed, 1.04 m
Mass of carbon, 2.98 kg
Water flow rate, 12.39 L/h
Unit flow rate, 0.486 L/s-m^2
Packed carbon density, 400 kg/m^3

Determine the following:

1. The design reaction constant k_1
2. The design maximum solid-phase concentration q_0

3. The breakthrough time T_B
4. The breakthrough volume V_B
5. The mass of carbon M
6. Diameter and length of carbon bed
7. Weight of carbon required per unit volume treated

Solution:

The data from the breakthrough test is plotted in Fig. 20.13. From the data we obtain

The allowable effluent concentration, $c_a = 5\% \times 200 = 10$ mg/L.
The exhausted TOC concentration, $c_e = 95\% \times 200 = 190$ mg/L.

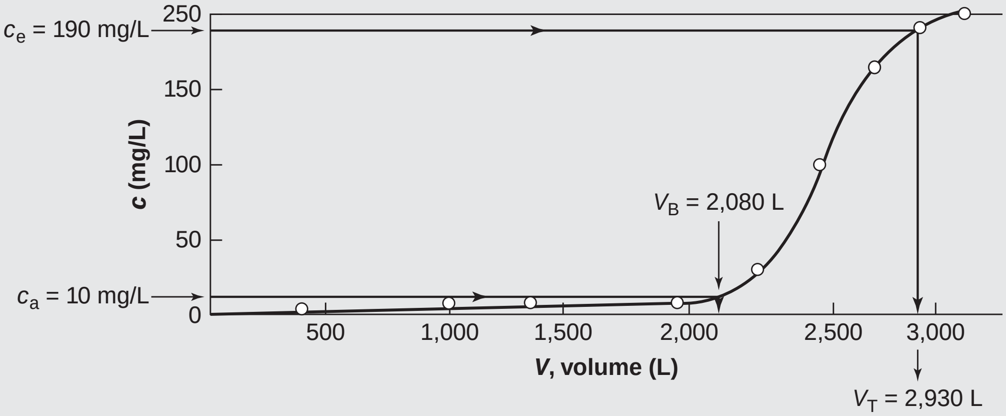


Figure 20.13 Breakthrough curve (Example 20.3).

From the breakthrough curve (Fig. 20.13):

The breakthrough volume $V_B = 2{,}080$ L.
The total volume of water treated $V_T = 2{,}930$ L.

The calculation of $(c_0/c - 1)$ is shown in Table 20.3. The $(c_0/c - 1)$ values are then plotted as a function of the volume V on a semi-logarithmic scale as shown in Fig. 20.14.

Table 20.3 Data from the breakthrough test and calculations for plotting Fig. 20.14 (Example 20.3)

V (L)	c (mg/L)	c_0/c	$c_0/c–1$
400	9	Not accurate	–
1,000	11	Not accurate	–
1,300	8	Not accurate	–
1,900	9	22.2	21.2
2,300	30	6.67	5.67
2,500	100	2.00	1.10
2,700	165	1.21	0.21
2,930	193	1.04	0.04
3,120	200	1.00	0.00

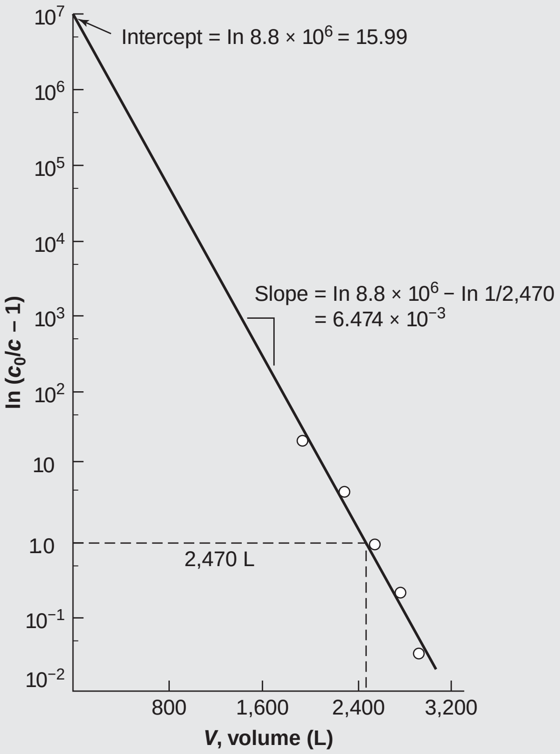


Figure 20.14 Semilog Plot for the determination of intercept and slope of column pilot test (Example 20.3).

From Fig. 20.14 we find that

$$\text{The slope} = \frac{k_1 c_0}{Q} = 6.474 \times 10^{-3}. \tag{20.23}$$

$$\text{The intercept} = \frac{k_1 q_0 M}{Q} = 15.99. \tag{20.24}$$

1. The design reaction constant k_1

From Eq.(20.23)

$$\begin{aligned} k_1 &= 6.474 \times 10^{-3} \frac{Q}{c_0} \\ &= 6.474 \times 10^{-3} \frac{12.39 \text{ L/h}}{0.2 \text{ g/L}} \\ &= 0.40 \text{ L/h g} \\ &= 0.40 \text{ L/h g} \times (24 \text{ h/day}) \times (1{,}000 \text{ g/kg})/(1{,}000 \text{ L/m}^3) \\ \boldsymbol{k_1} &= \mathbf{9.60\ m^3/day\ kg}. \end{aligned}$$

2. The design maximum solid-phase concentration q_0

From Eq.(20.24)

$$\begin{aligned} q_0 &= 15.99 \times \frac{Q}{k_1 M} \\ &= 15.99 \times \frac{12.39 \text{ L/h}}{\left(0.40 \frac{\text{L}}{\text{hg}}\right)\left(2.98 \times \frac{1{,}000 \text{ g}}{\text{kg}}\right)} \\ &= 0.17 \text{ g TOC/g carbon} \\ \boldsymbol{q_0} &= \mathbf{0.17\ kg/kg}. \end{aligned}$$

3. **The breakthrough time T_B**

From the breakthrough curve (Fig. 20.14), $V_B = 2{,}080$ L.
Flow rate given = 12.39 L/h.

$$T_B = \frac{2{,}080\ \text{L}}{12.39\ \text{L/h}} = 168\ \text{h}$$

$$T_B = 168\ \text{h}/(24\ \text{h/day}) = \mathbf{7\ days}.$$

4. **The breakthrough volume V_B**

$$V_B = QT_b$$

$$V_B = 150\ \text{m}^3/\text{day} \times 7\ \text{days}$$

$$\boldsymbol{V_B = 1{,}050\ \text{m}^3}.$$

5. **The mass of carbon *M***

$$\ln = \left(\frac{c_0}{c} - 1\right) = \frac{k_1 q_0 M}{Q} - \frac{k_1 c_0}{Q} V \qquad \textbf{(20.20)}$$

$$\ln\left(\frac{200}{10} - 1\right) = \frac{(0.40\ \text{m}^3/\text{h kg})(0.17\ \text{kg/kg})}{(150\ \text{m}^3/\text{day})/(24\ \text{h/day})} M - \frac{(0.40\ \text{m}^3/\text{h kg})(0.2\ \text{kg/m}^3)(1{,}050\ \text{m}^3)}{(150\ \text{m}^3/\text{day})/(24\ \text{h/day})}$$

$$2.94 = 0.01065M - 13.47$$

$$M = 16.41/0.01065$$

$$\boldsymbol{M = 1{,}540\ \text{kg}}.$$

6. **Diameter and length of carbon bed**

$$\begin{aligned} \text{Bed volume} &= M/\rho_s \\ &= 1{,}540\ \text{kg}/(400\ \text{kg/m}^3) \\ &= 3.85\ \text{m}^3. \end{aligned}$$

Using a unit flow rate of 3 gpm/ft^2 or 7.5 m^3/h/m^2

$$\begin{aligned} \text{Area} &= [(150\ \text{m}^3/\text{day})/24\ \text{h/day})]/7.5\ \text{m}^3/\text{h/m}^2 \\ &= 0.833\ \text{m}^2 \\ \pi D^2/4 &= 0.833\ \text{m}^2 \\ \textbf{Diameter} &= \mathbf{1.0\ m}. \end{aligned}$$

$$\begin{aligned} \text{Length of carbon bed} &= V/A \\ &= 3.85\ \text{m}^3/(\pi 1^2/4) \\ &= 4.9\ \text{m}. \end{aligned}$$

Use 5 m long bed.

7. **Weight of carbon required per unit volume treated**

$$\begin{aligned} \text{Actual volume of bed} &= 5\ \text{m} \times (\pi \times 1^2/4) \\ &= 3.925\ \text{m}^3. \end{aligned}$$

$$\begin{aligned} M &= 3.925\ \text{m}^3 \times 400\ \text{kg/m}^3 \\ &= 1{,}570\ \text{kg} \\ M/V_B &= 1{,}570\ \text{kg}/1{,}050\ \text{m}^3 \end{aligned}$$

1.5 kg carbon/m^3 of water.

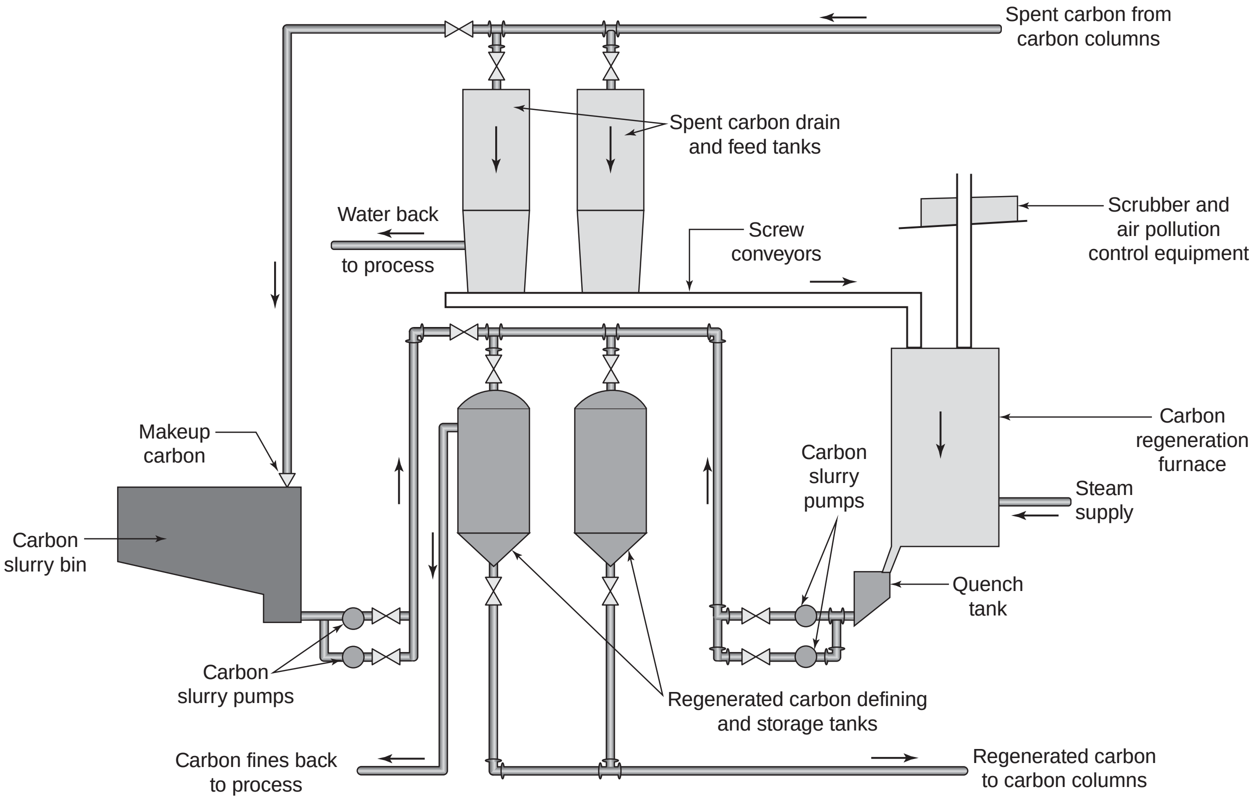


Figure 20.15 Carbon regeneration system (*Source*: US EPA).

20.7.4 Carbon Regeneration Systems

To make GAC economically feasible for treatment in most applications, the exhausted carbon must be regenerated and reused. When the plant effluent quality reaches the minimum effluent quality standards or when a predetermined carbon dosage is achieved, spent carbon is removed from the columns to be regenerated.

Closely controlled heating in a multiple hearth furnace is the best procedure for removing adsorbed organics from activated carbon. Thus, maximum effort has been concentrated on optimization of thermal regeneration techniques in an atmosphere of limited oxygen and steam.

A typical basic sequence for the thermal regeneration of carbon is as follows:

1. The granular carbon is hydraulically transported (pumped) in a water slurry to the regeneration station for dewatering.
2. After dewatering, the carbon is fed to a furnace (usually of the multi-hearth type) and is heated to 1,500–1,700°F (815–927°C) in a controlled atmosphere which volatilizes and oxidizes the adsorbed impurities.
3. The hot regenerated carbon is quenched in water.
4. The cooled regenerated carbon is washed to remove carbon fines and hydraulically transported to the adsorption equipment or to storage.
5. The furnace off-gases are scrubbed (the scrubber water is returned to the plant for processing) and may also pass through an afterburner.

A typical carbon regeneration system is shown in Figs. 20.15 and 20.16.

The thermal regeneration process itself involves three steps:

1. Drying

Figure 20.16 World's largest carbon reactivation plant located in Feluy, Belgium (Courtesy http://en.wikipedia.org/wiki/File:Reactivation_Furnace_Feluy_Belgium.jpg).

2. Baking (pyrolysis of adsorbates)
3. Activating (oxidation of the residue from the adsorbate)

The total regeneration process requires about 30 min: the first 15 min is a drying period during which the water retained in the carbon pores is evaporated, a 5-min period during which the adsorbed material is pyrolyzed and the volatile portions thereof are driven off, and a 10-min period during which the adsorbed material is oxidized and the granular carbon reactivated.

EXAMPLE 20.4 GRANULAR ACTIVATED CARBON ADSORPTION SYSTEM

A GAC bed is similar to a gravity filter bed or a pressure filter shown in Fig. 11.1 in Chapter 11 from structural view points, except that additional parts will be needed for removal of spent GAC and refilling of regenerated GAC. Although gravity GAC filters or pressure GAC filters are equally effective for removing organic impurities and certain inorganic impurities, the pressure GAC filters are traditionally used for groundwater treatment, while gravity GAC filters are commonly used for treating surface water (including GWUDI). If the gravity sand filters and pressure sand filters shown in Fig. 11.1 are named as primary filters, then the gravity GAC filters and pressure GAC filters placed behind the primary filters would be named as secondary filters. In other words, the sand filter effluent is fed to the GAC filter. Recommend a water treatment system to treat raw surface water containing excessive turbidity, color, coliforms, and volatile organic compounds (VOCs), assuming GAC filters will be used.

Solution:

There are usually multiple solutions to do any design. Using Fig. 11.1 as a guide, VOCs can be removed from water (a) by aeration before chemical feeding and rapid mixing units in a water treatment plant; (b) by GAC filters after sand filters; and (c) by PAC treatment (Example 20.5). The choices of selection depend on the raw quality and engineering situations. For instance, if the raw water contains excessive amount of VOC, iron, and manganese, the logic choice will be adoption of aeration near the head of the water treatment plant, so all three impurities can be removed by aeration. If only VOC is the problem (i.e., no iron and manganese problem) all year around, either aeration or GAC filter can be adopted for a new plan design aiming at VOC removal. For improvement of an existing water filtration plant shown in Fig. 20.17, GAC filters are frequently added after sand filters, especially

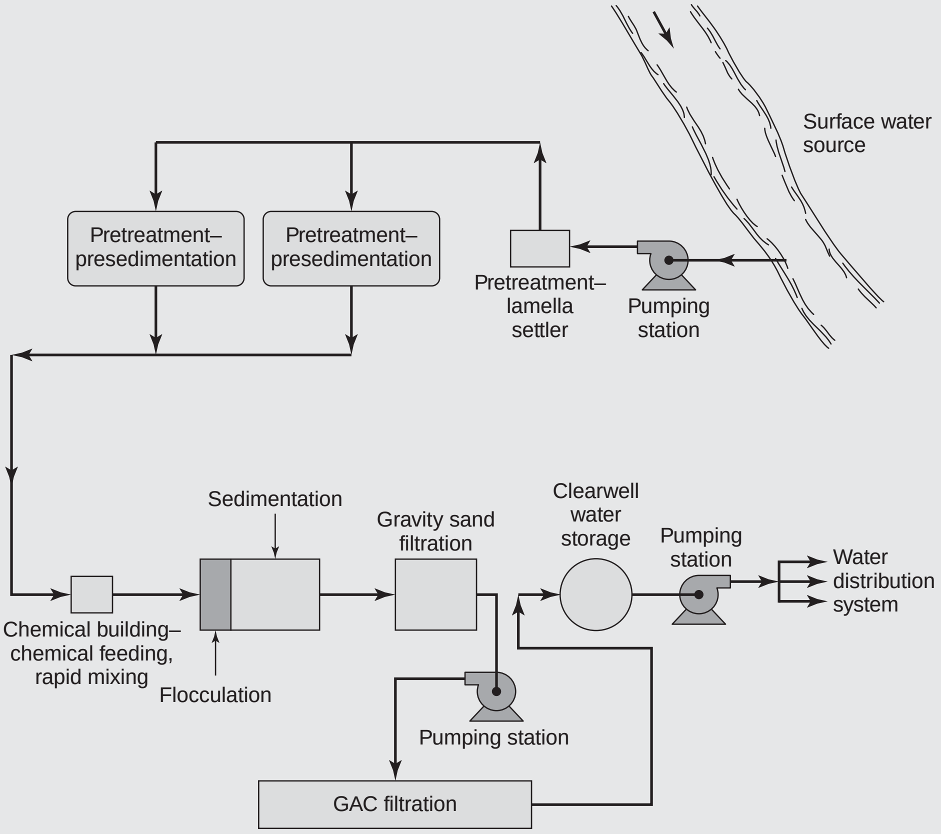


Figure 20.17 Granular activated carbon (GAC) filtration for advanced treatment in a conventional water treatment plant (*Source*: US EPA).

if the plant was overdesigned with many extra filter beds available for conversion from sand filter beds to GAC filter beds. The final choice of engineering design (aeration or GAC) is decided by not only the overall costs, but also the customer's preference. In this case, the preference is adoption of gravity GAC filters after the gravity sand filters for VOC removal. The flow diagram of a recommended water system may consist of presedimentation, pumping, chemical feeding, primary disinfection (chlorination), rapid mixing, coagulation-flocculation, sedimentation clarification, gravity sand filters, gravity GAC filters, corrosion control, secondary disinfection (chlorination), clearwell, water storage, and distribution (Figs. 11.1 and 20.17).

EXAMPLE 20.5 POWDERED ACTIVATED CARBON ADSORPTION SYSTEM FOR SEASONAL CONTROL OF VOCS

Propose a PAC adsorption system for treating surface water containing excessive amount of color-causing substances, turbidity, coliforms, and seasonal surge of VOCs. Explain the advantage and disadvantage of PAC treatment for VOC removal.

Solution:

Addition of PAC to a water treatment plant for removing VOC all year around is not economically feasible. Although PAC addition is the simplest process method (having the lowest capital cost), it is the most expensive one (having the highest O&M costs) in comparison with aeration and GAC filtration. For seasonal or occasional surge of VOCs (such as the taste- and odor-causing substances) in drinking water, dosing PAC for a short period of time to overcome the problem is a logic choice. PAC treatment is practiced at Albany Water Treatment Plant, New York. The flow diagram of a water treatment system involving the use of PAC for removal of color, turbidity, coliforms, and VOCs is similar to that of a conventional water treatment system shown in Figs. 11.1 and 11.4 in which PAC is fed in slurry form as one of the chemicals to the rapid mixer and/or to the flocculator.

20.8 ION EXCHANGE

Ion exchange is the reversible interchange of ions between a solid ion exchange medium and a solution. Water softening by ion exchange is an important example. In industry both cation and anion exchangers are employed to prepare boiler feed-water, deionize or demineralize process waters, concentrate dilute solutions of electrolytes, and prepare chemical reagents. Typical ion exchange units consist of prefiltration, ion exchange, disinfection, storage, and distribution elements (see Fig. 20.18).

Inorganics removal is accomplished through adsorption of contaminant ions onto a resin exchange medium. As the name implies, one ion is substituted for another on the charged surface of the medium, which is a resin, usually a synthetic plastic. This resin surface is designed as either cationic or anionic.

The exchange medium is saturated with the exchangeable ion before treatment operations. During ion exchange, the contaminant ions replace the regenerant ions because they are preferred by the exchange medium. After the exchange medium reaches equilibrium with the contaminant ions, the medium is regenerated with a suitable solution, which then resaturates the medium with the appropriate ions. Because of the required "down time," the shortest economical regeneration cycles are once per day. Ion exchange waste is highly concentrated and requires careful disposal, analogous to RO reject streams.

The porous exchange medium is covered with tiny holes, which clog when significant levels of suspended solids are in the influent stream. Consequently, filtration may be a necessary pretreatment for ion exchange units. Ion exchange units are also sensitive to the presence of competing ions. For example, influent with high levels of hardness will effectively compete with other cations for sites on the exchange medium.

The resin exchange capacity is expressed in terms of weight per unit volume of the resin. Factors that affect the design of an ion exchange system include the presence of oil and grease, contaminant concentration, exchange capacity of the resin, suspended solids, metals, oxidant content, inorganic ions in the water, and pH of water. The calculation of the breakthrough time for an ion exchange unit requires knowledge of the resin exchange capacity, the influent contaminant concentration, and the desired effluent quality. The basis for the design of an ion exchange bed using the breakthrough curve is the same as discussed previously for carbon adsorption.

The ion exchange process is relatively insensitive to flow rate and virtually fully operational upon process initiation. Each resin, of the many available from suppliers, is effective in removing specific contaminants. In addition, ion exchange is able to achieve very low contaminant concentrations in finished water.

20.8.1 Advantages and Disadvantages of Ion Exchange

The primary advantage of the ion exchange process is that like RO it can be used with fluctuating flow rates. Another advantage is that large varieties of specific resins are available

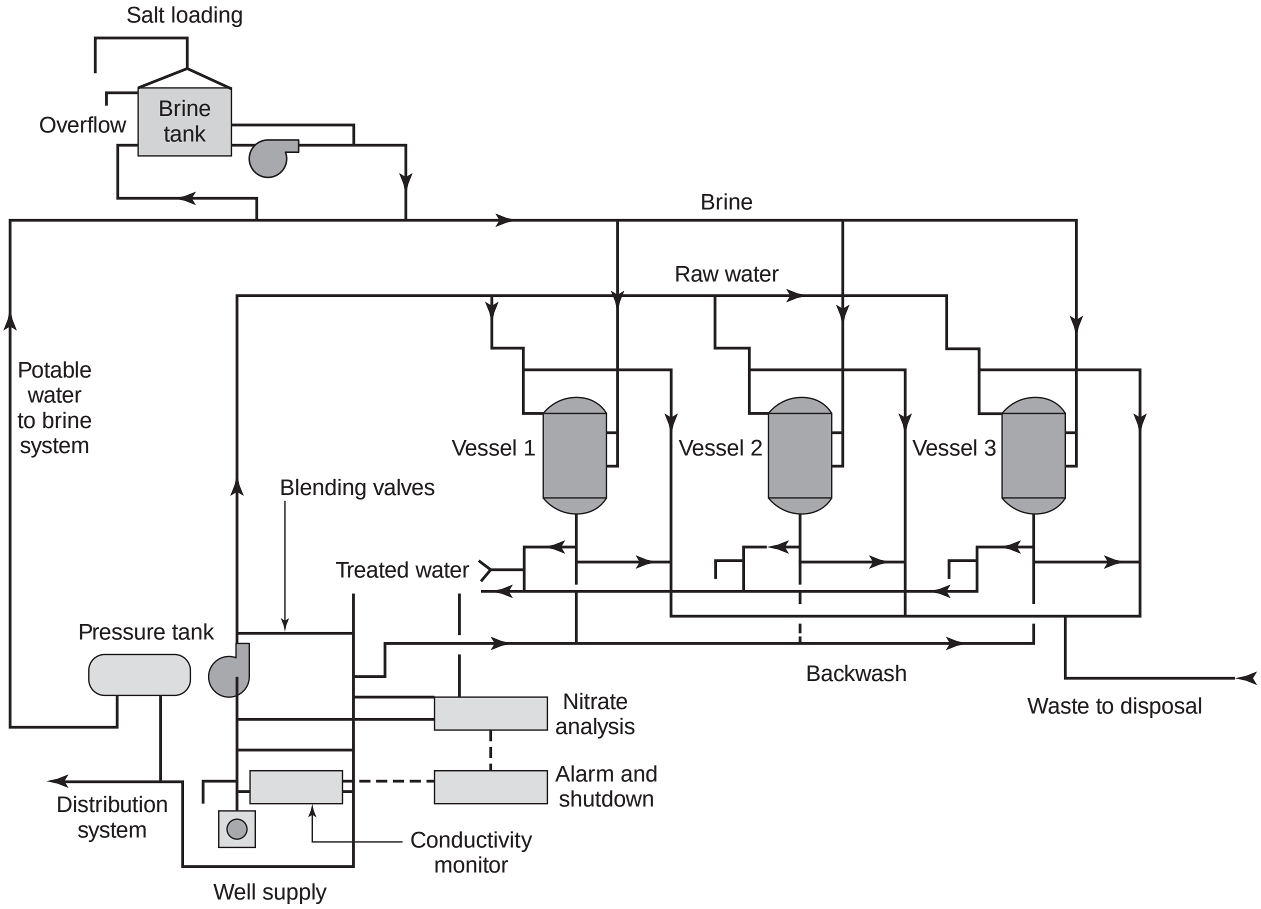


Figure 20.18 Ion exchange treatment system (*Source*: US EPA).

from suppliers where each resin is effective in removing specific contaminants.

The primary disadvantage of ion exchange concerns effluent peaking. Effluent peaking occurs when contaminant ions compete with other ions for exchange medium sites, resulting in unacceptable levels (peaks) of contamination in the effluent. It is most common with poorly adsorbed contaminants, such as nitrate. Effluent peaking necessitates more frequent regeneration of the exchange medium.

In addition, ion exchangers that use sodium chloride to saturate the exchange medium may experience problems with sodium residual in the finished water. Sodium is used because of its low cost, but high sodium residual is unacceptable for individuals with salt-restricted diets. This problem may be avoided by using other saturant materials, such as potassium chloride.

Some other disadvantages include the following:

1. Wastewater generated during the regeneration step will require additional treatment and disposal.
2. Oil and grease in the groundwater may clog the exchange resin.
3. Suspended solids content greater than 10 mg/L may cause resin blinding.
4. Oxidants in groundwater may damage the ion exchange resin.
5. Pretreatment is required for most surface waters.
6. The pH of the influent water may affect the ion exchange resin selection.
7. Ion exchange units are also sensitive to the presence of competing ions. For example, influent with high levels of hardness will compete with other cations (positive ions) for space on the exchange medium, and the exchange medium must be regenerated more frequently.

20.8.2 System Performance

Ion exchange effectively removes 90–95% of barium, cadmium, chromium(III), silver, and radium using cationic resins as the exchange medium. In addition, it achieves greater than 90% removal of nitrites, selenium, arsenic(V), chromium(VI), and nitrate using anionic resins. While it is effective for all of the contaminants listed above, ion exchange is especially well suited to remove barium, radionuclides (e.g., radium), nitrate, and selenium.

20.9 ION EXCHANGERS

To be effective, solid ion exchangers must (a) contain ions of their own, (b) be insoluble in water, and (c) provide enough space in their porous structure for ions to pass freely in and out of the solid. Cation exchangers have a negatively charged framework, but their pores contain cations that maintain electroneutrality. Anion exchangers carry just the opposite electrical charges.

Soils are important ion exchangers—among them clay soils, the humus produced by decaying vegetation, and the bottom sediments in rivers and lakes. Alumina, SiO_2, MnO_2, metal phosphates and sulfides, lignin, proteins, cellulose, wool, living cells, carbon, and resins likewise have ion exchange properties. The silicate minerals differ considerably in ion exchange capacity. Some of the natural aluminum silicate minerals—for instance, greensand (glauconite) zeolites—have served as commercial ion exchangers. They are derived from greensand by washing, heating to slight surface fusion, and treatment with NaOH. Zeolites have also been synthesized by mixing solutions of sodium silicate and sodium aluminate, drying the resulting white gel, and crushing it to the right size. An example of ion exchange resin beads is shown in Fig. 20.19.

20.9.1 Synthetic Ion Exchangers

Modern ion exchange technology began in 1935 with the discovery of synthetic ion exchange resins. However, natural as well as synthetic zeolites have continued in commercial use. The synthetic ion exchangers are pervious, reasonably stable, and have high capacities (Table 20.4). Relatively large particles, 1–2 mm in diameter and synthesized with specific ionic functional groups, can be employed. The chemical structure of two synthetic ion exchange resins is shown in Fig. 20.20. They act essentially as a single, large, many-charged ion in which thousands of atoms are linked into a three-dimensional network. The charge of the polymeric ion is neutralized by small ions of opposite charge within the network, where they are held in place by electrostatic forces without being bound to the ion exchange matrix. Thus they are able to change places with ions in a solution. The charge in the matrix is allied to functional groups.

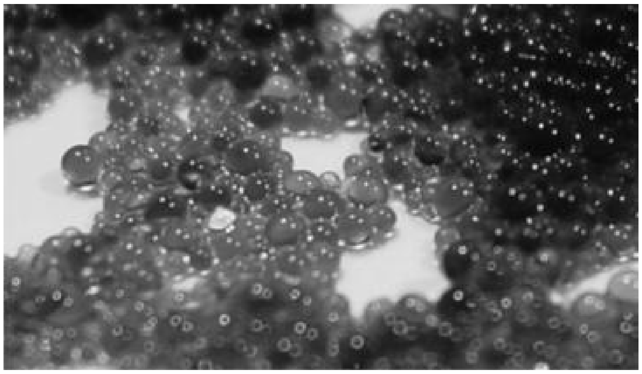


Figure 20.19 Ion exchange resin beads (Courtesy http://en.wikipedia.org/wiki/File:Ion_exchange_resin_beads.jpg).

The cation exchanger of Fig. 20.20 contains $-SO_3^-$ groups. Different functional groups, such as $-COO^-$, $-PO_3H^-$, and $-C_6H_5O^-$, can be introduced instead. Exchangers with functional groups derived from a strong acid, $-SO_3^-$ from H_2SO_4, for instance, are called *strongly acidic*. Exchangers with carboxylic groups derived from a weak acid are referred to as *weakly acidic*.

Anion exchangers contain positively charged functional groups, among them quaternary ammonium ($-NR_3^+$), amino ($-NH_3^+$), imino ($-NRH_2^+$), phosphonium ($-PR_3^+$), and sulfonium ($-SR_3^+$). A strongly basic exchanger is one that contains quaternary ammonium groups.

20.9.2 Activated Alumina and Zeolite

Activated alumina (AA) is manufactured from aluminum hydroxide by dehydroxylating it in a way that produces a highly porous material; AA can have a surface area significantly over 200 m^2/g. The compound is used as a desiccant (to keep things dry by absorbing water from the air) and as a filter of fluoride, arsenic, and selenium in drinking water. AA is made of aluminum oxide (alumina; Al_2O_3) and has a very high surface-area-to-weight ratio. It has many small pores like tunnels that run throughout it.

AA filters are widely used in the defluoridation of drinking water because they can easily reduce fluoride levels from 0.5 mg/L to less than 0.1 mg/L. The more alumina in the filters, the less fluoride will be in the final filtered water. Lower temperature water and lower pH water (acidic water) are filtered more effectively too. AA can be regenerated by a solution of lye (sodium hydroxide; NaOH), sulfuric acid (H_2SO_4), or alum [$KAl(SO_4)$]. Figure 20.21 shows schematic diagrams of the operational process.

Before processing, the AA medium is saturated with hydroxyl ions from a strong sodium hydroxide solution. During the removal process, AA exchanges the hydroxyl ions for fluoride anions. When the medium becomes saturated, the AA must be regenerated with sodium hydroxide. This increases the pH level of the water in the treatment unit to a point where buffer solutions of 3% sulfuric acid are required for neutralization, after the water leaves the exchange unit.

While AA requires pretreatment for suspended solids, it can tolerate high levels of total dissolved solids. As with other ion exchange processes, the AA process is sensitive to competing ions. It is also sensitive to pH. Optimum removal efficiency for fluoride occurs below a pH of 8.2.

While AA effectively removes several contaminants, it requires the handling of potentially hazardous strongly

Table 20.4 Exchange capacities and regeneration requirements of natural and synthetic ion exchangers

		Exchange capacity			
Substance and ion	Functional group	Wet (me/mL)	Dry (me/g)	Regeneration into H^+, Na^+, or OH^- form by	Trade name
		A. ***Cation-exchange zeolites***			
Natural zeolite, Na^+	Silicate	0.15–0.3	–	Excess NaCl	Zeolite
Synthetic zeolite, Na^+	Silicate	0.3–0.8	…	Excess NaCl	Zeolite
		B. ***Cation-exchange resin***			
Strong acid, H^+ and Na^+	$-SO_3H$	4–5		Excess strong acid or NaCl	Zeo-Karb/225 Amberlite IR-120 Dowex 50
Weak acid, H^+ and Na^+	$-COOH$	4–5[a]	8–10[a]	Acid or NaCl	Zeo-Karb/226 Amberlite IRC-50
		C. ***Anion-exchange resins***			
Strong base, OH^-	$-R_3N^+$	1.2–1.4	3.5–4.5	Excess strong base	Amberlite IRA-400, 410 Dowex 1,2
Weak base, OH^-	$-RH_2N^+$	2–2.5[b]	4.5–5.5[b]	Na_2CO^3	Amerlite IR Dowex 3
		D. ***Clays***			
Montmorillonite, Na^+	Silicate	–	0.8–1.0	…	…
Kaolinite, Na^+	Silicate	…	0.02–0.1	…	…
Illite, Na^+	Silicate	…	0.2–0.4	…	…
		E. ***Hydrous oxides***			
$Fe_2O_3xH_2O$	Fresh precipitate	…	4[c]	…	….
MnO_2xH_2O	$-MnO_2$	…	15[c]	…	….

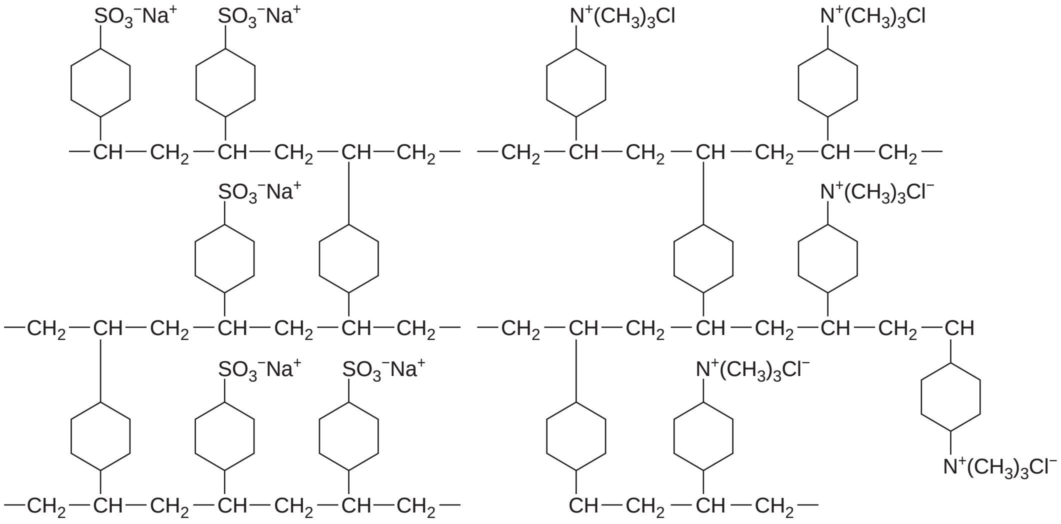
Source: After Fair et al. (1971).

Conversion factors: 1 kg/ft^3 = 35.33 kg/m^3; 1 lb/ft^3 = 16.02 kg/m^3.

[a] At high pH.

[b] At low pH.

[c] At pH 8.3. Observed capacity for divalent transition elements. In practice, exchange capacities are commonly expressed in kg/ft^3, and regeneration requirements in lb/ft^3.



(*a*) Cation exchanger, {Na^+R^-}

(*b*) Anion exchanger, {R^+Cl^-}

Figure 20.20 Schematic representation of the three-dimensional network of (a) a strongly acidic cation exchanger, {Na^+R^-}, and (b) a strongly basic anion exchanger, {R^+Cl^-}. (After Fair et al., 1971).

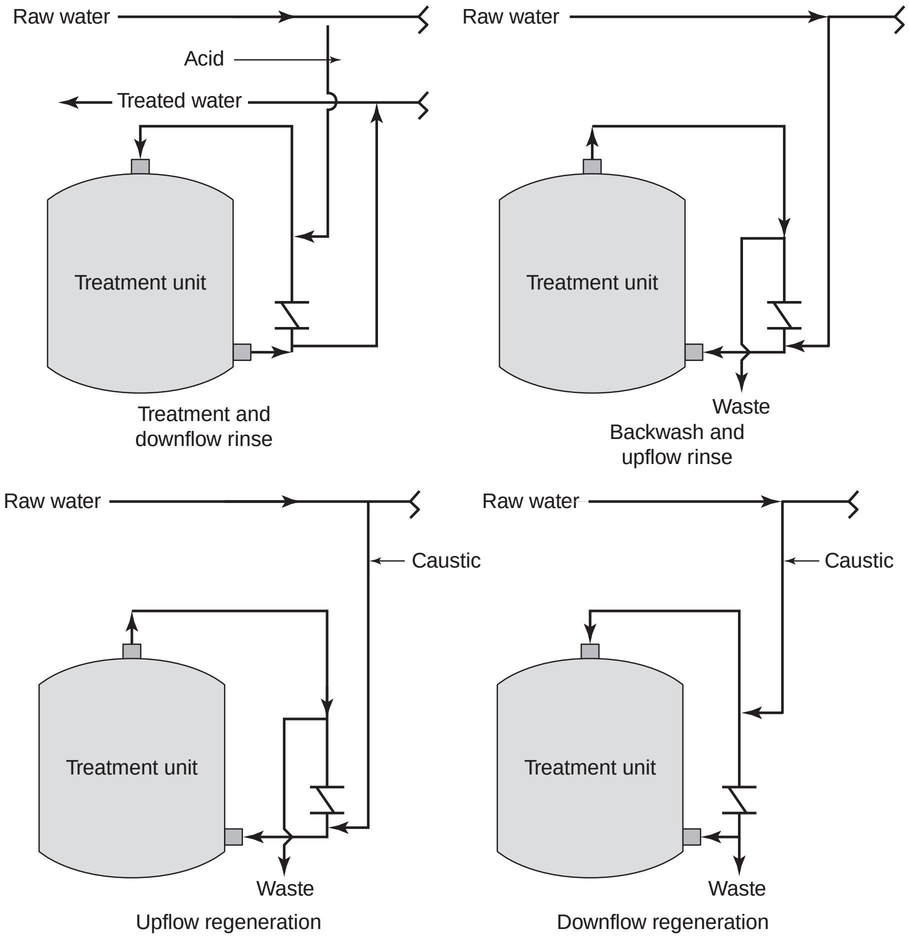


Figure 20.21 Activated alumina systems: operating mode flow schematics (*Source*: US EPA).

acidic and basic solutions. The process of heating the sodium hydroxide solution to maintain it as a liquid in cold climates is particularly hazardous.

AA's costs are also higher than ion exchange costs. The loss of AA to the sodium hydroxide solution during processing can range up to 20% annually. In addition, waste management may also increase costs because of the high contaminant and aluminum concentrations in the waste stream, as well as the high pH.

AA is operational immediately because of its relative insensitivity to flow rates. It effectively removes 90–95% of arsenic(V), fluoride, and selenium(IV); removes 70% of selenium(VI); and also effectively removes iron. It is ineffective for barium, cadmium, and radium removal, since these contaminants occur primarily as cations.

Zeolites are natural or synthetic crystalline aluminosilicates, which have a repeating pore network and release water at high temperature. Zeolites are polar in nature.

They are manufactured by hydrothermal synthesis of sodium aluminosilicate or another silica source in an autoclave followed by ion exchange with certain cations (Na^+, Li^+, Ca^{2+}, K^+, $NH_4{}^+$). The channel diameter of zeolite cages usually ranges from 2 to 9 Å (1 Å = 1×10^{-8} cm). The ion exchange process is followed by drying of the crystals, which can be pelletized with a binder to form macroporous pellets.

Zeolites are applied in drying of process air, CO_2 removal from natural gas, CO removal from reforming gas, air separation, catalytic cracking, water softening, and catalytic synthesis and reforming.

Nonpolar (siliceous) zeolites are synthesized from aluminum-free silica sources or by dealumination of aluminum-containing zeolites. The dealumination process is done by treating the zeolite with steam at elevated temperatures, typically greater than 500°C (1,000°F). This high-temperature heat treatment breaks the aluminum–oxygen bonds and the aluminum atom is expelled from the zeolite framework.

20.10 THE ION EXCHANGE PROCESS

Electrically charged atoms or molecules are known as ions. The ion exchange process uses special *resins* (known as *ion exchangers*), to remove charged, inorganic contaminants like arsenic, chromium, nitrate, calcium, radium, uranium, and excess fluoride from water. When source water is passed through a series of resin beads, it exchanges its charged contaminants for the harmless charged ions stored on the resin surface. Ion exchange resins then store the attracted contaminants. Because of this accumulation process, resins must be periodically cleaned with a solution that *recharges* their supply of harmless, interchangeable ions.

Ion exchange resins come in two forms: cation exchange resins (*cation exchangers*), which exchange cations like calcium, magnesium, and radium, and anion exchange resins (*anion exchangers*) used to remove anions like nitrate, arsenate, arsenic, or chromate. Both are generally regenerated with a salt solution (sodium chloride). In the case of cation exchangers, the sodium ion displaces the cation from the exchange site; and in the case of anion exchangers, the chloride ion displaces the anion from the exchange site. As a rule cation exchangers are more resistant to *fouling* than are anion exchangers. Resins can be designed to show a preference for specific ions, so that the process can be easily adapted to a wide range of different contaminants. The ion exchange process works best with particle-free water, because particulates can accumulate on the resin and limit its effectiveness.

The ion exchange process can be formulated as follows:

$$\begin{gathered}\{H^+R^-\} + Na^+ \rightarrow \{Na^+R^-\} + H^+ \text{and} \\ 2\{Na^+R^-\} + Ca^{2+} \rightarrow \{Ca^{2+}R_2^-\} + 2Na^+ \end{gathered} \qquad \textbf{(20.25)}$$

$$\begin{gathered}\{R^+Cl^-\} + OH^- \rightarrow \{R^+OH^-\} + Cl^- \text{and} \\ 2\{R^+OH^-\} + SO_4^{2-} \rightarrow \{R_2^{2+}SO_4^{2-}\} + 2OH^- \end{gathered} \qquad \textbf{(20.26)}$$

where R^- and R^+ symbolize the negatively and positively charged network of the cation or anion exchanger, respectively.

Within the solution and the ion exchange medium, a charge balance (electroneutrality) must be maintained; the number of charges, and not the number of ions, must stay constant within or on the exchanger granule. One Ca^{2+} ion displaces two Na^+ ions, for instance. Consequently the *exchange capacity* of an ion exchanger is expressed by the number of charges, that is, the equivalents of ions necessary to maintain electroneutrality within the solid phase for a given weight or volume of ion exchange material. The capacity can be established analytically by determining the quantity of ions exchangeable when the exchange reaction is driven to completion. Because cation exchangers in the $\{H^+R^-\}$ form are essentially solid *polyprotonic acids* (acids that can donate more than a single hydrogen atom), they can be titrated alkalimetrically (Fig. 20.22).

The design of ion exchange processes is based on knowledge of *ion exchange equilibria*, which govern the distribution of ions between an aqueous phase and the ion exchanger phase. Because ion exchange reactions obey the mass law, the exchange reactions may be characterized as follows:

$$\frac{[Na^+R][H^+]}{[H^+R][Na^+]} = Q_{HR\rightarrow NaR} \qquad \textbf{(20.27)}$$

and

$$\frac{[Ca^{2+}R][Na^+]^2}{[Na^+R]^2[Ca^{2+}]} = Q_{NaR\rightarrow CaR} \qquad \textbf{(20.28)}$$

The resulting equilibrium concentration quotients, Q, are not thermodynamic equilibrium constants and provide only a semiquantitative interpretation of the ion exchange equilibrium. Because they characterize the relative selectivities of an ion exchanger for specific ions, the quotients are called *selectivity coefficients*. In the exchange of monovalent ions by other monovalent ions, the magnitudes of Q are very

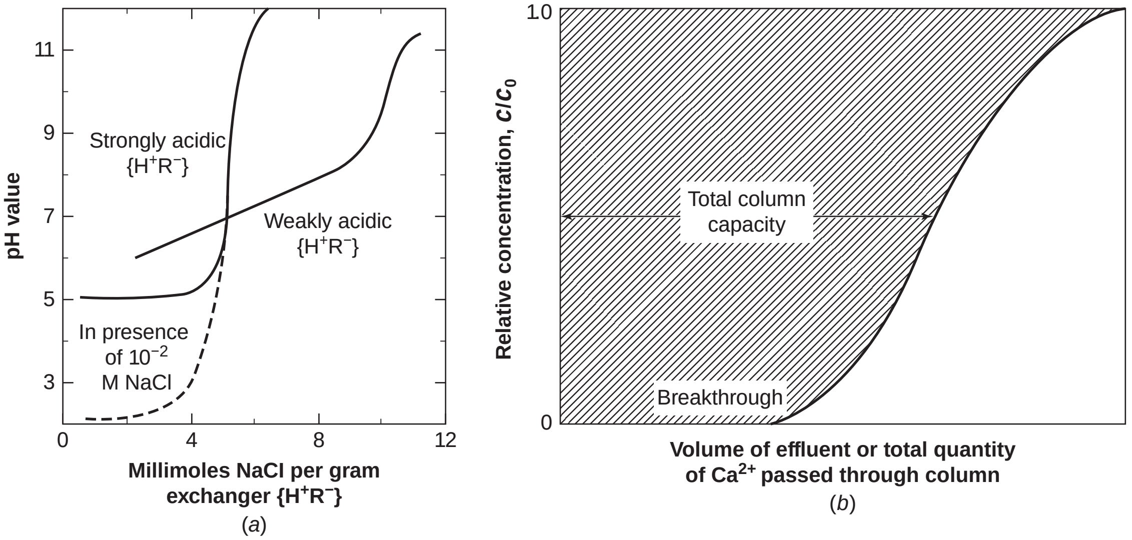


Figure 20.22 Determination of ion exchange capacity. (a) Alkalimetric titration of cation exchanger in the $\{H^+R^-\}$ form provides information on the acidity of the exchanger and permits the determination of the analytical exchange capacity. (b) Determination of useful capacity of an ion exchange column (After Fair et al., 1971).

close to unity. The affinity of exchangers for bivalent ions is much larger than for monovalent ions; selectivity coefficients of 20–40 are typical. The mass law formulation for bivalent–monovalent exchange (Eq. 20.27) shows that a dilute Ca^{2+} ion solution in contact with a cation exchanger in the Na^+ form, $\{Na^+R\}$, drives the reaction to almost stoichiometric completion as written, that is, from left to right. However, the resulting exchanger in the Ca^{2+} form, $\{Ca^{2+}{R_2}^{2-}\}$, can be regenerated into the Na^+ form by treatment with a concentrated solution of Na^+.

It follows from Eq. (20.28) that the distribution of two ions of differing valences is strongly dependent on the concentration of the solution. In a hetero-ionic reaction, the selectivity of the exchanger for the bivalent ion over the monovalent ion rises with increasing dilution of the solution. An exchanger will selectively *sorb* Ca^{2+} ions with great preference from a mixed Na^+–Ca^{2+} solution when it is sufficiently dilute. In solutions of high concentration, however, the exchanger loses its selectivity. A cation exchanger can remove Ca^{2+} ions selectively from a dilute solution, that is, natural water, even when $[Ca^{2+}] \ll [Na^+]$ in the solution. However, an exhausted exchanger in the Ca^{2+} form can be regenerated or reconverted into a $\{Na^+R^-\}$ exchanger by a concentrated brine solution or full-strength seawater. Exchange reactions are generally slower than reactions between electrolytes in solution. In reactions with synthetic resin exchangers, for example, the time of half-exchange is of the order of a few minutes when the reaction is carried out under the usual conditions of small-particle resins, small-diameter ions, approximately 10^{-3} M solutions, and room temperature.

Table 20.5 Ion-exchange selectivity coefficients at 25°C[a]

	Crosslinking			
Replaceable ion	2%	4%	8%	16%
Monovalent cations				
Li		1.00	1.0	1.0
H		1.32	1.27	1.47
Na		1.58	1.98	2.37
K		2.27	2.90	4.50
Rb		2.46	3.16	4.62
Cs		2.67	3.25	4.66
Ag		4.73	8.51	22.9
TI		6.71	12.4	28.5
Divalent cations				
Mg		2.95	3.29	3.51
Ca		4.15	5.16	7.27
Sr		4.70	6.51	10.1
Ba		7.47	11.5	20.8
Pb		6.56	9.91	18.0
Monovalent anions				
OH	0.80		0.50	
F			0.08	
Cl	1.0		1.0	
Br	2.7		3.5	
I	9.0		18.5	
NO_3			3.0	
SCN	6.0		4.3	
ClO	9.0		10.0	

Source: US EPA.

[a] Data are for polystyrene base resins, sulfonic acid cation exchanger, and type-2 quaternary base anion exchanger. A value greater than 1 indicates preferential adsorption of ion named against the reference ion (Li for cations, Cl for anions).

20.11 ION SELECTIVITY

The selectivity of a resin determines whether or not an equilibrium is favorable for a given ion system. The selectivities of resins are well known and, therefore, useful for determining the resin that is most suitable for a given application. Resin manufacturers are now able to synthesize a resin incorporating particularly high selectivity for certain ions. The selectivity of resins is expressed in terms of the *selectivity coefficient K.*

$$M_1 + R - M_2 \leftrightarrow M_2 + R - M_1 \qquad \textbf{(20.29)}$$

The selectivity coefficient, K, is given by

$$K = \frac{[M_2][R - M_1]}{[M_1][R - M_2]} = \left[\frac{M_2}{M_1}\right] \times \left[\frac{R - M_1}{R - M_2}\right] \qquad \textbf{(20.30)}$$

That is, K = solution concentration ratio × Solid-phase concentration ratio.

The greater is K, the greater the preference for the ion by the exchanger. Table 20.5 lists the selectivity coefficients for common cations and anions.

The various factors affecting selectivity are given below:

1. *Magnitude of charge on the ion*: In general, the selectivity of a functional resin for exchange of ions is dependent upon the ionic charge. As one might suspect, the most significant influence is the magnitude of the charge on the ion; an exchanger prefers counter-ion of high valence. Thus for a series of common anions in water and wastewater treatment one would expect the following order of preference:

$$PO_4^{3-} > SO_4^{2-} > Cl^- \qquad \textbf{(20.31)}$$

Similarly, for a series of cations,

$$Th^{4+} > Nd^{3+} > Ca^{2+} > Na^+ \qquad \textbf{(20.32)}$$

The preference for counter-ions of highest charge decreases with concentration of the external electrolyte. The force with which an ion is attracted is proportional to its ionic charge and, therefore, the counter-ion of lower valence exhibits less attraction by the exchanger. Although in general ions of higher charge are preferentially exchanged, there are exceptions. For example, the monovalent I^- and $NO_3{}^-$ ions have a higher exchange preference than the divalent chromate $CrO_4{}^{2-}$ ion, as indicated by the order of the following series of anions:

$$SO_4^{2-} > I^- > NO_3^- > CrO_4^{2-} > Br^- \qquad \mathbf{(20.33)}$$

2. *Atomic number and ionic radius*: The exchanger preference for counter-ions increases with increasing atomic number and increasing ionic radius. Hence, for a series of monovalent anions, the order of exchanger preference is as follows:

$$\begin{aligned} &CNS^- > C1O_4^- > I^- > NO_3^- > Br^- > CN^- \\ &\quad > HSO_4^- > NO_2^- > Cl^- > HCO_3^- > CH_3COO^- \\ &\quad > OH^- > F^- \qquad \mathbf{(20.34)} \end{aligned}$$

3. *Hydration radius*: Another factor influencing the selectivity of an exchange reaction is the degree of hydration. In water, fixed ions within the exchanger and mobile ions in both the resin and the solution tend to surround themselves with water molecules. Hydration of the ions exerts a swelling pressure within the exchanger. The consequence to this observation is that exchangers prefer the ions of smallest hydrated radius. According to Weber, "multivalent hydrated ions are smaller in size than an equivalent charge unit of ions of lower valence and, therefore, are preferred. Within a given series of ions the hydrated radius is generally inversely proportional to the unhydrated ionic radius.'' Thus for the alkaline earth metals the order of preference for ion exchange resin is

$$Ba^{2+} > Sr^{2+} > Ca^{2+} > Mg^{2+} > Be^{2+} \qquad \mathbf{(20.35)}$$

and for alkali metals

$$Cs^+ > Rb^+ > K^+ > Na^+ > Li^+ \qquad \mathbf{(20.36)}$$

4. *Interaction of ions*: A fourth factor bearing an influence on selectivity is the interaction of ions within the resin and in bulk solution. The exchanger prefers counter-ions that associate strongly with the fixed ionic groups. For example, a resin carrying the carboxylic group will prefer the hydronium ion because carboxylic acid is largely undissociated. For the interaction of ions in the bulk solution, the exchanger prefers the counter-ions that are least attracted to co-ions in solution phase.
5. *Screening action*: Screening action is the exclusion of ions by screening or sieving action. Organic ions and inorganic complexes are often too large to penetrate the matrix of the exchange resin. Naturally, this effect is more pronounced for exchangers having a high degree of crosslinkage.
6. *Crosslinkage*: Last among significant selectivity factors is the extent of crosslinkage in the resin. The greater the crosslinkage, the greater is the selectivity of a resin for one ion over another. As the crosslinkage decreases, the effect of this factor diminishes to eventual negligibility.

Selectivity can be useful in the preparation of specific resins. For example, successful removal of cesium has been accomplished through development of an exchanger that preferentially accepts cesium ions from a mixture of many others. However, this same selectivity characteristic can play a major negative influence in the regeneration of the resin. The resin will exhibit difficulty in releasing the adsorbed ion; hence, a higher concentration of regenerant would be required to enable the regeneration process to proceed efficiently.

20.12 KINETICS OF ION EXCHANGE

The kinetics of ion exchange is similar to that involved in activated carbon adsorption. As in porous carbon adsorption the rate of ion exchange depends upon the rates of the following steps:

1. Transport or movement of the ions from bulk solution to the boundary layer or surface of the resin
2. Boundary layer diffusion: transport through the boundary layer to the solid surface
3. Pore diffusion: diffusion of the ions inward through the pores of the resin particles to the sites of active exchange
4. Exchange of ions
5. Diffusion of the exchanged ions outward through the pores
6. Diffusion of the exchanged ions through the boundary layer surrounding the solids
7. Movement of the exchanged ions from the boundary layer to the bulk solution

The rate at which an ion exchange reaction approaches the condition of equilibrium depends on the rate limiting step. The controlling step is usually either step 2 or step 3 and not the actual exchange process (step 4). The rate equation can be expressed by the Thomas relationship as explained previously in adsorption:

$$\frac{c}{c_0} = \frac{1}{1 + e^{\frac{k_1}{Q}(q_0 M - c_0 V)}} \qquad \mathbf{(20.18)}$$

where

c = effluent solute concentration

c_0 = influent solute concentration

k_1 = rate constant, L/h-kg (gal/h-lb)

q_0 = maximum solid-phase concentration of the exchanged ions, kg/kg (lb/lb)

M = mass or weight of resin, kg (lb)

V = throughput volume (total volume treated from breakthrough curve), L (gal)

Q = water or wastewater flow rate, L/h (gal/h)

Rearranging Thomas equation we get,

$$\frac{c_0}{c} = 1 + e^{\frac{k_1}{Q}(q_0 M - c_0 V)} \quad \textbf{(20.19)}$$

$$\ln\left(\frac{c_0}{c} - 1\right) = \frac{k_1 q_0 M}{Q} - \frac{k_1 c_0}{Q} V \quad \textbf{(20.20)}$$

This equation plots $\log\left(\frac{c_0}{c} - 1\right)$ versus V as a straight line (Fig. 20.13) with

$$\text{Intercept} = \frac{k_1 q_0 M}{Q} \quad \textbf{(20.21)}$$

and

$$\text{Slope} = \frac{k_1 c_0}{Q} \quad \textbf{(20.22)}$$

The data required for the solution of the Thomas equation can be obtained from the pilot ion exchange column test. The methodology for the design of the ion exchanger bed is identical to the procedure followed in Example 20.3.

20.13 ION EXCHANGE TECHNOLOGY

Solutions are put into contact with ion exchangers either in batch or in column operations. In batch operations exchanger and solution are stirred in a vessel until equilibrium is reached. Because only a small portion of the exchange capacity is generally satisfied, batch processes are of limited usefulness. In column operations the solution flows through a vertical cylinder filled with the ion exchanger. The column can be considered a series of sequent batch operations, with progressively more ion-depleted solution coming into contact with less-exhausted ion exchanger, until exchange is ultimately complete. Multiple-equilibrium or near-equilibrium steps are established. Accordingly, even solute ions with unfavorable selectivity coefficients can be removed in quantity by this simple application of the mass-action principle. The reaction is driven to completion in much the same way as in adsorption chromatography or countercurrent operations.

The columnar uptake of Ca^{2+} ions by an exchanger in the Na^+ form (Na^+R^-) is shown schematically in Fig. 20.23.

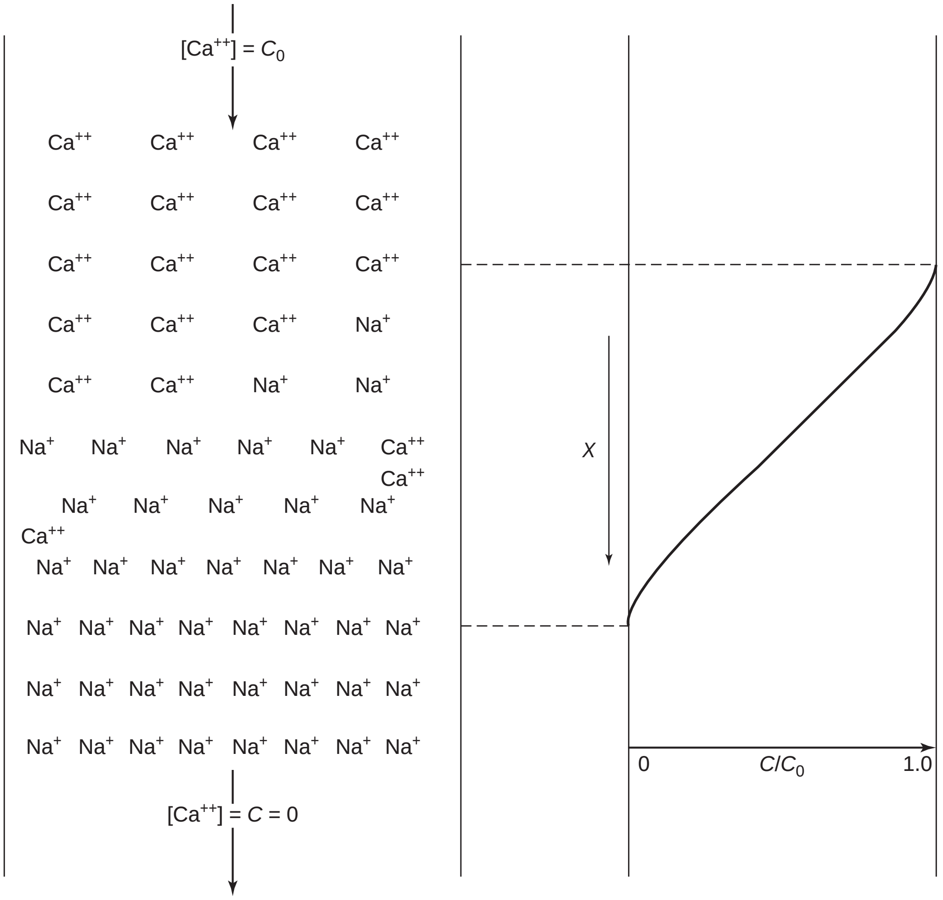


Figure 20.23 Columnar operation of an ion exchanger (After Fair et al., 1971).

The concentration ratio (C/C_0) or $[Ca^{2+}]/[Ca^{2+}_{initial}]$ along the length of the column is also indicated. The effluent from the column is free of Ca^{2+} ions, but it contains an equivalent concentration of Na^+ ions. Eventually the column will become saturated with Ca^{2+} ions, and Ca^{2+} ions will appear in the effluent at this breakthrough point for Ca^{2+}. Figure 20.22 shows, instead, the concentration ratio C/C_0 or $[Ca^{2+}]/[Ca^{2+}_{initial}]$ as a function of the volume or total quantity of Ca^{2+} passed through the column. The area enclosed by the breakthrough curve is a measure of the total capacity of the ion exchange bed. However, the useful or breakthrough capacity is smaller in actual operations. Breakthrough capacity is influenced by exchanger particle size, column dimensions, flow rate, temperature, and solution composition. The breakthrough curve generally steepens with increase in column length, decrease in flow rate, and decrease in particle size. The curve is not necessarily symmetrical. There may be considerable tailing at the end and beginning of the curve when the bed contains air pockets or air channels, or deposits of colloidal substances (e.g., iron oxide). Breakthrough capacity is then reduced. Similar effects accompany fouling of the organic ion exchange material. An experimental determination of the breakthrough curve is valuable for characterizing the useful capacity of an ion exchange column as a function of hydraulic parameters.

20.14 WATER SOFTENING BY ION EXCHANGE

Replacement of hardness-causing calcium and magnesium ions and lesser amounts of the heavier metal ions (Fe^{2+}, Mn^{2+}, and Sr^{2+}) by sodium ions is an important function of ion exchange. Water softening by cation exchange is represented by Eqs. (20.37)–(20.40) in which “R” represents the exchanger medium:

$$Ca(HCO_3)_2 + Na_2R \rightarrow CaR + 2NaHCO_3 \quad \textbf{(20.37)}$$

$$Mg(HCO_3)_2 + Na_2R \rightarrow MgR + 2NaHCO_3 \quad \textbf{(20.38)}$$

$$CaSO_4 + Na_2R \rightarrow CaR + Na_2SO_4 \quad \textbf{(20.39)}$$

$$MgSO_4 + Na_2R \rightarrow MgR + Na_2SO_4 \quad \textbf{(20.40)}$$

When the sodium is all replaced, it is necessary to regenerate the exchange medium. This is done by passing a strong solution of salt, NaCl, through the exchange medium so that sodium replaces the calcium and magnesium as shown in Eq. (20.41):

$$2NaCl + CaR + MgR \rightarrow Na_2R + CaCl_2 + MgCl_2 \quad \textbf{(20.41)}$$

Strongly acidic synthetic cation exchange resins are the exchangers of choice, but synthetic or natural zeolites continue to be used in high-temperature industrial operations and small-scale domestic installations. The removal of Ca^{2+} and Mg^{2+} is much more complete than in the lime-soda process; virtual freedom from Ca^{2+} and Mg^{2+} ions is attained, but alkalinity is not removed. Some residual hardness can be left in the finished water by split treatment, only part of the water being passed through the ion exchange bed. The exchanger effluent is blended with the untreated water stream to produce a resultant water of desirable hardness, usually between 70 and 100 mg/L. For waters of moderate hardness ($[Ca^{2+}]$ <200 mg/L as $CaCO_3$) and waters with much non-carbonate hardness, ion exchange softening is generally less expensive than lime-soda softening. For very hard waters, partial softening by lime-soda treatment may be followed by ion exchange. Regeneration of ion exchange columns requires NaCl considerably in excess of the column capacity—approximately 5 moles of NaCl for 1 mole of Ca^{2+}. Partial regeneration with less NaCl may be more economical.

Successful operation will depend on completion of the reactions described by Eqs. (20.29)–(20.33) in which adequate contact of the water with the exchange media is most important. To achieve maximum contact, the water should not be turbid since undesirable coating of the exchanger particles would result. In addition, fouling of exchange surfaces or underdrain systems can occur. The exchange medium should be used for softening and not for filtering. Iron in the soluble state will be removed in a manner similar to that for calcium and magnesium. However, aeration of the water will result in precipitation of the iron and fouling of the exchange medium. Other factors for maximum contact include flow rate, depth of bed, and character of exchanger.

A cation exchanger is similar to a sand filter. There is a container to hold the medium and a means to pass water to be softened and salt for regeneration through the exchanger. The tank can be open for gravity flow or closed for pressure flow. Pressure units can, in many cases, eliminate the need for double pumping of the water. The flow direction may be either up or down. Downflow softeners are most widely used, as they may be operated efficiently at varying flow rates, whereas upflow softeners should be operated at a constant rate. A typical downflow unit is shown in Fig. 20.24. The size of the softener will depend primarily on the character of the water and the exchange material, the maximum volume of softened water per unit time, and the volume of water required between regenerations. Consideration should be given to the disposal of the used regenerant brine solutions so as not to pollute the receiving waters.

Effluent from the cation exchanger will have practically zero hardness until the exchange capacity is approached. This can be detected by testing the effluent for hardness. When the hardness begins to increase, the unit needs to be regenerated. The time between regenerations can also be calculated if the following are known: raw water hardness, volume of exchanger medium, and softening capacity per unit volume.

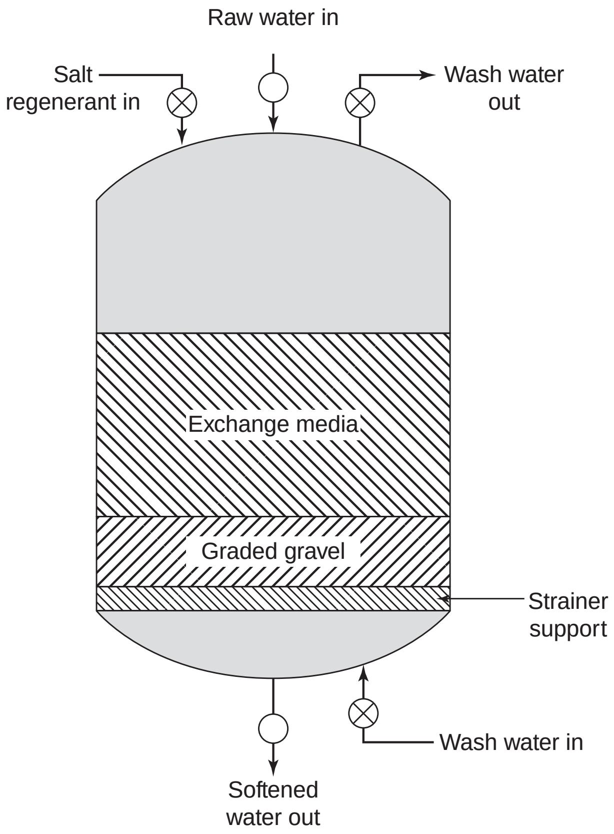


Figure 20.24 Ion exchange column for water softening (*Source*: US EPA).

20.15 DEMINERALIZATION

The deionization of water is generally effected in a two-step process, in which the water is passed successively through a cation exchanger in the H^+ form, $\{H^+R^-\}$, and an anion exchanger in the OH^- form, $\{R^+OH^-\}$. On entering the cation exchanger, all cations are exchanged for an equivalent quantity of H^+ ions. The effluent, actually a solution of the acids of the anions, enters the anion exchanger, where all anions are exchanged for hydroxide ions that neutralize the equivalent quantity of H^+ formed in the cation exchanger. An equivalent amount of water is produced. A mixed-bed exchanger (Fig. 20.25) is an alternative for water demineralization. A single column contains a mixture of equivalent quantities of cation and anion exchangers. The effluent is generally superior in quality (lower in conductivity). To regenerate a mixed bed, the resins must be separated. This can be done by differential backwashing, because cation and anion exchange resins normally have different densities. Demineralizing may be as effective as distillation. However, nonelectrolytes (organic materials) are not quantitatively removed in the ion exchange process, even though there may be some removal through adsorption.

The cation exchange resins used in the demineralization process are regenerated with strong acids. H_2SO_4 is generally used, although it occasionally precipitates $CaSO_4$ in the exchange bed. If weak acids such as carbon dioxide are to be removed, strongly basic anion exchangers must be employed. These must be regenerated with sodium hydroxide. Some weakly basic anion exchangers can be regenerated with soda ash.

EXAMPLE 20.6 CALCULATION OF TIME BETWEEN REGENERATION OF AN ION EXCHANGE SOFTENING BED

Calculate the time between regeneration of an ion exchange bed treating water at a rate of 100 m^3/day. The following parameters are given for the water softener:

Raw water hardness = 160 mg/L
Softening capacity = 10 g/L, and
Volume of medium = 1.6 m^3.

Solution:

$$\begin{aligned}\text{Total hardness removed between regenerations} &= 1,600\ \text{L} \times 10\ \text{g/L} = 16{,}000\ \text{g of hardness.}\\ \text{Volume of water treated between regenerations} &= (16{,}000\ \text{g} \times 1,000\ \text{mg/g})/(160\ \text{mg/L})\\ &= 100{,}000\ \text{L}\\ &= 100\ \text{m}^3.\end{aligned}$$

$$\begin{aligned}\text{Time between regeneration} &= \text{Volume of water treated/Water flow rate}\\ &= 100\ \text{m}^3/(100\ \text{m}^3/\text{day})\\ &= \mathbf{1\ day}.\end{aligned}$$

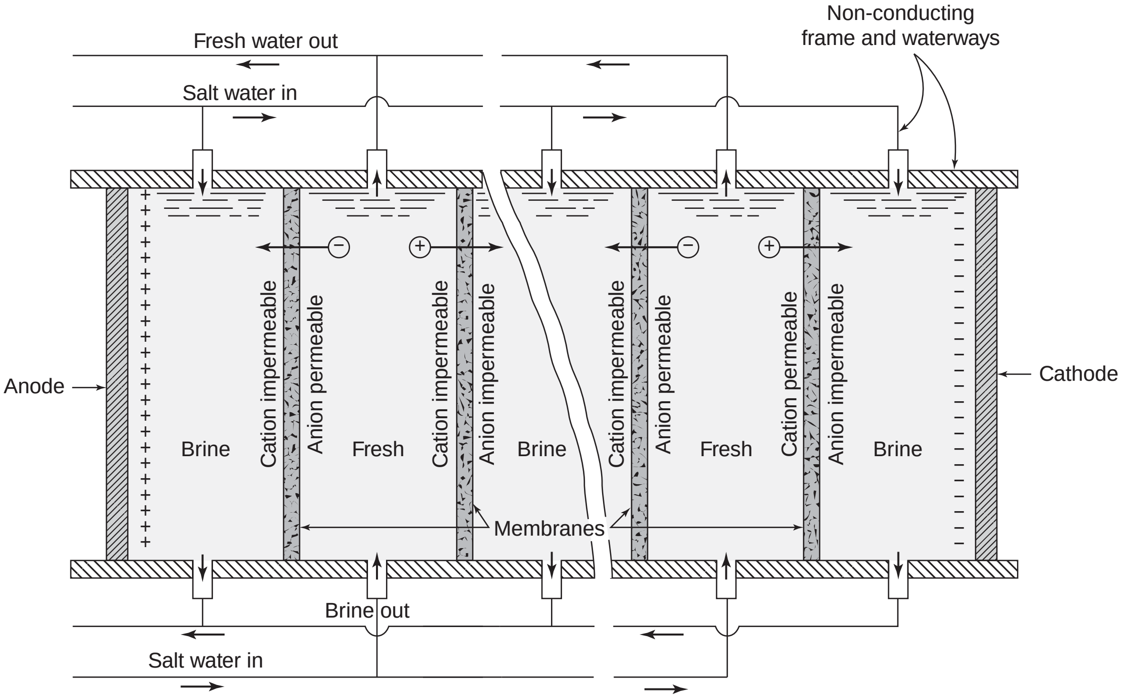


Figure 20.25 Schematic drawing of electrochemical desalting of water (After Fair et al., 1971).

Demineralization produces high-quality water for industry, especially for service as makeup water in steam-power plants. Ion exchange resins are commonly employed only for supplies with less than 500 mg/L of dissolved solids. The introduction of weak-electrolyte exchange resins of high capacity and high regeneration efficiency has raised the economy of treating waters up to concentrations of dissolved solids of 1,000–2,000 mg/L.

20.16 CONCENTRATION OF IONS

Ion exchange fosters the concentration, isolation, and recovery of ionic materials from dilute solutions. There can be savings in treatment chemicals, and process waters can be recycled. Wastewaters are reduced in quantity and strength, and by-products may possibly be worth reclaiming. For example, cation exchangers can recover and concentrate copper, zinc, and chromic acid from dilute washwaters for reuse in metallurgical processing; anion exchange resins can concentrate cyanide and fatty acids from a number of different waste streams; and ion exchange can play an important role in the decontamination of radioactive wastes in general.

Ion exchange is highly selective. It is possible to synthesize resins that incorporate chelating reagents, such as EDTA (ethylenediaminetetraacetate), into the polymeric network. This should make exchangers more selective for specific metal ions such as nickel, copper, and cobalt.

Under well-chosen conditions, fluoride removal by hydroxylapatite, $Ca_{10}(PO_4)_6(OH)_2$, exemplifies a reasonably selective ion exchange process. Hydroxylapatite is converted into fluoroapatite, $Ca_{10}(PO_4)_6F_2$, and OH^- ions are released. Degreased, protein-free bone contains hydroxylapatite and will defluoridate water. The exchanger is regenerated with NaOH.

20.17 ION EXCHANGE MEMBRANES AND DIALYSIS

Water can be desalinized electrochemically by electrodialysis through membranes selectively permeable to cations or anions. Dialysis is the fractionation of solutes made possible by differences in the rate of diffusion of specific solutes through porous membranes. Semipermeable membranes are thin barriers that offer easy passage to some constituents of a solution but are highly resistant to the passage of other constituents. Highly selective membranes have been prepared by casting ion exchange resins as thin films. Membranes made from cation exchange resins are cation permeable; those made from anion exchange resins are anion permeable. Dialytic processes are common separation techniques in laboratory and industry. The recovery of caustic soda from industrial wastes, such as viscose press liquor from the rayon industry and mercerizing solutions, is an example of continuous-flow dialysis.

Diffusion of ions through membranes can be accelerated by applying a voltage across the membrane. The resulting electrodialytic separation with ion exchange membranes permits the desalination of brackish waters. As shown schematically in Fig. 20.26, a series of chambers includes alternating anion-permeable and cation-permeable membranes, together

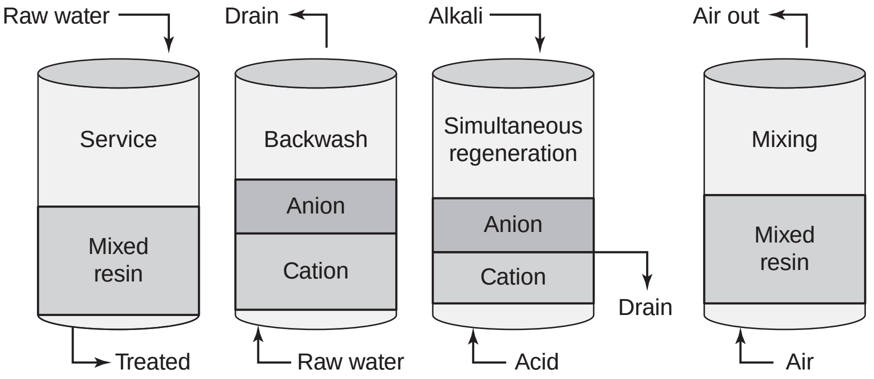


Figure 20.26 Principle of mixed-bed ion exchange (*Source*: US EPA).

with inert electrodes in the outermost compartments. Water introduced into alternate chambers is demineralized by passage of a direct current through the battery of compartments. The applied voltage drives anions toward the anode and out of cells with anion-permeable membranes on the anode side. However, the anions are trapped in the adjacent cell because it has a cation-permeable (anion-impermeable) membrane on the side facing the anode. In this way, compartments from which anions migrate toward the anode also lose cations toward the cathode, while alternate compartments retain both anions and cations. If saline water is fed to the ion-losing compartments and brine is bled from the ion-concentrating compartments, the water can be demineralized electrochemically in continuous flow. Power loss is minimized if the water is demineralized only partially to final concentrations of less than 500 mg/L in a multicompartment cell. Necessary power rises with increasing salinity. By contrast, the cost of distillation and freezing does so only slightly. For this reason only water containing less than 5000–10,000 mg/L of dissolved solids are normally desalted electrochemically.

20.18 MODULAR TREATMENT UNITS FOR REMOVAL OF RADIONUCLIDES

The US Department of Energy (DOE) has developed compact processing units (CPUs), or “modular treatment units,” which are relatively small mobile equipment modules. They perform unit chemical process operations. The CPUs allow rapid deployment of technologies for the treatment of radioactive contaminants in water. The modules would be manufactured off-site by commercial vendors and moved into place using trucks or special transports. The concept of having standardized modules is based on the notion that various radioactive water treatment subsystems could be standardized to match the CPU hardware package, leading to more rapid, cost-effective deployment. The cost–benefits are realized even when multiple units are deployed to achieve greater processing rates. The modular design concept will also allow for reuse of CPU components for different unit processes or process deployments.

Another DOE technology, the resorcinol-formaldehyde ion exchange (ReFIX) resin, was developed for the particular use of treating high-level streams containing cesium salt solutions.

The cost for such a typical ion exchange system ranges from USD 0.08 to 0.21/1,000 L (USD 0.30–0.80/1,000 gal) treated. According to DOE, the key cost factors include

- Pretreatment requirements
- Discharge requirements and resin utilization
- Regenerant used and efficiency

20.19 CASE STUDY I: NITRATE REMOVAL: MCFARLAND, CA

This case involves a 0.044 m^3/s (1 MGD) plant with four groundwater wells contaminated with nitrates from agricultural application of fertilizers and manure. Influent concentrations of nitrate ranged from 6.8 to 22.1 mg/L and averaged 16 mg/L.

To address this contamination problem, ion exchange units were selected for wellhead application because they are effective and easy to operate. The treatment process included

- Anion exchange with A-101-D, Duolite resin
- Sodium chloride regeneration with slow rinse and declassification
- Aerated lagoons and spray irrigation for brine waste treatment

The ion exchange units used three reaction basins, each measuring 1.8 m (6 ft) in diameter and 3 m (10 ft) in height. The standard operational height of the reaction basin is only 0.9 m (3 ft), with operational maximums of 1.5 m (5 ft). According to system design, one of the basins undergoes regeneration, while the other two operate. The plant uses a 2.5-min EBCT (empty bed contact time); the treated water flow rate was 15.77 L/s (250 gpm) with surface loading rates of 6.13 $L/s/m^2$ (9.03 gpm/ft^2). Treated water was blended with raw water in a 7-to-3 ratio.

Table 20.6 Ion exchange cost components for McFarland, CA (USD 2014)

Construction costs components	USD
Ion exchange unit vessels	238,557
Onsite construction	173,276
Resin	120,910
Engineering	99,050
Brine tank	39,964
Miscellaneous	85,386
Total	757,143
Operation and maintenance costs components	¢/1,000 gal
Salt	8.1
Resin replacement	6.8
Power	4.7
Normal operating and maintenance	4.1
Operating labor	2.8
Miscellaneous	1.7
Total	28.2

Source: US EPA.
Conversion factor: 1 ¢/1,000 gal = 0.264 ¢/1000 L.

The regeneration process used a 6% sodium chloride brine. Regeneration involved quick rinse, slow rinse, and resin reclassification procedures that required 981 kg (2,162 lb) of salt daily during periods of continuous operation. The process produced saturated brine at a rate of 2.27 L/s (36 gpm) and diluted brine at 12 L/s (190.5 gpm). Brine was discharged to a municipal wastewater treatment plant, where it was diluted by the other waste streams and then placed in aeration lagoons. This aerated solution was spray-irrigated onto animal feed crops and cotton.

The treated water had nitrate levels of 2–5 mg/L. The blended finished water nitrate levels ranged from 6 to 10 mg/L with a 7 mg/L average.

The total construction costs for the ion exchange units were USD 757,143 in 2014 dollars. The operating and maintenance costs totaled 7.4 ¢/m^3 (28.2 ¢/1,000 gal) of treated water. Table 20.6 shows the components of both types of costs. These costs include the annual loss of 20% of the resin.

20.20 CASE STUDY II: FLUORIDE REMOVAL IN GILA BEND, AZ

This case involves a groundwater supply with undesirable levels of fluoride ranging from 4 to 6 mg/L, with a 5 mg/L average. The plant, with an average capacity of 37.8 L/s (600 gpm) and a maximum capacity of 56.77 L/s (900 gpm), was equipped with the following elements to manage the fluoride contamination:

- Activated alumina: Alcoa activated alumina, grade F-1, –28 to + 48 mesh
- Caustic regeneration
- Acid neutralization
- Evaporation pond for regenerant waste treatment

The treatment process produces 90% treated finished water and 10% wastewater. The finished water averages 0.7 mg/L of fluoride, with a maximum of 1.4 mg/L.

The alumina medium was placed in two vessels 3 m (10 ft) in diameter and height. The alumina takes up 1.5 m (5 ft) of vessel height and expands about 50% of its original height during backwash operations. Approximately 15 cm (6 in.) of basin freeboard is provided. The water's superficial residency time is 5 min. The maximum operational flow rate in the basins is 4.75 L/s/m^2 (7 gpm/ft^2), while the backwash rate is 7.47 L/s/m^2 (11 gpm/ft^2).

For every 13.2–15.1 thousand cubic meters (3.5–4 MG) of water treated, a 10-h regeneration cycle is required. Annual losses of alumina due to regeneration range from 10% to 12%. Regeneration of the alumina medium is accomplished with a 1% solution of sodium hydroxide. The 1% solution flows through the basins at a maximum rate of 1.70 L/s/m^2 (2.5 gpm/ft^2), with a detention time of 24 min. The regeneration process uses 757 L (200 gal) of sodium hydroxide solution per lb fluoride removed.

The caustic water from the regeneration process requires a 0.04% solution of sulfuric acid for neutralization. The acid solution is derived by diluting a bulk 93% acid solution. (The neutralization process flow rate is 4.75 L/s/m^2 [7 gpm/ft^2] at most.) The goal of the neutralization process is to produce acceptable pH levels from 6.5 to 8.5 for disposal. The backwash and neutralization rinse water wastes are discharged to the sewer. The regenerant waste is discharged to a lined evaporation pond which is 73 m (240 ft) by 134 m (440 ft) by 2.7 m (9 ft).

The construction costs for the unit totaled USD 760,000 in 2014 dollars and included treatment facility, well, 1,892,500 L (500,000 gal) steel tank, evaporation pond, booster pumps, standby generator, and chlorination facilities. The plant's operating costs were 23.5–24.8 ¢/m^3 (88–93 ¢/1,000 gal) of treated water. These costs included salaries, power, chemicals, and media replacement.

PROBLEMS/QUESTIONS

20.1 There are four consecutive steps in the adsorption of dissolved materials in water and wastewater by the adsorbent. Name and define these steps.

20.2 What are the factors that bear an influence on adsorption?

20.3 What are the three adsorption isotherms that define conditions of equilibrium?

20.4 State the characteristics of adsorbents and explain their industrial classes.

20.5 Explain the methodologies for manufacturing and activation of activated carbon.

20.6 Pilot carbon column tests are performed for the purpose of obtaining design data for full-scale plant construction. What are the things that are made to be possible by pilot column tests?

20.7 Explain the thermal regeneration process for activated carbon.

20.8 Define the resin exchange capacity and list the factors which affect the design of an ion exchange system.

20.9 What are the advantages and disadvantages of the ion exchange process?

20.10 What are the characteristics of an effective ion exchanger?

20.11 Use chemical equations to describe the ion exchange process.

20.12 List and explain the various factors that affect the selectivity of an ion exchanger.

20.13 Describe, using chemical equations, the process of water softening by ion exchange.

20.14 Describe the process of water demineralization by ion exchange.

20.15 Calculate the total volume of water that can be treated between regenerations of an ion exchange water softening bed. The following parameters are given for the water softener:

Raw water hardness = 200 mg/L,
Softening capacity = 12 g/L, and
Volume of medium = 2 m^3.

20.16 Calculate the time between regeneration of an ion exchange bed treating water at a rate of 60 m^3/day. The following parameters are given for the water softener:

Raw water hardness = 200 mg/L,
Softening capacity = 12 g/L, and
Volume of medium = 2 m^3.

20.17 Well water has a TOC concentration c_0 of 400 mg/L. It is to be treated by activated carbon. Data obtained in laboratory batch testing are shown below. Determine the Freundlich isotherm constant *K* and *n*.

Mass of carbon (mg/L)	TOC in effluent (mg/L)
520	322
2,320	117
3,460	51
3,840	39
4,500	23
5,400	12
6,670	6.1
7,600	4.2
8,820	1.1

20.18 A water flow of 200 m^3/day with 107 mg/L of TOC is to be treated by an activated carbon adsorption column. The allowable effluent concentration, c_a, is 5% of c_0. The data from a breakthrough test column are as follows:

Throughput volume (L)	TOC concentration (mg/L)
159	4.45
169	9.85
181	17.16
191	27.56
195	40.03
200	49.56
207	62.90
212	68.89
220	86.41
225	94.03
234	98.17
240	102.93
260	107.00

The characteristics of the test column are

- Cross-sectional area = 5.0 cm^2
- Depth of carbon bed = 60 cm
- Mass of carbon = 120 g
- Applied flow rate = 10.8 L/day
- Packed carbon density = 400 kg/m^3

Using the kinetic expression

$$\frac{c}{c_0} = \frac{1}{1 + e^{\frac{k_1}{Q}(q_0 M - c_0 V)}}$$

for the adsorption column design and bed depth four times the column diameter, determine

1. The reaction constant, *k*
2. The maximum solid-phase concentration (gm TOC/gm of carbon)
3. The breakthrough time (days)
4. Mass of carbon required for the design column (kg)
5. The diameter and depth of carbon bed required

20.19 There are two types of ion exchange resins: cationic and anionic. Cationic ion exchange resins exchange positive ions, such as bivalent calcium, bivalent magnesium, and trivalent iron, and replace them with sodium ions available in the ion exchange bed. When the cationic ion exchange bed is nearly exhausted, the bed can be regenerated with sodium chloride solution. This is the common ion exchange softening process. Write chemical equations for the stated ion exchange softening process.

20.20 Recommend a flow diagram of the ion exchange process system described in Example 11.5.

20.21 Inorganic impurities, such as fluoride arsenic and selenium, can be removed by activated alumina ion exchange process. Inorganic chemical removal occurs through absorption on the activated alumina filter bed. The process equipment of activated alumina is similar to a pressure granular filter bed. Recommend a flow diagram of an activated alumina water treatment system for treating a groundwater containing 4–6 mg/L of fluoride and some coliforms. All other water quality parameters of the groundwater meet the government's drinking water standards. Discuss the applications, limitations, and process regeneration of this process. Recommend the detailed schematics of the pressure-activated alumina unit during operation and during regeneration.

20.22 Anionic ion exchange resins (anion exchanger) exchange negative ions, such as nitrate, sulfate, and chloride ions, and replace them with hydroxide ions available in the anionic ion exchange bed. When the anionic ion exchange bed is nearly exhausted, the bed can be regenerated with sodium hydroxide solution. This is the common ion exchange denitrification process aiming at removal of excessive nitrates from groundwater. Write chemical equations for the stated ion exchange denitrification process, assuming the weakly basic anion exchanger is used.

20.23 Recommend a flow diagram of the ion exchange denitrification process as described in Example 20.22.

20.24 Sodium cation exchanger Na_2R is commonly used for softening, while hydrogen cation exchanger H_2R is commonly used for demineralization (removal of all cations). Write chemical equations showing how sodium is removed by H_2R and how the spent cation exchanger is regenerated by sulfuric acid.

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Chapter 21

Chemical Stabilization and Control of Corrosion and Biofilms

21.1 CHEMICAL STABILIZATION

Within the frame of reference of this book, *chemical stabilization* is the adjustment of the pH, $[Ca^{2+}]$, and alkalinity of water to its $CaCO_3$ saturation equilibrium. Because stabilized water neither dissolves nor precipitates $CaCO_3$, it will neither remove coatings of $CaCO_3$ that may protect pipes against corrosion, nor lay down deposits of $CaCO_3$ that may clog pipes. *Equilibrium* conditions can be defined by the equations:

$$CaCO_3(s) + H^+ = Ca^{2+} + HCO_3^- \qquad \textbf{(21.1)}$$

$$[Ca^{2+}][HCO_3^-]/[H^+] = K \qquad \textbf{(21.2)}$$

By convention, the symbol (s) indicates that the substance is present as a solid and K is the equilibrium constant. Incidentally, pure solid substances have an activity of 1 in equilibrium expressions.

The $CaCO_3$ *saturation point* can also be characterized by

$$[H^+]_{eq} = [H^+]_s \qquad \textbf{(21.3)}$$

or

$$pH_{eq} = pH_s \qquad \textbf{(21.4)}$$

respectively the hydrogen ion concentration or pH at the hypothetical equilibrium or saturation with $CaCO_3$.

Hence

$$[H^+]_{eq} = [Ca^{2+}][HCO_3^-]/K \qquad \textbf{(21.5)}$$

Below pH_{eq}, or when $[H^+] > [H^+]_{eq}$, no $CaCO_3$ will be deposited. Some natural waters of low alkalinity and hardness and of high CO_2 content, as well as coagulated, ion-exchange-softened, or demineralized waters, fall into this category. Above pH_{eq}, or when $[H^+] < [H^+]_{eq}$, $CaCO_3$ will precipitate. Carbonate deposits may then accumulate in distribution mains, boilers, and other equipment, and on sand and gravel in water filters. To provide a measure of the stability of a given water, Langelier (1946) proposed calling the difference between the measured pH and the calculated or determined equilibrium pH value ($pH_s = pH_{eq}$) the *saturation index I*:

$$I = pH - pH_s \qquad \textbf{(21.6)}$$

When $I = 0$, the water is in equilibrium; when I is positive, the water is oversaturated; and when I is negative, the water is undersaturated or aggressive.

The equilibrium constant of Eq. (21.2) is given by the ratio of the mass-law expression for Reaction 4 to Reaction 3 in Table 21.1, namely,

$$[Ca^{2+}][HCO_3^-]/[H^+] = K_S/K_2 \qquad \textbf{(21.7)}$$

and the pH for $CaCO_3$ saturation becomes

$$pH_s = \log K_s/K_2 - \log[Ca^{2+}] - \log[HCO_3^-] \qquad \textbf{(21.8a)}$$

$$pH_s = pCa^{2+} + (pK_2 - pK_s) + p(Alk) \qquad \textbf{(21.8b)}$$

where $p(Alk) = p[HCO_3^-]$.

The concentration of HCO_3^- can be calculated from analytically more readily determinable parameters than HCO_3^- itself. For example,

$$[HCO_3^-] = \{T - K_W/[H^+] + [H^+]\}/\{1 + 2K_2/[H^+]\} \qquad \textbf{(21.9)}$$

where T is the total alkalinity (eq/L). Below pH 8.5, $[HCO_3{}^-] \approx T$.

The saturation index can be evaluated by using the following simplified relationship and Table 21.1:

$$I = pH - (TF + DF - HF - AF) \qquad \textbf{(21.10)}$$

where,

I = Langlier Saturation Index
pH = actual water pH
TF = temperature factor; use value from Table 21.1
HF = calcium hardness factor; use value from Table 21.1

Water Engineering: Hydraulics, Distribution and Treatment, First Edition. Nazih K. Shammas and Lawrence K. Wang.

Table 21.1 Table for calculation of Langelier saturation index

Temperature		Calcium hardness		Total alkalinity		Total dissolved solids	
°C	TF Temperature factor	mg/L (as $CaCO_3$)	HF Hardness factor	mg/L (as $CaCO_3$)	AF Alkalinity factor	mg/L	DF Dissolved solids factor
0	2.60	10	1.00	10	1.00	0	9.7000
4	2.50	20	1.30	20	1.30	25	9.7175
8	2.40	30	1.48	30	1.48	50	9.7350
10	2.35	40	1.60	40	1.60	100	9.7700
12	2.30	50	1.70	50	1.70	150	9.8000
14	2.25	60	1.78	60	1.78	200	9.8300
16	2.20	70	1.84	70	1.84	250	9.8375
18	2.15	80	1.90	80	1.90	300	9.8450
20	2.10	100	2.00	100	2.00	350	9.8525
25	2.00	200	2.30	200	2.30	400	9.8600
30	1.90	300	2.48	300	2.48	450	9.8675
35	1.80	400	2.60	400	2.60	500	9.8750
40	1.70	500	2.70	500	2.70	550	9.8825
45	1.62	600	2.78	600	2.78	600	9.8900
50	1.55	700	2.84	700	2.84	700	9.8925
60	1.40	800	2.90	800	2.90	800	9.8950
70	1.25	900	2.95	900	2.95	900	9.8975
80	1.15	1,000	3.00	1,000	3.00	1,000	9.9000

AF = alkalinity factor; use value from Table 21.1

DF = total dissolved solids (TDS) factor; use value from Table 21.1

TF + DF − HF − AF = pHs

If the saturation index is between ± 0.3, the water is in balance and is stabilized; if $I > 0.3$ the water is oversaturated and scale forming; and if $I < -0.3$ the water is undersaturated and corrosive.

EXAMPLE 21.1 COMPUTATION OF LANGLIER SATURATION INDEX USING TRADITIONAL AND SIMPLIFIED METHODS

What is the Langelier Index for neutral water (pH = 7) with calcium hardness of 100 mg/L alkalinity equal to 150 mg/L as calcium carbonate and TDS equal 800 mg/L at a temperature of 10°C? Please discuss the results.

Solution 1 (Traditional Method):

The number of moles of calcium (Ca^{2+})/L = (100 mg/L $CaCO_3$) × (1 mol $CaCO_3$/100 g) × (1 g/1,000 mg) = 0.001 mol/L

$$pCa^{2+} = -\log Ca^{2+} = -\log 0.001 = -(-3) = 3$$

The alkalinity must be expressed in equivalents per liter. The equivalent weight of calcium carbonate is 50, thus

$$\begin{aligned}(\mathrm{Alk}) &= 150\ \mathrm{mg/L}/(50\ \mathrm{eq/g}) \times (1\ \mathrm{g}/1{,}000\ \mathrm{mg}) = 0.003\ \mathrm{eq/L}\\ \mathrm{P(Alk)} &= -\log(\mathrm{Alk}) = -\log 0.003 = -(-2.52) = 2.52\end{aligned}$$

The value of $pK_2 - pK_s$ is found from Appendix 26 (using TDS = 800 mg/L and temperature = 10°C) to be 2.71

Entering the above values into Eq. 21.8b for the pH of saturation:

$$\begin{aligned}pH_s &= pCa^{2+} + (pK_2 - pK_s) + p(\mathrm{Alk})\\ pH_s &= 3.0 + 2.71 + 2.52 = 8.23\end{aligned}$$

and from Eq. 21.6

$$\mathrm{I} = pH - pH_s = 7 - 8.23 = \mathbf{-1.23}$$

Solution 2 (Simplified method):

From Table 21.1

$$\begin{aligned}\text{Water temperature} &= 10^{\circ}\text{C; TF} = 2.35\\ \text{Total dissolved solids} &= 800\ \text{mg/L as CaCO}_3\text{; DF} = 9.8950\\ \text{Calcium hardness} &= 100\ \text{mg/L as CaCO}_3\text{; HF} = 2.00\\ \text{Alkalinity} &= 150\ \text{mg/L as CaCO}_3\text{; AF} = 2.15\\ \text{I} &= \text{pH} - (\text{TF} + \text{DF} - \text{HF} - \text{AF})\\ &= 7.0 - (2.35 + 9.8950 - 2.00 - 2.15)\\ &= 7.0 - (8.19)\\ &= \mathbf{-1.19}\end{aligned}$$

Discussion:

The general and detailed indications for the Langiere Saturation Index (I) are presented below.

a. General indications:

I < 0	Water is under saturated with respect to calcium carbonate. Undersaturated water has a tendency to remove existing calcium carbonate protective coatings in pipe lines and equipment.
I = 0	Water is considered to be neutral. Neither scale-forming nor scale-removing
I > 0	Water is supersaturated with respect to calcium carbonate and scale forming may occur.

b. Detailed indications:

I = −2.0 to <−0.5	Serious corrosion
I = −0.5 to <0	Slightly corrosive but non-scale forming
I = 0.0	Balanced but pitting corrosion possible
I = 0.0 to <0.5	Slightly scale-forming and corrosive
I = 0.5 to <2.0	Scale-forming but noncorrosive

Since the calculated I is −1.19 using the simplified method (or −1.23 using the traditional method), the water is undersaturated with respect to calcium carbonate. Undersaturated water has a tendency to remove existing calcium carbonate protective coatings in pipe lines and equipment. Serious corrosion may occur.

21.2 CORROSION

The corrosion of metals is a complex chemical or electrochemical phenomenon. In water and wastewater systems, corrosion results in (a) reduced hydraulic carrying capacity of pipes, fittings, and other systems components; (b) possible structural failures; and (c) debased quality of the transported water.

Corrosion occurs because metals tend to oxidize when in contact with potable water and form stable solids on metal surfaces. All metals in contact with water will corrode to some extent. Corrosion has implications for health, costs, and aesthetics. Drinking water contaminated with metals adversely affects human health. Corrosion reduces the useful life of water distribution systems and household plumbing and is thus responsible for higher costs due to problems with

1. Pumping caused by narrowed pipe diameters resulting from corrosion deposits
2. Pumping and water production caused by corrosion holes, which reduce water pressure and increase the power required to deliver a gallon of water to the point of consumption
3. Water damage caused by corrosion-related pipe failures
4. Replacement frequency of hot water heaters, radiators, valves, pipes, and meters
5. Customer complaints of color, staining, and taste problems
6. Repairs for pipe leaks and breaks

All water is corrosive to some degree, but water that is acidic will have faster corrosion rates. Many naturally occurring acidic waters are also soft. Soft water is generally defined as water with less than 100 mg/L of calcium as calcium carbonate ($CaCO_3$); acidic water has a pH of less than 7.0.

Loss in carrying capacity by corrosion and tuberculation (knoblike mounds) is economically as well as operationally important. A comprehensive survey of water systems before the days of successful corrective treatment of aggressive waters showed, for example, that tar-coated, cast-iron pipe carrying the relatively soft waters of New England lost about half its capacity in 30 years of service. Then, pumped supplies might see their power requirements greatly increased and gravity systems might have to meet failing capacities by adding new conduits. Capacity loss through other than direct corrosion is produced by scale formation, biological growths, and silt deposition. Other indirect effects of corrosion are the adsorption of organic substances on corrosion products and encouragement of microscopic growths, the creation of chlorine demands—often high enough to exhaust chlorine residuals, and the production of odors, tastes, and other objectionable changes in water quality. In systems that include iron piping, red water is produced. However, the appearance of red water is not necessarily a measure of the degree of corrosion. Depending on pH, electrolyte content, oxygen concentration, and velocity of flow, coatings and tubercles may incorporate different proportions of the corrosion products. In some circumstances corrosion, although extensive, may be hidden, because virtually all the corroded iron may be deposited on the interior of the pipe. In other circumstances corrosion, although low, may be visible, because the corroded iron remains in suspension within the flowing waters.

A dry-cell battery is a reasonable analog model of the corrosion of metals in water. The cell is composed of two electrodes (C and Zn in the common flashlight battery) separated from each other by an electrolyte, such as a solution of NH_4C1. In the cell there is chemical reduction at the cathode (the zinc electrode) and chemical oxidation at the anode, the carbon electrode. Electrical energy is generated by chemical reactions at each electrode. The more intense the reactions, the greater the flow of current when the circuit is closed. Most metals contain minute amounts of impurities, and their surfaces are neither chemically nor physically homogeneous. Instead, they are composed, in essence, of arrays of microcells with anodic and cathodic microareas. Because of this, exposure of a metal to an aqueous solution permits chemical reductions and oxidations to take place. By convention, the electricity flowing through the metal, that is, the external circuit, is called a positive current even though the electrons moving through the metal carry a negative charge.

21.3 THE CORROSION REACTION

There are dozens of types of corrosion, but the two broad categories of particular concern in water treatment are uniform and nonuniform corrosion. Nonuniform corrosion shortens the useful life of pipes and plumbing more quickly than uniform corrosion and is thus more of a problem for water systems. The five most common types of nonuniform corrosion to affect water systems are

1. *Galvanic*: Occurs when two different metals are joined together; the more electrochemically active of the joined metals will corrode.
2. *Pitting*: Occurs as uneven pits or holes in the pipe surface, which are undetectable until the pipe fails. Pitting is usually caused by pockets of corrosion initiated by tiny imperfections, scratches, or surface deposits in the pipe.
3. *Crevice*: Occurs locally around gaskets, lap joints, rivets, and surface deposits. It is caused by changes in acidity, oxygen concentrations, dissolved ions, and the absence of corrosion inhibitors.
4. *Erosion*: Occurs due to the removal of protective coatings through high water velocities, turbulence, sudden changes in flow direction, and abrasive action of suspended solids or gases.
5. *Biological*: Occurs in mechanical crevices or accumulations of corroded materials due to the interaction between the metal and bacteria, algae, or fungi.

The overall corrosion reaction can be formulated in single chemical equations. Table 21.2 lists some of them, together with pertinent values of electromotive force, emf. The larger the potential, the greater, in general, is the tendency of the reaction to proceed. For instance, iron is seen to be less corrodible than magnesium, but more so than copper. That the standard emf for gold is negative means that gold does not corrode in water.

By definition, the standard emf is the potential in volts of an ideal electrochemical cell in which the reaction under consideration takes place at unit activity of all reacting substances. Accordingly, the emf is the algebraic difference between the electrode potentials of any two half-reactions so combined as to cancel the electrons and yield the overall (cell) reaction.

When the emf is positive, the reaction tends to proceed in the direction shown in Table 21.2. However, reaction tendency is not reaction rate. Large positive values of the standard emf may not signify rapid corrosion, but negative

Table 21.2 Electromotive force of corrosion reactions[a]

Reaction	E^0 (25°C), (V)
$Mg(s) + 2H^+ = Mg^{2+} + H_2(g)$	+2.37
$Zn(s) + 2H^+ = Zn^{2+} + H_2(g)$	+0.76
$Zn(s) + H_2O = ZnO(s) + H_2(g)$	+0.42
$Zn(s) + \frac{1}{2}O_2(g) = ZnO(s)$	+ 1.65
$Fe(s) + 2H^+ = Fe^{2+} + H_2(g)$	+0.41
$Fe(s) + \frac{1}{2}O_2(g) + 2H^+ = Fe^{2+} + H_2O$	+1.77
$Fe(s) + H_2O + \frac{1}{2}O_2(g) = Fe(OH)_2(s)$	+1.27
$Cu(s) + 2H^+ = Cu^{2+} + H_2(g)$	−0.34
$Cu(s) + \frac{1}{2}O_2(g) = CuO(g)$	+0.66
$Cu(s) + \frac{1}{2} O_2(g) + H_2O = Cu(OH)_2(s)$	+0.62
$Au(s) + \frac{3}{2} H_2O + \frac{3}{4}O_2(g) = Au(OH)_3(s)$	−0.23

Source: Fair et al. (1971).

[a] Here (s) stands for solid and (g) for gas.

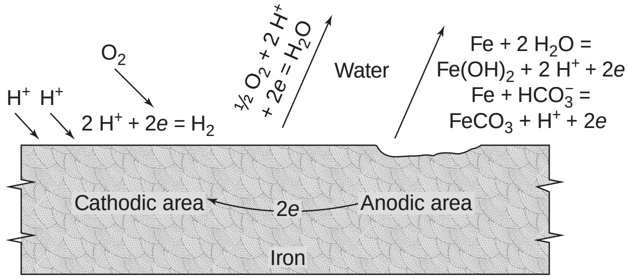

Figure 21.1 Corrosion cell on the surface of iron in water (After Fair et al., 1971).

values do imply that the reaction will not proceed under the conditions described. In a thermodynamic sense the metals commonly used in water and wastewater systems are bound to corrode in aqueous environments, that is, to return, in essence, to the condition in which they were won from the soil.

Depending on conditions of exposure, both oxidation and reduction reactions can be involved. Reactions associated with the existence of microcells are illustrated in Fig. 21.1 The *feedback* of electrons is seen to maintain the corrosion process. The corrosion reactions illustrated in Fig. 21.1 have been written to show the passage of two electrons through the metal in each instance.

Contact between dissimilar metals, or the existence of areas of dissimilar oxidation potentials in the same metal, normally promotes corrosion. A galvanic cell is formed, and the rate of corrosion is stepped up. The anodic metal or area possessing the highest oxidation potential corrodes; the cathodic metal or area does not corrode. The relative behavior of different metals may be gauged from the electromotive force series of Table 21.2 or, better, from a galvanic series that takes into account the environmental conditions normally encountered in water. Table 21.3 is such a table.

To give an example, zinc is anodic to both copper and iron, and iron, in turn, is anodic to copper. Among other things, this accounts for (a) the dezincification of yellow brass, an alloy of copper and zinc, (b) the reduced corrosion of iron in galvanized-iron pipes for as long as the zinc coating lasts, and (c) red or rusty water issuing from a bronze faucet attached to an iron pipe.

Table 21.3 Galvanic series–order of electrochemical activity of common metals used in water distribution system

Metal	Activity
Zinc	More active
Mild steel	
Cast iron	
Lead	
Brass	
Copper	
Stainless steel	Less active

Source: US EPA.

Corrosion of cast-iron pipes produces tubercles of rust and other precipitates above pits in the metal. The pipe surface becomes rough and flow is markedly impeded. Localized corrosion or pitting is common, too, in steel pipes. Moreover, they are so relatively thin that pits soon break through the pipe shell. Copper pipes, too, fail by pitting. Sand, dirt, mill scale, residual lubricants, suspended solids, and iron and manganese, as well as nonhomogeneities at interfaces—scratches on the metal and differences in velocities of flow, for instance—establish concentration cells and promote pitting. Clean pipes and clean water are the answer. Copper corrodes fast in waters of low pH and high concentrations of sulfides, chlorides, and nitrates.

The rate of reduction of oxygen at the cathodic area, which depends on the rate at which oxygen has access to the area, governs the transfer of electrons from the metal to the solution. The area to which oxygen has easiest access tends to become the cathodic area; the area to which oxygen has access with difficulty becomes the anodic area. The resulting corrosion cell is called a *differential-aeration cell* or, in a more inclusive sense, a *differential-concentration cell*. In water supply and wastewater disposal such cells may take many forms. Examples of anodic areas, or areas sheltered against oxygen, are pits or depressions in the metal, areas underlying mill scale or products of corrosion, and areas below biological growths. In water pipes the rate of corrosion slows up in time, owing to the accumulation of rust or similar oxidation products. However, corrosion quickens at high velocities of flow, owing to the more rapid removal of corrosion-retarding substances and the replenishment of corrosion-promoting substances. On the other hand, high flow rates also carry more corrosion-retarding substances to surfaces. If these substances outweigh the corrosion-promoting substances that are present, rates of corrosion may indeed fall off while rates of flow are rising.

The presence of electrolytes other than hardness and alkalinity promotes corrosion because their peptizing action destroys passivating or otherwise protective oxide films. Chloride and sulfate are examples.

Corrosion is more rapid in acid than in neutral or alkaline solutions, but its ultimate extent is often larger in alkaline than in neutral solutions. In the range of pH values generally encountered in water treatment, faster corrosion in acid solutions is related more closely to the stability of protective films on the metal than to the actual rate of corrosion. Although CO_2 or H_2CO_3 does not participate in the electrochemical reaction, the concentration of free CO_2 is an important function of pH at given alkalinities.

Direct-current electricity corrodes the metal of the pole serving as the anode. Hence underground pipes are corroded by stray electrical currents at points where positive electricity leaves the pipe.

In water distribution systems the deterioration of water quality by corrosion is most noticeable in dead-ends. It is there that products of corrosion accumulate and there, too, that oxygen may be absent and sulfides present.

21.3.1 Factors Affecting Corrosion

The degree of corrosion is determined primarily by the characteristics of the metal and water, and the nature and duration of the contact between the two. Table 21.4 summarizes the factors affecting drinking water corrosivity. Water treatment processes can change water quality characteristics that significantly affect the water's corrosion potential. For example, pH is lowered with the use of coagulants or disinfectants. Other treatment processes affect water chemistry parameters such as disinfectant residual, hardness, and alkalinity.

Table 21.4 Factors affecting the corrosivity of drinking water

Factor	Effect on corrosivity
pH	Low pHs generally accelerate corrosion
Dissolved oxygen	Dissolved oxygen in water induces active corrosion, particularly of ferrous and copper materials
Free chlorine residual	The presence of free chlorine in water promotes corrosion of ferrous metals and copper
Low buffering capacity	There is insufficient alkalinity to limit corrosion activity
High halogen and sulfate–alkalinity ratio	A molar ratio of strong mineral acids much above 0.5 results in conditions favorable to pitting corrosion (mostly in iron and copper pipe)
Total dissolved solids	Higher concentrations of dissolved salts increase conductivity and may increase corrosiveness. Conductivity measurements may be used to estimate total dissolved solids
Calcium	Calcium can reduce corrosion by forming protective films with dissolved carbonate, particularly with steel, iron, or galvanized pipe
Tannins	Tannins may form protective organic films over metals
Flow rates	Turbulence at high flow rates allows oxygen to reach the surface more easily, removes protective films, and causes higher corrosion rates
Metal ions	Certain ions, such as copper, can aggravate corrosion of downstream materials. For example, copper ions may increase the corrosion of galvanized pipe
Temperature rates	High temperature increases corrosion reaction rates. High temperature also lowers the solubility of calcium carbonate, magnesium silicates, and calcium sulfate and thus may cause scale formation in hot-water heaters and pipes

Source: US EPA.

The type of corrosion products present depends on the metals composing the solder, pipes, valves, meters, and faucets in distribution and plumbing systems. The most common metals used are steel, iron, galvanized steel, copper, lead, brass, and bronze. Zinc, cadmium, and some lead are present in galvanized coatings. The most frequent sources of lead are brass faucets, meters, pipe solder, and valves. Lead is no longer generally used for pipes, but in older cities service connections between houses and water mains still contain lead pipe. Many cities have initiated lead service line replacement programs to address this source.

While lead pipes have long been recognized as hazardous, lead-based solder was used in the United States until it was banned in the 1986 US Safe Drinking Water Act (SDWA) Amendments. This ban prohibits the use of solder containing lead and the associated high lead levels often found in newly constructed homes and in new plumbing in existing homes. Precise estimates of lead levels in drinking water resulting from lead-based solders and fluxes vary. This stems from the difficulty in consistently measuring lead levels from the tap, especially if there is a brass faucet, which contributes to lead levels as well. The method of solder application affects the amount of lead imparted to the water. Improper application, which is difficult to determine in retrospect, allows solder to flow on to the inner portion of the pipe, thus increasing its area of contact with the water.

21.3.2 Corrosion Indices

Corrosion related to calcium carbonate deposition can be estimated using indices derived from common water quality measures.

1. *Langelier saturation index* (*LSI*) is the most commonly used and is equal to the water pH minus the saturation pH. The saturation pH refers to the pH at the water's calcium carbonate saturation point, the point where calcium carbonate is neither deposited nor dissolved. Calcium carbonate may precipitate and form a protective layer on metals. The saturation pH is related to the water's calcium ion concentration, alkalinity, temperature, pH, and presence of other dissolved solids, such as chlorides and sulfates.
2. *Aggressive index* (*AI*) is a simplification of the LSI that only approximates the solubility of calcium carbonate and may not be useful.
3. *Ryznar stability index* (*RSI*), also a modification of the LSI, uses visual inspections.
4. *McCauley's driving force index* (*DFI*) estimates the amount of calcium carbonate that will precipitate based on the same factors as the LSI.

Table 21.5 Typical consumer complaints due to corrosion

Customer complaint	Possible cause
Red water or reddish-brown staining of fixtures and laundry	Corrosion of iron pipes or presence of natural iron in raw water
Bluish stains on fixtures	Corrosion of copper lines
Black water	Sulfide corrosion of copper or iron lines or precipitations of natural manganese
Foul taste and/or odors	By-products from microbial activity
Loss of pressure	Excessive scaling, tubercle buildup from pitting corrosion, leak in system from pitting or other type of corrosion
Lack of hot water	Buildup of mineral deposits in hot water system (can be reduced by setting thermostats to under 60°C [140°F])
Short service life of household plumbing	Rapid deterioration of pipes from pitting or other types of corrosion

Source: US EPA.

5. *Riddick's corrosion index* (*CI*) is distinct from the LSI. Its empirical equation incorporates different factors to predict corrosion, such as dissolved oxygen, chloride ion, noncarbonate hardness, and silica.

21.3.3 Consumer Complaints

Many times a consumer complaint is the first indication of a corrosion problem. Table 21.5 lists the most common complaints and their causes. In investigating the extent of the corrosion, complaints can be plotted on a map of a water service area. Then investigators can examine the construction materials used in the water distribution system and in the plumbing of the complaint areas. Random sample surveys are commonly used to confirm the extent of the corrosion problems flagged by these complaints.

21.3.4 Scale or Pipe Surface Examination

Pipe scale (see Fig. 21.2) and the inner surface of pipes can be examined by optical or microscopic observation, X-rays, wet chemical analysis, and Raman and infrared spectroscopy. The most practical and economic method is simple observation. The other methods require expensive equipment and highly skilled personnel. Even simple observation, however, requires taking sections of pipe out of service. Chemical examinations can determine the composition of pipe scale, such as the proportion of calcium carbonate present that shields pipes from dissolved oxygen and thus reduces corrosion. Chemical analyses and microscope assistance then should be used to augment these findings.

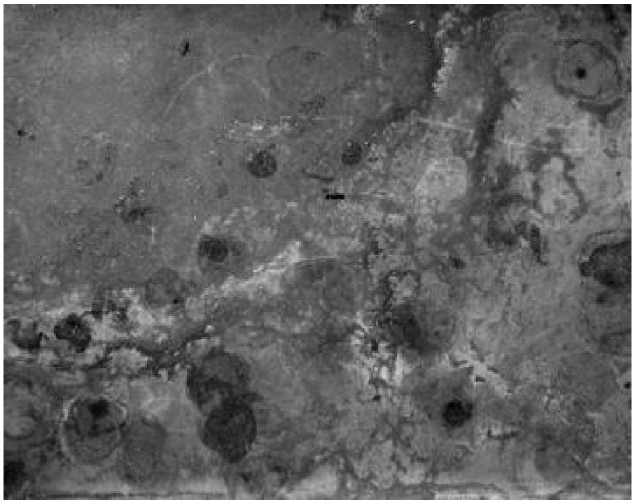

Figure 21.2 Rust, the most familiar example of corrosion (http://en.wikipedia.org/wiki/File:Rust_and_dirt.jpg).

21.3.5 Measurement of Corrosion Rate

The corrosion rate is usually expressed in thousands of an inch (mils) per year (MPY). The three methods for calculating this rate are

1. Coupon weight loss
2. Loop system concentration increase
3. Electrochemical rate measurement

The first two methods are much less expensive than the third.

The coupon weight-loss method uses flat metal pieces or test sections of pipes called coupons or inserts. The flat coupons are commonly placed in the middle of a pipe in the system. The coupons are weighed before and after they are put in place and the recorded weight loss is translated into a uniform corrosion rate with the following formula:

$$R = \frac{534W}{D \times A \times T} \quad \text{(US customary system)} \qquad \textbf{(21.11a)}$$

where

R = rate of corrosion (MPY)
W = weight loss (mg)
D = density of the specimen (g/cm^3)
A = surface area of the specimen (in.^{2})
T = exposure time (h)

An equivalent metric equation for calculation of the rate of corrosion is presented below:

$$\mathrm{R} = \frac{3{,}445W}{D \times A \times T} \quad \text{(SI system)} \qquad \textbf{(21.11b)}$$

where

R = rate of corrosion (MPY)
W = weight loss (mg)
D = density of the specimen (g/cm^3)
A = surface area of the specimen (cm^2)
T = exposure time (h)

This method can be used to monitor corrosion progress over time or to spot check the corrosion rate.

The loop system weight-loss method is similar to the flat coupon method, except that it uses actual sections of pipe in the system instead of flat coupons. Loop system corrosion rates are usually determined by analyzing corrosion over a period of time.

Electrochemical rate measurement requires expensive and sophisticated equipment beyond the means of small water systems. In this method, two or three electrodes are placed in the corrosive environment, and instrumentation measures the corrosion rate in MPY.

21.4 CONTROL OF CORROSION

Selection of corrosion-resistant materials and methods of corrosion control should be directed toward interrupting or otherwise modifying the cycle of corrosion. Commonly employed materials and methods include the following:

1. Corrosion-resistant metals or alloys that either possess potentials in the emf range of noble metals or lay down protective coatings of dense oxides as they corrode. Examples are stainless steel, tin, copper and Monel metal (silvery alloys containing copper, nickel, and smaller quantities of such metals as iron, manganese, and aluminum)
2. Coatings and linings that bar both anodic and cathodic reactions by preventing escape of cations

EXAMPLE 21.2 DETERMINATION OF CORROSION RATE

A metal coupon 1 ft (30 cm) long and 2 in. (5 cm) wide was place in a water pipe for measuring its corrosion rate. The coupon was inserted in the pipe for a period of 3 days. Upon retrieval of the coupon it was weighed and found to have lost 10 mg. Knowing that the coupon density is 7,870 kg/m^3, determine the rate of pipe corrosion in MPY (mil/year).

Solution 1 (US Customary System):

W = weight loss = 10 mg.
D = coupon density = 7,870 kg/m$^{3.}$
= 7,870 × (10^3/10^6) g/cm^3
= 7.87 g/cm^3.
A = surface area of the coupon = 12 × 2 = 24 in.^{2}.
T = exposure time = 3 × 24 = 72 h.

$$R = \frac{534W}{D \times A \times T} = \frac{534\,(10)}{(7.87)(24)(72)} = \mathbf{0.4\,MPY}.$$

Solution 2 (SI System):

W = weight loss = 10 mg.
D = coupon density = 7,870 kg/m^3
= 7,870 × (10^3/10^6) g/cm^3
= 7.87 g/cm^3.
A = surface area of the coupon = 30 × 5 = 150 cm^2.
T = exposure time = 3 × 24 = 72 h.

$$R = \frac{3{,}445W}{D \times A \times T} = \frac{3{,}445 \times 10}{7.87 \times 150 \times 72} = \mathbf{0.4\,MPY}.$$

Note: 1 MPY = 1 mil/yr = 1 × 10^{-3} in/yr = 25.4 × 10^{-3} mm/yr

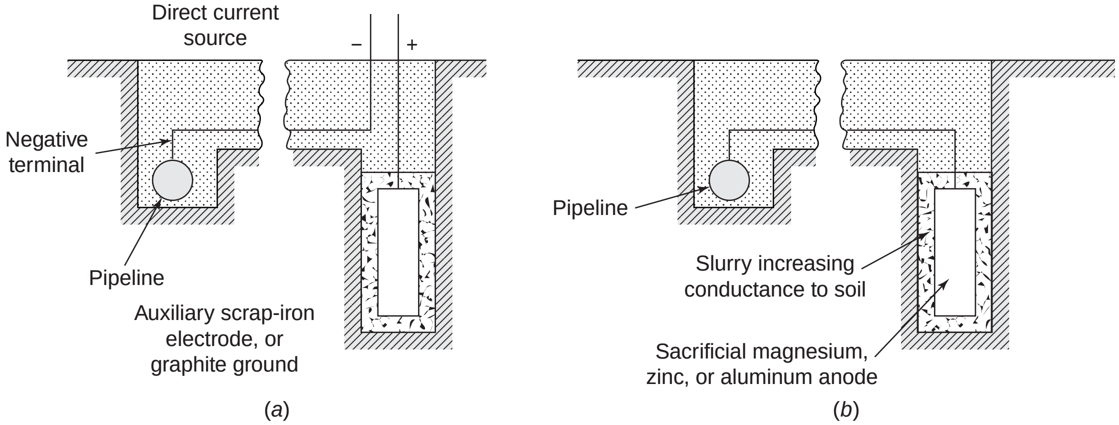

Figure 21.3 Pipeline protected (a) by auxiliary anode and impressed voltage and (b) by sacrificial anode forming galvanic cell (After Fair et al., 1971).

and denying access of water to the underlying metal, either metallic (e.g., zinc, tin, and chromium) or nonmetallic coatings (e.g., paints, plastics, and bituminous materials) being suitable

3. Deaeration for the removal of oxygen, accomplished by the direct application of a vacuum, by heating and degasification, or by passage of hot water over large surfaces of steel followed by removal of corrosion products by filtration (called deactivation)
4. Cathodic protection, which, as shown in Fig. 21.3, can be provided by (a) using direct-current electricity to feed electrons into a metal and render it cathodic or (b) introducing into the system a metal higher in the electromotive series to become the anode and be corroded or sacrificed, electrical bleeding of distribution mains and other underground utilities and cathodic protection of steel water storage tanks being examples of the first method, and the attaching of plates of zinc to the hulls or other underwater metal parts of ships, gates, and locks and the insertion of magnesium plugs into hot water heaters being examples of the second
5. Insulation by the creation of resistance to the flow of electrical currents as exemplified by (a) the insertion of insulating couplings or connectors between dissimilar metals to prevent generation of galvanic currents and (b) the use of insulating joints in water mains to oppose the flow of stray electrical currents. Under some conditions the deposition of corrosion products will nullify the effect of insulating couplings. Where it is possible to avoid doing so, metallic pipes should not be laid in natural soils or fills through which current can skip around insulation couplings and joints.
6. Deposition of protective coatings, calcium carbonate coatings being common in water supply systems (Section 21.1)
7. Application of inhibitors, namely substances that form protective films and thereby inhibit electron transfer and diffusion of reactants such as H_2O, H^+, and O_2. Deposition of dense, adherent, but slightly permeable silicate films is an example. Recommended is an initial dosage of 12–16 mg/L as SiO_2 for about a month, to be followed by the maintenance of a residual of 1 mg/L in remote parts of the distribution system

In the application of inhibitors, film structure and relative degree of surface coverage are controlling elements. Greater coverage of anodic than cathodic areas may increase current densities at anodic sites, intensify reactions, and produce pitting. Pore-free, *passive* films may be formed in the presence of oxidizing agents such as nitrate and chromate. Toxic chemicals should be introduced only into closed-circuit cooling systems. They must be kept out of drinking water systems or connecting circuits that may be subject to backflow. Some nonoxidizing inhibitors—for instance, metaphosphate, silicate, bicarbonate, organic color, and $CaCO_3$ permit dissolved oxygen to induce passivity.

Although calcium carbonate can be a good inhibitor, it performs well only if it is laid down properly in high concentrations of Ca^{2+} and HCO_3^-. Protection is poorer in the presence of Cl^- and SO_4^{2-}. It is better when the water is not stagnant. The Langelier saturation index does not predict how much $CaCO_3$ will be deposited. This is so because the pH of the solution in immediate contact with the metal is not the same as the pH at large, and because $CaCO_3$ deposition is affected by the concentration of $CaCO_3$, by electrochemical changes at the surface and rates of corrosion, and by the buffer capacity of the water and its rate of flow. Some inhibitors, among them polyphosphates and silicates, will reduce corrosion in some circumstances and promote it in others. Sequestering and reduction of detectable iron—by polyphosphates, for instance—only masks corrosion. In recirculated hot water systems, moreover, polyphosphates hydrolyze to nonprotective orthophosphates.

A summary of available methods of corrosion control leads to the conclusion that chemicals should be employed only on the basis of careful experimentation. Laboratory tests must be followed up in the field. Acknowledged, too, must be the difficulty of determining how much corrosion has taken place. Flow tests and water quality determinations are the most instructive overall measures of the effectiveness of corrosion control in pipelines and distribution systems.

21.5 LEAD AND COPPER CORROSION

Lead and copper are found in drinking water chiefly because the metals used in water distribution systems, home plumbing, and appliances, such as water coolers, corrode. These metals generally are not found in significant amounts in source waters, but rather at the final point of use. Zinc and iron also are commonly present in water as corrosion products. Cadmium has been found in water systems with new galvanized pipe. Lead and cadmium are toxic at low concentrations; copper and zinc are toxic only at much higher levels. Because of its ubiquity and toxicity, lead is the corrosion product of most concern. Cadmium also has significant health implications, but it is not as widespread in drinking water as lead.

Lead and copper are some of the elements regulated by the United States Environmental Protection Agency (US EPA). In 1991, the US EPA implemented the *Lead and Copper Rule* under authority granted by the *SDWA* and established *action levels* for lead and copper in drinking water of 15 μg/L and 1.3 mg/L, respectively. The Lead and Copper Rule also established the maximum contaminant level goals (MCLGs) of 0 mg/L for lead and 1.3 mg/L for copper.

The Lead and Copper Rule is unique in two respects. First, it requires water systems to collect samples from the customer tap, rather than from the distribution system, to determine compliance. The rule's basic requirements are to optimize corrosion control and, if appropriate, treat source water (using the best available technologies (BATs): coagulation/filtration, ion exchange, lime softening, and reverse osmosis), deliver public education, and replace lead service lines. Second, it does not establish a maximum contaminant level (MCL). Instead, the regulation for lead requires that no more than 10% of customer samples taken at the tap exceed the US EPA action level of 15 μg/L for lead. To put that amount into perspective, 15 μg/L is equal to 15 cents in 10 million dollars. Prior to 1991, US EPA had established an MCL for lead (as opposed to an action level) of 50 μg/L.

21.5.1 Health Effects

Lead can cause a variety of adverse health effects in humans. At relatively low levels of exposure, these effects may include interference with red blood cell chemistry, delays in normal physical and mental development in babies and young children, slight deficits in the attention span, hearing, and learning abilities of children, and slight increases in the blood pressure of some adults. Chronic exposure to lead has been linked to cerebrovascular and kidney disease in humans. Lead also has the potential to cause cancer from a lifetime exposure at levels above the action level. Lead can build up in the body over years and cause damage to the brain, red blood cells, and kidneys.

Copper is an essential nutrient, required by the body in very small amounts. However, US EPA has found copper to potentially cause the following health effects when people are exposed to it at levels above the action level. Short periods of exposure can cause gastrointestinal disturbance, including nausea and vomiting. Use of water that exceeds the action level over many years could cause liver or kidney damage.

21.5.2 Occurrence as a Corrosion By-product

Lead in drinking water results primarily from corrosion of materials located throughout the distribution system containing lead and copper and from lead and copper plumbing materials used to plumb public and privately owned structures connected to the distribution system. The amount of lead in drinking water attributable to corrosion by-products depends on a number of factors, including the amount and age of lead- and copper-bearing materials susceptible to corrosion, how long the water is in contact with the lead containing surfaces, and how corrosive the water in the system is toward these materials.

The potential sources of lead corrosion by-products found in drinking water can include water service mains (rarely), lead goosenecks or pigtails, lead service lines and interior household pipes, lead solders and fluxes used to connect copper pipes, and alloys containing lead, including some faucets made of brass or bronze.

Most public water systems serve at least some buildings with lead solder and/or lead service lines. US EPA estimates that there are about 10 million lead service lines/connections. About 20% of all public water systems have some lead service lines/connections within their distribution system.

The amount of lead in drinking water depends heavily on the corrosivity of the water. All water is corrosive to metal plumbing materials to some degree, even water termed noncorrosive or water treated to make it less corrosive. The corrosivity of water to lead is influenced by water quality parameters such as pH, total alkalinity, dissolved inorganic carbonate, calcium, and hardness. Galvanic corrosion of lead into water also occurs with lead-soldered copper pipes, due to differences in the electrochemical potential of the two metals. Grounding of household electrical systems to plumbing may also exacerbate galvanic corrosion.

Table 21.6 Percentage of test sites with lead in drinking water greater than 20 μg/L at low pH (6.4 and less)

Age of test site (Years)	First draw	10 s	20 s	30 s	45 s	60 s	90 s	120 s
0–1	100%	100%	100%	100%	100%	86%	86%	88%
1–2	100	71	86	57	57	29	43	14
2–3	86	86	57	57	43	43	43	29
3–4	100	86	100	71	71	71	29	29
4–5	86	57	29	43	43	43	14	0
6–7	78	44	33	33	11	11	11	0
9–10	71	29	14	14	14	14	0	0
15–16	57	14	14	14	14	14	14	14
20 and Older	86	27	29	0	14	0	14	0

Source: US EPA.

21.5.3 Diagnosing and Evaluating the Problem

Most lead corrosion problems are diagnosed indirectly. Risk factors that indicate potentially high lead levels at the tap are as follows:

1. The water distribution system or structure's plumbing is made of lead.
2. The structure's plumbing has solder containing lead.
3. The structure is more than 10 years old.
4. The tap water is soft and acidic.
5. The water stays in the plumbing for 6 or more hours.
6. The structure's electrical system is grounded to the plumbing system.
7. The presence of any of these factors justifies further investigation.

One direct method of measuring lead corrosivity of water is to sample standing water from the consumer's tap. In addition, consumer complaints, corrosion indices, sampling and chemical analysis, pipe scale examination, and measurement of the corrosion rate over time are all proxies for corrosion contaminants.

21.5.4 Sampling and Chemical Analysis

Corrosion can also be assessed by conducting a chemical sampling program. Water with a low pH (less than 6) is more corrosive. Temperature and total dissolved solids can be important indicators of corrosivity, although this varies case by case.

Proper sampling and analysis methods are essential to obtain accurate and meaningful test results. Determining whether lead and/or copper is from a service pipe or internal plumbing requires sampling at multiple locations or getting accurate and precise samples representing water in prolonged contact with the suspected section of pipe (e.g., service line, soldered joints). Experiments show that sample volume also affects the amount of lead detected in tap water. Other critical sampling elements include sampling location, amount of water in each sample, flow rate of sample, and the contact time of the water with the metal. The amount of time the water remains in the pipes significantly affects lead levels. Also, the time of day is a critical factor in accurately assessing sampling results. Early morning sample results will reflect water that has been held in the pipes overnight, while evening water samples may assess water intermittently drawn by daily activities.

The age of the pipe solder has been shown to affect lead levels in water. One study found that 4–5 weeks after an application of lead-based solder, lead levels declined by 93% from initial measurements. The effects of solder age and the length of time the water stands in the pipes are illustrated in Tables 21.6 and 21.7 for low and high water pH levels, respectively.

21.6 LEAD CORROSION CONTROL

If any of the tests described earlier reveal unacceptably high levels of lead, then immediate measures should be taken to minimize human exposure until a long-term action plan is developed, approved, and implemented. Some short-term measures for water consumers are

1. Running the water for 1–3 minutes before each use
2. Using only cold tap water for drinking or cooking
3. Using a home-scale reverse osmosis unit or other treatment process, in extreme cases
4. Using bottled water

Longer term solutions fall into six categories:

1. Distribution and plumbing system design considerations
2. Water quality modification
3. Corrosion inhibitors
4. Coatings and linings

Table 21.7 Percentage of test sites with lead in drinking water greater than 20 μg/L at low pH (6.4 and less)

Age of test site (Years)	First draw	10 s	20 s	30 s	45 s	60 s	90 s	120 s
0–1	100%	100%	60%	10%	20%	10%	20%	0%
1–2	67	22	11	11	11	0	11	0
2–3	30	10	10	0	0	0	0	0
3–4	25	0	0	0	0	0	0	13
4–5	30	10	0	0	0	0	0	0
6–7	20	0	0	0	0	0	0	0
9–10	10	0	10	0	0	10	0	10
15–16	33	22	11	11	0	0	0	0
20 and older	20	0	0	0	0	0	0	0

Source: US EPA.

5. Use of only nonlead-based solder for construction and repairs
6. Replacement of lead pipes

21.6.1 Distribution and Plumbing System Design Considerations

Many distribution and plumbing system design considerations will reduce corrosion. For example, water distribution systems designed to operate with lower flow rates will have reduced turbulence and decreased erosion of protective layers. Other measures that minimize corrosion include

1. Using only lead-free pipes, fittings, and components
2. Selecting appropriate system shape and geometry
3. Avoiding sharp turns and elbows
4. Avoiding dead-ends and stagnant areas
5. Eliminating shielded areas
6. Providing adequate drainage
7. Selecting appropriate metal thickness
8. Using welded ends, instead of rivets
9. Reducing mechanical stresses
10. Avoiding uneven heat distribution
11. Providing adequate insulation
12. Providing easy access for inspection, maintenance, and replacement
13. Eliminating the grounding of electrical circuits to the system

To implement these measures effectively, local plumbing codes may need to be modified.

Distribution system designers should base their materials selection criteria on system water characteristics. For example, water with low pH levels and high dissolved oxygen levels corrodes metals quickly. Concrete may be more appropriate than metal in this case because, although concrete dissolves under low pH conditions, the rise in dissolved calcium in the water is not objectionable and since concrete pipes are thick they have a longer lifetime. Table 21.8 presents the corrosion properties of common distribution system materials.

Metal electrochemical activity is the measure of a metal's tendency to oxidize. When placed together, metals with different activities create galvanic corrosion cells. Therefore, if different metals must be placed together, minimizing the differences in their activity will lessen corrosion. Galvanic corrosion is also avoided by placing dielectric insulating couplings between the dissimilar metals.

While many corrosion control programs address the lead content of pipes, solder, and other plumbing components, they do not target lead-bearing plumbing fixtures, such as brass faucets or valves. Changing the composition of solder used in plumbing is considered the most important factor in any lead corrosion control strategy. In the past, solder used in plumbing has been 50% tin and 50% lead. Alternative lead-free solders are available at a higher cost.

Two alternative solders are made from 95% tin and 5% antimony or silver. The 2015 costs of standard lead, antimony, and silver-based solders were USD 8.67, USD 15.41, and USD 38.53/lb, respectively (USD 19.00, USD 34.00, and USD 85.00/kg, respectively). Plumbers estimate that plumbing in an average residence requires less than 1 lb of solder, which makes solder costs relatively insignificant. However, these alternative solders do not perform exactly the same as lead-based solder. For instance, both silver-tin and antimony-tin solders are more difficult to work with than lead solder because of higher and more narrow melting ranges.

21.6.2 Water Quality Modifications

Measures that change water quality to reduce corrosion contamination, especially lead corrosion, include pH and alkalinity adjustment, lime-soda softening, and adjustment of dissolved oxygen levels, although altering oxygen levels is not a common method of control. Any corrosion adjustment program should include a monitoring component that enables

Table 21.8 Corrosion properties of frequently used materials in water distribution systems

Distribution material	Corrosion resistance	Associated potential contaminants
Copper, iron	Good overall corrosion resistance; subject to corrosive attack from high velocities, soft water, chlorine, dissolved oxygen, and low pH	Copper and possibly zinc, tin, arsenic, cadmium, and lead from associated pipes and solder
Lead	Corrodes in water with low pH and high alkalinities	Lead (can be well above MCL[a]), arsenic, and cadmium
Mild steel	Subject to uniform corrosion; affected primarily by high dissolved oxygen levels	Iron, resulting in turbidity and red-water complaints
Cast or ductile iron (unlined)	Can be subject to surface erosion by aggressive waters	Iron, resulting in turbidity and red-water complaints
Galvanized iron	Subject to galvanic corrosion of zinc by aggressive waters; corrosion is accelerated by contact with copper materials; corrosion is accelerated at higher temperatures as in hot-water systems	Zinc and iron; cadmium and lead (impurities in galvanizing process may exceed primary MCLs)
Asbestos-cement	Good corrosion resistance; immune to electrolysis; aggressive waters can leach calcium from cement	Asbestos fibers
Plastic	Resistant to corrosion	
Brass	Fairly good; subject to dezincification depending on the water quality and alloy	Lead, copper, zinc, arsenic

Source: US EPA.

[a] MCL = maximum contaminant level.

dosage modification in response to changing water characteristics over time.

1. *pH adjustment*: Frequently corrosion is mistakenly associated merely with water acidity. The pH level plays a central role in determining the corrosion rate and is relatively inexpensive to control. Generally, water pH less than 6.5 is associated with uniform corrosion, while pH values between 6.5 and 8.0 can be associated with pitting corrosion. Systems using only pH to control corrosion should maintain their water above pH 9.0 to reduce the availability of hydrogen ions as electron acceptors. However, pH is not the only factor in the corrosion equation; carbonate and alkalinity levels may affect corrosion as well.

   By adjusting pH, alkalinity, and calcium levels, operators can promote the precipitation of a protective calcium carbonate (scale) coating onto the metal surface of plumbing. While these protective coatings build up over time, they are disturbed by (a) water turbulence due to high velocities and flow rates; (b) stagnation of water causing prolonged periods of contact; and (c) variations in the control of velocities, flow rates, and stagnation.

   Calcium carbonate scaling occurs when water is oversaturated with calcium carbonate. Below the saturation point, calcium carbonate will redissolve and at saturation, calcium carbonate is neither precipitated nor dissolved. The saturation point of any particular water source depends on the concentration of calcium ions, alkalinity, temperature, and pH, and the presence of other dissolved materials, such as phosphates, sulfates, chlorides, sulfides, and some trace metals. A coating is usually attained through simple pH control, given adequate concentrations of alkalinity and carbonate. Adding lime or alkaline substances also promotes the formation of calcium carbonate scale in most systems. The Langelier index is useful for initiating this corrosion inhibiting strategy because it estimates calcium carbonate saturation. Further pH and/or carbonate and/or calcium adjustments may be required to reach equilibrium, as established by system monitoring.
2. *Lime softening*: Lime softening, which is sometimes known as lime-soda softening when soda ash is required in addition to lime, affects lead's solubility by changing the water's pH and carbonate levels. Increasing pH levels reduces the presence of hydrogen ions and increases the presence of hydroxide ions. Hydroxide ions decrease lead solubility by promoting the formation of solid basic lead carbonates that passivate the surface of the pipe. Similarly, elevating carbonate to a certain level increases the presence of carbonate ions, which, in turn, boost the presence of basic lead carbonate. However, continuing to increase carbonate levels at a pH above about 7.5 will increase lead solubility (see Fig. 21.4). If carbonate levels are the only other factors under

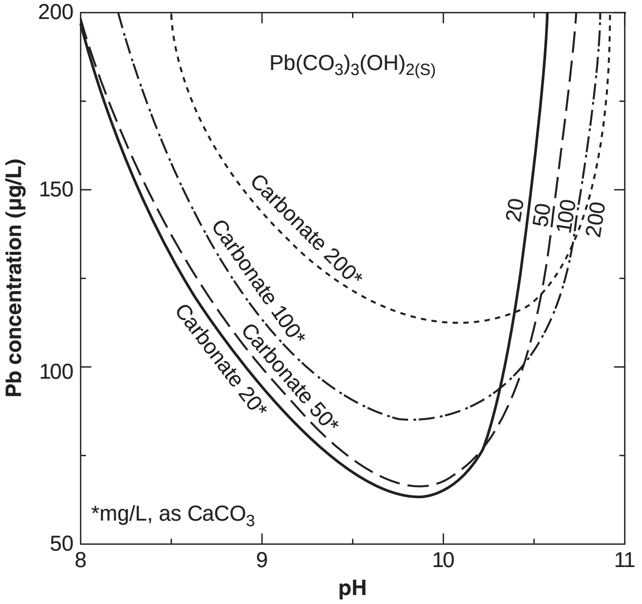

Figure 21.4 Solubility of lead as a function of pH and carbonate (*Source*: US EPA).

consideration, then the optimal pH range to avoid lead corrosion is between 9.0 and 10.0.

Usually water carbonate levels are directly related to water alkalinity because alkalinity refers to the water's ability to neutralize acids and the most common base is dissolved inorganic carbon (carbonate species). At drinking water treatment plants, neutralization is performed primarily with calcium and sodium hydroxides, although carbonates may also be added. Carbon dioxide and water form bicarbonate as the pH increases; when the pH reaches 10.3, the carbonic species is expressed as carbonate. Coordinating pH level with alkalinity supplementation may reduce lead corrosion. Optimum alkalinity levels are 30–60 mg/L as calcium carbonate ($CaCO_3$).

Adjustment of both pH and alkalinity using lime-soda softening is an effective method for controlling lead corrosion. However, optimum water quality for corrosion control may not coincide with optimum hardness reduction. Water hardness is due mostly to the presence of calcium and magnesium ions. Lime-soda softening reduces the presence of these ions by adding hydrated lime, $Ca(OH)_2$, caustic soda, NaOH, or soda ash (sodium carbonate), Na_2CO_3, under certain water quality conditions. This technique has been used to address water hardness, rather than corrosion. Comprehensive water treatment, however, must balance water hardness, carbonate, and alkalinity, as well as corrosive potential.

In addition, pH levels that are well suited for corrosion control may not be optimum for other water treatment processes, such as coagulation or disinfection. To avoid this type of conflict, the pH level should be adjusted for corrosion control immediately prior to water distribution, but after the other water treatment requirements have been satisfied.

3. *Oxygen levels*: The other major water quality factor affecting corrosion is the presence of excessive dissolved oxygen. Dissolved oxygen increases water's corrosive activity by providing a potent electron acceptor. The optimum level of dissolved oxygen for corrosion control is 0.5–2.0 mg/L. However, removing oxygen from water is not practical because of cost. The most reasonable strategy is to minimize the presence of oxygen.

   Minimizing dissolved oxygen levels is effective for groundwater supplies. Ground water is sometimes aerated prior to treatment to address high levels of iron, hydrogen sulfide, and carbon dioxide. This aeration step eliminates free carbon dioxide, which, in turn, reduces the amount of lime necessary in lime-soda softening or for pH control; however, aeration increases corrosion by increasing dissolved oxygen. Consequently, excluding the aeration step and increasing lime softening can effectively reduce unnecessary and potentially counterproductive high oxygen concentrations.

   Oxygen requirements are also reduced by extending the detention periods for treated water in reservoirs. Longer detention times allow the oxidation of hydrogen sulfide and organic carbon at the water surface. In addition, correct sizing of the water pumps used in the treatment plant minimizes the introduction of air during pumping.

4. *Other modifications*: The potential of other substances to affect lead's solubility has been investigated. Tests show that adding sulfate, chlorine, and nitrate to water has little impact on lead solubility and, thus, corrosion. However, chlorine lowers pH, and chloramines (as opposed to free chlorine) have been shown to increase the solubility of lead-based solder. A few studies indicate that some natural organic compounds increase lead solubility; however, others have shown that tannins reduce corrosion by forming protective coatings on the metal surface.

21.6.3 Corrosion Inhibitors

Corrosion inhibitors form, or cause to form, protective coatings on pipes that reduce corrosion, but may not totally arrest it. The success of any corrosion inhibitor hinges on the treatment plant operator's ability to (a) apply double and triple ultimate dosages of inhibitor during initial applications to build a base protective coat to prevent pitting; typical initial coatings take several weeks to form; (b) maintain continuous and sufficiently high inhibitor dosages to prevent redissolving; and (c) attain a steady water flow over all the system's metal surfaces for a continuous application of the inhibitor onto all exposed surfaces in the system.

There are several hundred commercially available corrosion inhibitors. Among the most common for potable water supply systems are (a) inorganic phosphates, (b) sodium silicates, and (c) mixtures of phosphates and silicates. These corrosion inhibitors can be applied with normal chemical feed systems.

1. *Inorganic phosphates*: Inorganic phosphate corrosion inhibitors include polyphosphates, orthophosphates, glassy phosphates, and bimetallic phosphates. Zinc, added in conjunction with polyphosphates or orthophosphates, also helps inhibit corrosion in some cases. Phosphates can inhibit excessive calcium carbonate scale formation, form a protective coating with the pipe metal, or prevent aesthetically objectionable corrosion by-products. Water characteristics that affect the efficiency of phosphate corrosion inhibition include (a) flow velocity, (b) phosphate concentration, (c) temperature, (d) pH, (e) calcium, and (f) carbonate levels.

   The effectiveness of potential phosphate inhibitors should be confirmed by laboratory and field tests. Glassy phosphates, such as sodium hexametaphosphate, effectively reduce iron corrosion at dosages of 20–40 mg/L. At lower dosages, this glassy phosphate may merely mask corrosion by eliminating the red color associated with iron corrosion. Under some circumstances, adding zinc in dosages of about 2 mg/L improves phosphate's corrosion control. Zinc phosphate also has been used effectively to inhibit corrosion.

   Some studies show that orthophosphate is an effective lead corrosion inhibitor within specific ranges of carbonate and hydrogen ion concentrations. Polyphosphates have not been shown to be more effective than simple orthophosphates, and adding zinc sulfate has not improved polyphosphates' performance. In addition, polyphosphates can increase lead solubility in the absence of orthophosphate.
2. *Silicates*: Sodium silicates have been used for over 50 years to inhibit corrosion, yet the process by which they form a protective layer is not completely understood. The effectiveness of sodium silicates depends on pH and carbonate concentration. Sodium silicates are particularly effective for systems with high water velocities, low hardness, low alkalinity, and pH of less than 8.4. Typical coating maintenance dosages of sodium silicates range from 2 to 12 mg/L. They offer advantages in hot water systems because of their chemical stability, as contrasted with many phosphates.

21.6.4 Cathodic Protection

Cathodic protection is an electrical method of inhibiting corrosion. However, this expensive corrosion control method is not practical or effective for protecting entire water systems. It is used primarily to protect water storage tanks when a minimum metal surface area is exposed.

Metallic corrosion occurs when contact between a metal and an electrically conductive solution produces a flow of electrons (or current) from the metal to the solution. The electrons given up by the metal cause the metal to corrode rather than remain in its pure metallic form. Cathodic protection overcomes this current with an external power source. The electrons provided by the external power source prevent the metal from losing electrons, forcing it to be a cathode. This reaction allows the metal to remain in its less stable metallic form.

Cathodic protection is accomplished either by inserting electrodes into the metal to induce an external current or by placing a sacrificial galvanic anode in the current system. The sacrificial anode then corrodes in place of the protected metal. Galvanizing is a form of cathodic protection.

21.6.5 Coatings and Linings

Mechanically applied coatings and linings differ for pipes and water storage tanks. They usually are applied prior to installation, although some pipes can be lined after installation. While coal-tar products for pipes and tanks have been widely used for linings, they are under regulatory scrutiny due to the presence of polynuclear aromatic hydrocarbons in their composition.

The most common pipe linings are coal-tar enamels, epoxy paints, cement mortar, and polyethylene. Table 21.9 summarizes the advantages and disadvantages of these four primary pipe coatings and linings.

The most common types of water storage tank coatings and linings include coal-tar paints and enamels, vinyls, and epoxy. Eleven of these coatings and linings are described in Table 21.10.

21.7 BIOFILM CONTROL

The current definition of a biofilm is an organic or inorganic surface deposit consisting of microorganisms, microbial products, and detritus. Growths of bacteria on pipe walls can provide a haven for potentially pathogenic (disease-causing) bacteria. Factors that influence the types and numbers of microorganisms found in finished drinking water include the type and quality of source water; the effectiveness of treatment and disinfection; physicochemical parameters (temperature, degree of corrosion); and the engineered system.

Knowing the types of organisms that can grow in a distribution system biofilm and their requirements for survival

Table 21.9 Pipe wall linings

Material	Use	Advantages	Disadvantages
Hot applied coal-tar enamel	Lining for steel pipes (used in 50–80% of pipes in distribution systems)	Long service life (>50 years) Good erosion resistance to silt or sand Resistant to biological attachment	Need to reapply to welded areas Extreme heat may cause cracking Extreme cold may cause brittleness May cause an increase in trace organics in water
Epoxy	Lining for steel and ductile iron pipes (can be applied in the field or in a foundry)	Smooth surface results in reduced pumping costs Formulated from components approved by the Food and Drug Administration	Relatively expensive Less resistant to abrasion than coal-tar enamel Service life <15 years
Cement mortar	Standard lining for ductile iron pipes, sometimes used in steel or cast-iron pipes	Relatively inexpensive Easy to apply (can be applied in place or in pipe manufacturing process) Calcium hydroxide release may protect uncoated metal at pipe joints	Rigidity of lining may lead to cracking or sloughing Thickness of coating reduces cross-sectional area of pipe and reduces carrying capacity
Polyethylene	Lining used in ductile iron and steel pipe (applied at foundry)	Long service life (50 years) Good erosion resistance to abrasives (silt and sand) Good resistance to bacterial corrosion Smooth surface results in reduced pumping costs	Relatively expensive

Source: US EPA.

Table 21.10 Water storage tank linings and coatings

Material	Comments
Hot applied coal-tar enamel	Most common coal-tar based coating used in water tanks; tends to sag or ripple when applied above the waterline when tank walls are heated
Coal-tar paints	Most commonly used to reline existing water tanks; those paints containing xylene and naphtha solvents give the water an unpleasant taste and odor and should be used only above the waterline Other coal tar paints containing no solvent bases can be used below the waterline but should not be exposed to sunlight or ice; service life of 5–10 years
Coal-tar epoxy paints	Less resistant to abrasion than coal tar enamel; can cause taste and odor problems in the water; service life of about 20 years
Coal-tar emulsion paint	Good adhesive characteristics, odorless, and resists sunlight degradation but not as watertight as other coal-tar paints, which limits use below waterline
Vinyl	Nonreactive; hard, smooth surface; service life (about 20 years) is reduced by soft water conditions
Epoxy	Forms hard, smooth surface; low water permeability; good adhesive characteristics if properly formulated and applied
Hot and cold wax coatings	Applied directly over rust or old paint, short service life (about 5 years)
Metallic-sprayed zinc coating	Relatively expensive process that requires special skills and equipment, good rust inhibition, and service life of up to 50 years
Zinc-rich paints	Hard surface; resistant to rust and abrasion; relatively expensive
Chlorinated rubber paints	Used when controlling fumes from application of other linings is difficult, or where their use is specified
Asphalt-based linings	Use is generally limited to relining existing asphalt-lined tanks

Source: US EPA.

can help facility operators provide safe drinking water by anticipating biofilms and taking precautions to prevent their occurrence.

Biofilms are formed in distribution system pipelines when microbial cells attach to pipe surfaces and multiply to form a film or slime layer on the pipe. Probably within seconds of entering the piping system, large particles, including microorganisms, adsorb to the clean pipe surface. Some microorganisms can adhere directly to the pipe surface via appendages that extend from the cell membrane; other bacteria form a capsular material of extracellular polysaccharides that anchors the bacteria to the pipe surface. The organisms take advantage of the macromolecules attached to the pipe surface for protection and nourishment. The water flowing past carries nutrients (carbon-containing molecules, as well as other elements) that are essential for the organisms' survival and growth.

Biofilms are dynamic microenvironments, encompassing processes such as metabolism, growth, and product formation, and finally detachment, erosion, or "*sloughing*" of the biofilm from the surface. The rate of biofilm formation depends on the physicochemical properties of the interface, the physical roughness of the surface, and physiological factors of the attached microorganisms. Sheer forces generated by fluid velocity and possible effects of disinfectants may be important in the release of biofilms from surfaces. The biofilm may grow until the surface layers begin to slough off into the water. The pieces of biofilm released into the water may continue to provide protection for the organisms until they can colonize a new section of the distribution system. Micro colonies of cells tend to form at rough surfaces, such as cracks, crevices, and pits in old and corroding pipes. Such corrosion provides an increased surface area and greater protection from the shear force of the flowing water.

Bacteria comprise the largest portion of the biofilm population. These organisms may survive the disinfection process to colonize the distribution system at the time of installation or they may be introduced through cross-connections, backflow events, line breaks, or repair operations. Fungi, which include yeasts and molds, can be found in finished water and can colonize and multiply in the pipe system. The primary concerns for fungi in drinking water are taste and odor complaints. Protozoa in potable water systems may contain a variety of nonpathogenic protozoa and other invertebrates including amoebae. Problems arise when healthcare facilities view disinfected water as sterile. Hospitals and other clinical facilities (including home healthcare agencies) need to be aware of the presence of microorganisms in finished water.

21.7.1 Factors That Favor Biofilm Growth

Growth occurs when organic materials and sediment accumulate in distribution pipes, disinfectant residuals dissipate, and water temperatures increase. Environmental factors (e.g., pH, temperature, and rainfall); nutrient availability; the presence and effectiveness of disinfectant residuals; internal corrosion and sediment accumulation; and hydraulic effects have been related to growth of coliform bacteria in drinking water.

1. *Environmental factors*: Water temperature is perhaps the most important rate controlling factor regulating microbial growth. Directly or indirectly, temperature affects all of the factors that govern microbial growth. Rainfall is another environmental factor that influences the bacterial quality of drinking water.
2. *Hydraulic effects*: Flow velocity may regulate microbial growth on pipe surfaces in several ways. Increased velocities cause greater flux of nutrients to the pipe surface, greater transport of disinfectants, and greater shearing of biofilms from the pipe surface. Reversal of water flows can shear biofilms and the "hammer" effect that occurs upon sudden opening or closing of the lines (e.g., when firefighters open and close hydrants) can dislodge tubercles from pipe surfaces. Distribution system hydraulics also can affect corrosion and sediment accumulation. Stagnation of water in the distribution system can result in loss of disinfectant residual and accumulation of sediment and debris, leading to microbial growth.
3. *Nutrient availability*: To grow, organisms must derive from the environment all the substances that they require to synthesize cell material and generate energy. For coliform and heterotrophic bacteria, the principal nutrient sources are phosphorus, nitrogen, and organic carbon. Trace nutrients also are required, but these compounds have not been investigated in drinking water.
4. *Disinfection residual concentrations*: Experience has shown that maintenance of a chlorine residual alone cannot be relied on to prevent bacterial occurrences.
5. *Corrosion*: Corrosion provides a protective surface for microorganisms, slows water flow, and contributes to backflow occurrences where iron pipe walls corrode.
6. *Sediment accumulation*: Sediments and debris in pipe systems can provide habitats for microbial growth and protection from disinfection.

21.7.2 Biofilm Control Strategies

Figure 21.5 describes steps that can be taken to control biofilms. When a biofilm problem occurs, drinking water systems should take immediate steps to limit the factors that favor bacterial growth. Sometimes a task force can be formed to deal with coliform occurrences. For example, Massachusetts established a group of US EPA, state health department, and local water treatment officials to address water quality problems. Water systems have found this approach

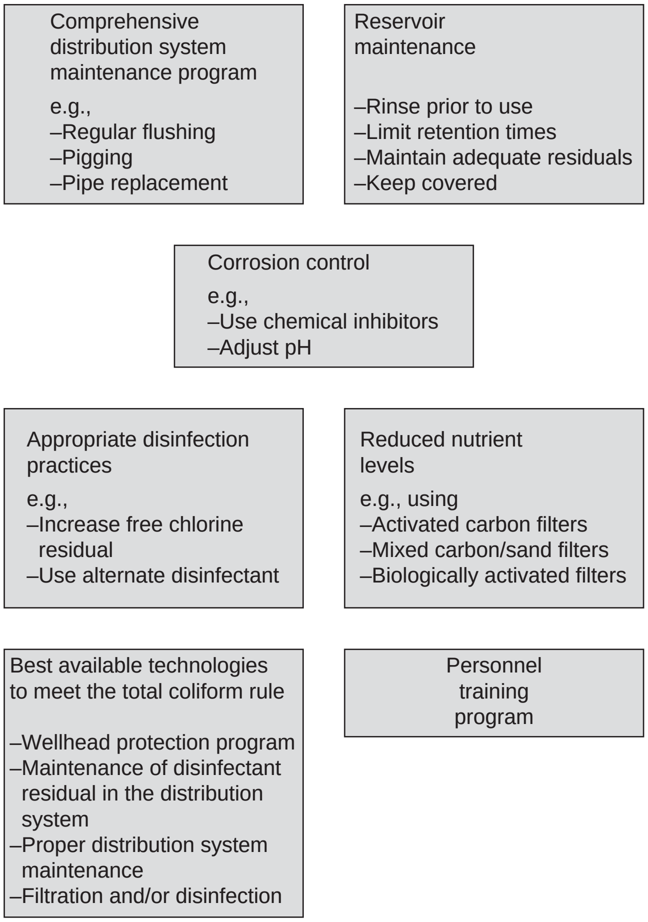

Figure 21.5 Biofilm control measures (*Source*: US EPA).

helpful because the problems of different treatment systems are not always alike, and group members' pooled knowledge can yield a solution more quickly.

The best way to avoid coliform biofilm problems is to anticipate their occurrence. Knowing the factors that contribute to biofilm growth gives the system a head start in ensuring that biofilm growth is limited in the distribution system. The system may consider instituting a coliform biofilm control plan before positive tests for coliforms appear. Biofilm control measures recommended for inclusion in the plan are described in the order of ease of implementation.

1. *Comprehensive maintenance program*: A maintenance program for the distribution system is central to controlling and preventing biofilm growth. However, routine systematic flushing, a primary component of distribution system maintenance, is frequently neglected due to a need to cut costs or lack of personnel. Regular flushing helps to distribute the disinfectant residual to all portions of the system and scour existing biofilms. More aggressive cleaning, using cable-drawn or water-propelled devices (pigging), may be necessary when corrosion tuberculation is severe. Flushing and mechanically cleaning distribution system lines can be effective preventive procedures, but may not be sufficient to resolve biological growth once the problem has become severe. In some cases, these procedures can rupture older water pipes. It may be more economical to replace or rehabilitate pipe sections than to continue to apply more temporary solutions such as flushing and pigging.
2. *Maintenance of reservoirs*: Reservoirs should be rinsed prior to use, retention times should be limited, and adequate disinfection residuals should be maintained. Reservoirs of treated water always should be covered to guard against contamination by animals, birds, insects, air pollution, accidental spills, and surface water runoff. Even covered reservoirs, however, can become contaminated when air is drawn through air vents to replace exiting water; installing air filters can help guard against pollution entering the system.
3. *Corrosion control*: Limiting corrosion in distribution system pipes inherently limits biofilm growth by reducing the numbers of places available for attachment by microorganisms. Corrosion control measures such as chemical inhibitors and pH adjustments have been shown to increase the effectiveness of free chlorine for disinfection of biofilms on iron pipes.
4. *Appropriate disinfection practices*: One of the first steps that utilities usually take to control bacterial problems is to increase disinfectant residuals. Many in the water industry have found that applying a second disinfectant such as combined chlorine residual just before the water enters the distribution system can effectively control bacterial levels in the system. Recent research has suggested that monochloramine may be more effective for biofilm control than free chlorine. The results of more recent studies suggest that biofilm control can be achieved using chloramine levels ranging from 2 to 4 mg/L.
5. *Controlling nutrient levels*: Controlling the levels of nutrients available for bacterial growth is the most direct means of resolving biofilm problems. Unfortunately, it is also the most difficult. To control bacterial nutrients, utilities must adopt new monitoring and treatment techniques. One way to reduce dissolved organics in water is through the use of activated carbon filtration. Mixed filters of granular activated carbon (GAC) and sand can be more effective for reducing organic levels than are filters made of sand alone. This is probably because GAC has a greater surface area to support biological growth and adsorb organic substrates. Biological removal of organic material using slow sand filters has been reported to be successful.

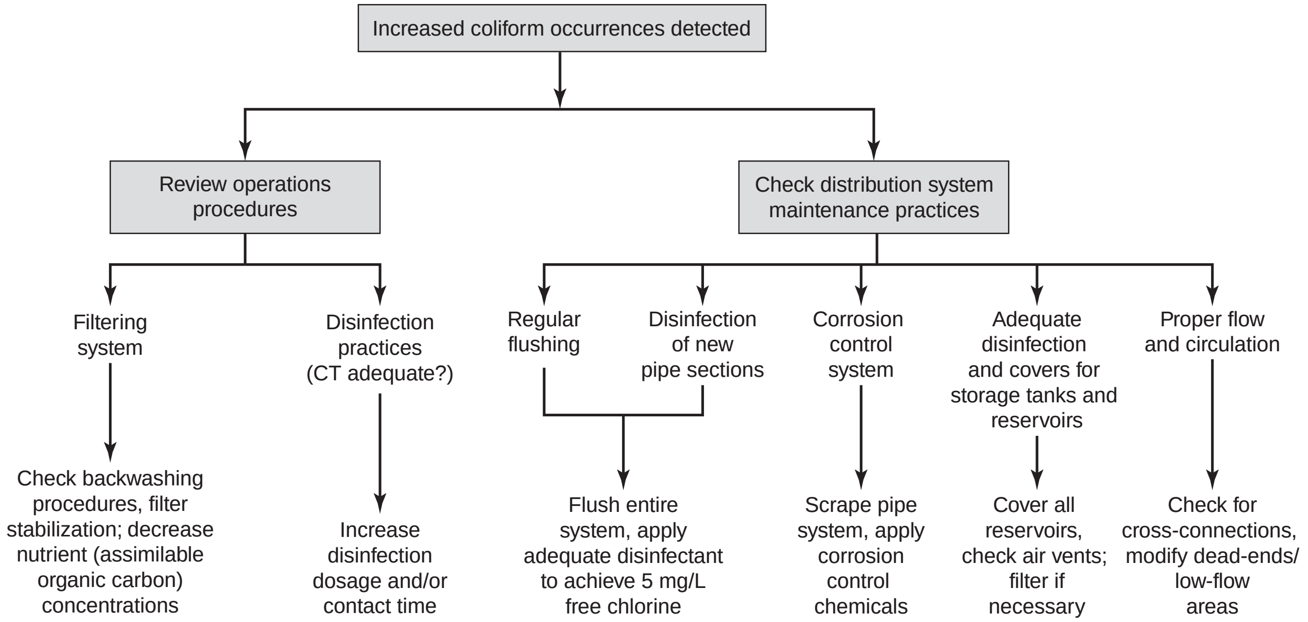

Figure 21.6 Steps for controlling biofilm growth (*Source*: US EPA).

6. *Training/upgrading personnel*: The technical ability of operators and their level of understanding of the water treatment plant are crucial to the success of the treatment and monitoring required under the SDWA.
7. *Applying best available technology*: Application of the BATs determined by the US EPA based on effectiveness for removing or treating contaminants should be taken into consideration in the formulation of the biofilm control plan.

The biofilm control plan is not only a remediation plan but a prevention program as well. Systems that maintain an adequate disinfectant residual, flush the distribution lines regularly, and practice good pipe maintenance will have a lower risk of developing a biofilm problem. Figure 21.6 summarizes the steps to be taken when increased coliform levels are detected. Finally, once a potential public health problem is identified, the system should take quick action to resolve the issue and protect the health of consumers. Providing the public with accurate information on the problem and its status will ensure that consumers understand the problem and its implications without undue alarm.

PROBLEMS/QUESTIONS

21.1 What is chemical stabilization? What is its effect on coating the inside of the pipe with calcium carbonate? Give the equations that define chemical equilibrium.

21.2 What is Langelier saturation index and what is its significance?

21.3 Determine the Langelier saturation index for water having a temperature of 20°C, a pH of 8.5, calcium hardness of 125 mg/L (as $CaCO_3$), a TDS of 275 mg/L, and a total alkalinity of 120 mg/L (as $CaCO_3$). Explain the significance of your result.

21.4 Compute the Langelier saturation index for water having a temperature of 20°C, a pH of 7.7, calcium hardness of 12.5 mg/L (as $CaCO_3$), a TDS of 20 mg/L, and a total alkalinity of 10 mg/L (as $CaCO_3$). Explain the significance of your result.

21.5 What are the implications of corrosive water?

21.6 Name and define the most common types of nonuniform corrosion.

21.7 A metal coupon 18 in. (45.72 cm) long and 3 in. (7.62 cm) wide was placed in a water pipe for measuring its corrosion rate. The coupon was inserted in the pipe for a period of 5 days. Upon retrieval of the coupon it was weighed and found to have lost 20 mg. Knowing that the coupon density is 7,500 kg/m^3, determine the rate of pipe corrosion in MPY.

21.8 A coupon 10 in. (25.4 cm) long and 4 in. (10.16 cm) wide was introduced in a water pipe for measuring its corrosion rate. The coupon was inserted in the pipe for a period of 2 days. Upon retrieval of the coupon it was weighed and found to have lost 30 mg. Knowing that the coupon density is 7,600 kg/m^3, determine the rate of pipe corrosion in MPY.

21.9 Explain the commonly used materials and methods for corrosion control.

21.10 What are the risk factors that indicate potentially high lead levels in tap water?

21.11 Name some short-term measures that water consumers should immediately take to minimize human exposure if tests reveal unacceptably high levels of lead in tap water.

21.12 Water distribution systems designed to operate with lower flow rates will have reduced turbulence and decreased erosion of protective layers. What other measures would minimize corrosion?

21.13 Explain briefly what water quality measures to implement in order to reduce corrosion contamination, especially lead corrosion?

21.14 What are the two mostly used corrosion inhibitors? Give a brief description of each.

21.15 Explain how the method of cathodic protection inhibits corrosion. Where is it best used and how is it accomplished?

21.16 Define the term biofilm; explain its importance and how it is formed.

21.17 Name and explain the factors that favor biofilm growth.

21.18 Describe the biofilm control measures that are recommended for inclusion in a typical biofilm control plan.

21.19 Conduct an internet literature study, and then determine the Ryznar Stability Index for the water described in Problem 21.4. Discuss the relationship between the Ryznar Stability Index and the Langlier Stability Index. Discuss the recommended corrosion prevention or scale prevention treatment.

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Chapter 22

Residues Management, Safety, and Emergency Response

22.1 MANAGEMENT OF RESIDUES

In addition to the production of safe drinking water, water treatment plants (WTPs) produce a wide variety of waste products or residues. Residues are defined as "floating solids, debris, sludge, deposits foam, scum, or any other material or substance" produced as a result of human activity. These residuals may be organic and inorganic compounds in liquid, solid, and gaseous forms depending on the source of raw water and the type of treatment processes. Most common processes are (a) clarification (either sedimentation or flotation), (b) coagulation/filtration, (c) water softening, (d) membrane separation, (e) ion exchange, and (f) granular activated carbon (GAC). The differences between these unit processes characterize the type of residuals generated at a given facility (Table 22.1).

1. *Clarification* (*sedimentation and flotation*): Sedimentation and flotation are used mainly to remove suspended solids.
2. *Coagulation and filtration*: This traditional form of WTP is typically used to remove turbidity and pathogenic organisms. These facilities may also be used to remove color-, taste-, and odor-causing compounds from the water supply. A variation of this process may use aeration and oxidation processes for the removal of iron and manganese. Unit processes may include screening, chemical pretreatment, presedimentation, microstraining, aeration, oxidation, coagulation/flocculation, sedimentation/flotation, filtration, and disinfection.
3. *Precipitative softening plant*: This variation of a coagulation/filtration facility uses additional processes to reduce water hardness. Unit processes may include screening, chemical pretreatment, presedimentation, microstraining, aeration, oxidation, coagulation/flocculation, lime softening, sedimentation/precipitation, filtration, and disinfection.
4. *Membrane separation*: This process is typically used to remove turbidity, total dissolved solids, hardness, nitrates, and radionuclides from a water supply. More recent applications address removal of microbiological contaminants. Membrane separation generally involves the use of microfiltration, ultrafiltration, nanofiltration, reverse osmosis, or electrodialysis, often times in combination with pretreatment practices.
5. *Ion exchange*: These facilities are used to remove inorganic constituents, including hardness, nitrates, arsenic, and radionuclides from water.
6. *Granular activated carbon adsorption*: GAC is used in many processes for the removal of naturally occurring and synthetic organic matter from water.

Residues from these processes, if disposed of in a body of water, may make the water unfit or unsafe for designated uses; may cause leaching of toxic or deleterious substances; and may cause a sludge, solid, or emulsion to be deposited on or in the water, on the bottom, or on adjoining shorelines. Thus, the residues criteria prohibit deposit of residues in the water or on the bottom, unless a zone of deposit is authorized by a US EPA permit.

The management program for a WTP should include the development of a cost-effective plan to remove and dispose of residuals. The following steps need to be considered when developing a comprehensive water treatment residuals management plan:

1. Characterize type, quantity, and quality of the residuals
2. Identify the applicable regulatory requirements
3. Determine the appropriate disposal options
4. Select best applicable treatment technologies
5. Develop a residuals management plan taking into consideration both economic and technical requirements

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Water Engineering: Hydraulics, Distribution and Treatment, First Edition. Nazih K. Shammas and Lawrence K. Wang.

Table 22.1 Treatment processes and waste streams

Treatment process	Residual waste streams generated	Contaminant categories	Disposal methods
Coagulation/ filtration	Aluminum hydroxide, ferric hydroxide, or polyaluminum chloride, sludge with raw water suspended solids, polymer and natural organic matter (sedimentation basin residuals)	Metals, suspended solids, organics, biological, radionuclides, inorganics	Landfiling Disposal to sanitary sewer/WWTP Land application Surface discharge
	Spent backwash filter-to-waste	Metals, organics, suspended solids, biological, radionuclides, inorganics	Recycle Surface discharge (pumping, disinfection, dechlorination) Disposal to sanitary sewer/WWTP
Precipitative softening	Calcium carbonate and magnesium hydroxide, sludge with raw water suspended solids, and natural organic matter	Metals, suspended solids, organics, unreacted lime, radionuclides	Landfiling Disposal to sanitary sewer/WWTP Land application
	Spent backwash filter-to-waste	Metals, organics, suspended solids, biological, radionuclides, inorganics	Recycle Surface discharge (pumping, disinfection, dechlorination) Disposal to sanitary sewer/WWTP
Membrane separation	Reject streams containing raw water suspended solids (microfiltration), raw water natural organics (nanofiltration), and brine (hyperfiltration, RO)	Metals, radionuclides, TDS, high molecular weight contaminants, nitrates	Surface discharge (pumping, etc.) Deep well injection (pumping) Discharge to sanitary sewer/WWTP Radioactive storage
Ion exchange	Brine stream	Metals, TDS, hardness, nitrates	Surface discharge Evaporation ponds Discharge to sanitary sewer/WWTP
Granular activated carbon	Spent GAC requiring disposal and/or reactivation, spent backwash	VOCs, SOCs (nonvolatile pesticides), radionuclides, heavy metals	Landfill Regeneration—on/off site Incineration Radioactive storage Return spent GAC to supplier

Source: US EPA.

22.2 TYPES OF RESIDUALS

WTP residuals form when suspended solids in the raw water react with chemicals (e.g., coagulants) added in the treatment processes and associated process control chemicals (e.g., lime). Some potable water treatment processes generate residuals that are relatively easy to process and dispose of. For example, leaves, limbs, logs, plastic bottles, and other large floating debris separated from water during the initial screening process can be disposed of at conventional solid waste landfills. However, most other treatment processes produce more complex residual waste streams that may require advanced processing and disposal methods to protect human health and the environment. The four major types of residuals produced from water treatment processes are

1. Sludges (i.e., water that contains suspended solids from the source water and the reaction products of chemicals added in the treatment process). Presedimentation, coagulation, filter backwashing operations, lime softening, iron and manganese removal, and slow sand and diatomaceous earth filtration all produce sludge.
2. Concentrate (brines) from ion exchange regeneration and salt water conversion, membrane reject water and spent backwash, and activated alumina waste regenerant.
3. Ion exchange resins, spent GAC, and spent filter media (including sand, coal, or diatomaceous earth from filtration plants).
4. Air emissions (off-gases from air stripping, odor control units, or ozone destruction).

The chemical characteristics and contaminant concentration levels in these residual waste streams often impose the ultimate disposal options. Furthermore, it is reasonable to expect that as drinking water quality is increasingly regulated, higher removal efficiencies of more contaminants will be required. To achieve these higher efficiencies, WTPs will need to use more sophisticated treatment technologies. Of particular concern are cases in which residuals are characterized as either hazardous or radioactive waste. Depending on the raw water quality and treatment process removal efficiency, hazardous or radioactive characteristics could be exhibited in potentially any residual waste stream mentioned above.

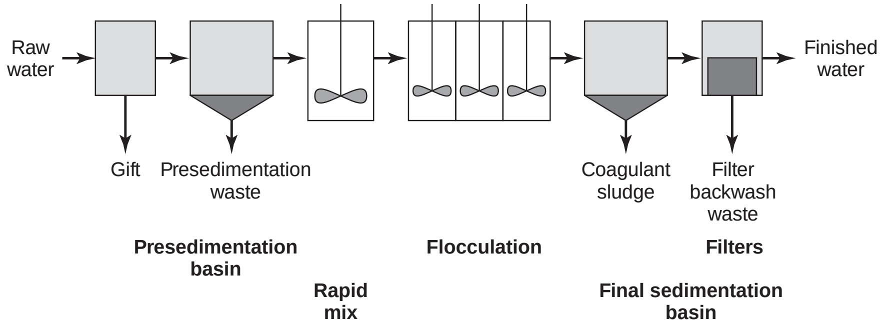


Figure 22.1 Residuals sources in water treatment plants (*Source*: US EPA).

Table 22.1 lists some typical residuals generated from drinking water treatment processes, possible contaminants normally found in each waste stream, and available residual disposal methods.

22.3 APPLICABLE REGULATIONS

Identifying the regulations that affect various management practices may be difficult for water treatment utility managers. The difficulty is due to the many different types of wastes produced by WTPs and various types of waste disposal: direct discharge, discharge to wastewater treatment plants (WWTPs), disposal in landfills, land application, underground injection, disposal of radioactive waste, and treatment of air emissions.

At the federal level, the US Environmental Protection Agency (US EPA) has not established any regulations that are specifically directed at WTP residuals. Most states are responsible for establishing and administering regulations that will meet the requirements of the federal acts. The regulation of wastes, therefore, is the responsibility of the states.

22.4 RESIDUAL SOLIDS TREATMENT

In many instances, regulatory requirements or the need for cost-effectiveness dictate that a residual receive further treatment to make it acceptable for disposal. The three classic treatment processes for residual solids are thickening, dewatering, and drying. Application of a particular process depends on the solids concentration of the residuals. Some water treatment processes that may produce solids are grit collection, sedimentation, and filtration (Fig. 22.1)

Table 22.2 indicates which common residual treatment processes are usually applied to residuals with low, medium, or high solids concentrations. The processes presented in Table 22.2 are often configured in series to create combination systems that can provide a low level of operational complexity with a high degree of operational flexibility.

Residuals handling process types are

1. *Thickening*: A process of concentrating the solids content of a residual stream to reduce the volume before disposal or further treatment.
2. *Coagulant recovery*: A treatment technique for improving solids dewatering characteristics and lowering the concentration of metallic ions in the residuals. Recalcination is a related process associated with lime softening sludges.
3. *Conditioning*: Adding a chemical to a residual or physically altering its nature. Conditioning is traditionally used as a method to optimize the dewatering process.
4. *Dewatering*: Similar to thickening in that both processes involve a liquid–solids separation approach with a goal of minimizing the amount of residuals for disposal. Dewatering is defined as a process to increase the solids concentration of residuals (by weight) to greater than 35%.
5. *Drying*: An extension of the liquid–solids separation approach of thickening and dewatering. Drying is defined as a process to increase the solids concentration of residuals (by weight) to greater than 35%.
6. *Disposal and reuse*: Removal of residuals from the WTP site or permanent storage of residuals at the WTP site. This category includes hauling to landfill, discharging to sanitary sewer or natural waterway, land application, and various reuse options (soil supplement, brick manufacturing).
7. *Recovered and non-recovered water handling*: Thickening, dewatering, and drying processes produce both liquid and solids components. The solids component

Table 22.2 Representative solids concentration treatment processes

Process	Solids concentration	Equalization	Gravity settling	Dissolved air flotation	Lagoon	Mechanical	Open air	Thermal drying
Thickening	Low	X	X	X	X	X		
Dewatering	Medium				X	X	X	
Drying	High				X	X	X	X

Source: US EPA.

may be further treated and disposed of. The liquid component is returned to the main WTP processes if it is recoverable, which means it has little impact on the main treatment process and no harmful effect on the finished water quality. Quality parameters that can affect the recoverable status of the water include the following: (a) residual metal concentrations, (b) disinfectant byproduct formation potential, and (c) use of unapproved polymers in the residuals handling processes. Non-recoverable water must either be disposed of or subjected to further treatment.

8. *Other processes*: Those that do not readily fit into a category listed above, including equalization, chemical conditioning, and residual conveyance. Equalization basins even out the flow of water streams to prevent surges of water from being introduced at the head of the treatment plant or at inlets to other residual handling processes.

22.5 RESIDUALS DISPOSAL

22.5.1 Underground Injection

Underground injection may be a disposal option for concentrates, brine, and hazardous waste from drinking water treatment processes. US EPA and state programs prevent underground injection unless authorized by a permit or rule, and only where the process will not endanger drinking water sources. Different criteria and standards apply to different classes of wells. The major criteria and standards are (a) construction requirements, (b) operating requirements, (c) monitoring requirements, and (d) reporting requirements.

22.5.2 Direct Discharge to Surface Waters

Water treatment residuals are primarily produced through the coagulation, filtration, and oxidation of surface waters to remove turbidity, color, bacteria, algae, organic compounds, and, frequently, iron and/or manganese. One type of residual is generated by the aluminum and iron salts that are commonly used for coagulation. A second type of residual results from the lime, sodium hydroxide, and/or soda ash added to reduce calcium and magnesium and soften the water.

Historically, direct discharge of water treatment effluent to surface waters has been the most commonly practiced disposal method. An American Water Works Association (AWWA) water utility survey indicated that approximately 50% of the total residuals generated by 429 utilities were pumped directly to surface waters. Most of these residuals were alum sludges. Recently, this disposal method has been questioned because of concerns about possible risk to public health and aquatic life.

Regulatory agencies have permitted the direct discharge of residuals to streams and impoundments because of the absence of information showing any adverse effects. Some states have taken a more conservative approach recently and have discontinued allowing discharge of residual solids to freshwater. The first step, therefore, by a utility considering direct discharge of WTP residuals into a receiving stream is to determine its state regulatory requirements.

The scientific literature shows that deposition of residual solids can adversely affect benthic community populations and diversity. These direct consequences on benthic organisms may limit a food source for certain fish species. In addition, solids deposition may affect fish egg survival. Discharge of WTP residuals in sluggish streams or lakes should be avoided. More frequent, reduced solids mass loading from the WTP during high stream flow will possibly reduce deposition and adverse effects to downstream benthic organisms.

The receiving water chemistry can minimize toxic effects of metals in the discharged waste stream. Hardness, alkalinity, pH, dissolved oxygen, sulfate, and other water quality parameters minimize the inhibitory effects of heavy metals. Because of the potential toxicity to aquatic organisms, utilities that are evaluating discharge to soft waters (hardness less than 50 mg $CaCO_3$/L) should consider an alternate residuals disposal method. Furthermore, receiving waters with a pH less than 6 should be avoided. Discharge to these waters could result in increased solubility of metals and increased toxic effects.

Residuals from WTPs using aluminum salts have been the focus of research on the impacts of direct discharge of waste streams to freshwater aquatic ecosystems. Aluminum may not be inhibiting organism growth or survival; other metals or contaminants contained in wasted residuals may be the primary toxicant. Detrimental effects of solids deposition on benthic organisms can be caused by limiting oxygen transfer to chemical toxicity. In any case, sedimentation of solids must be evaluated and anticipated. Other chemical residuals can affect benthic communities and fish spawning areas. In addition, chemical characteristics and quantities of the receiving water and the waste streams must be determined to anticipate chemical changes to the water downstream from the points of discharge. Toxicity testing must be conducted to determine potential toxic effects on the ecosystem.

22.5.3 Discharge to Sewers and WWTP

Many water utilities across the country discharge WTP residuals to a sewer and/or to a WWTP. These options are often economically attractive and transfer disposal liability to the WWTP. The AWWA survey of WTPs discharging to WWTP reveals that such plants are variable in size and typically treat surface water supplies as opposed to groundwaters.

Conventional WTPs (coagulation, sedimentation, and filtration) commonly discharge filter backwash solids and/or clarification basin residuals to a sanitary sewer system for eventual treatment at a WWTP.

Several factors must be considered when evaluating the feasibility of discharging residuals to a WWTP. The interests and concerns of WWTP managers and operators are different from those of similar personnel in water utilities. Factors for a WWTP to consider are available capacity (conveyance system and treatment plant), treatment process compatibility, and final disposal requirements. Introducing water treatment residuals to a WWTP may offer some benefits in terms of process performance. On the water treatment facility side, pretreatment requirements, storage facilities, and conveyance systems must be considered. Costs and service agreement terms must also be evaluated.

Regulatory requirements, especially those related to ultimate disposal, must also be addressed. Essentially, all states allow for discharge of WTP residuals (sludge and brines, or reject waters) to a WWTP, but the specific WWTP requirements and limitations vary. Regulatory requirements imposed on WWTPs will be directly or indirectly imposed on the WTP discharging to the WWTP. Quality issues, especially levels of metals and other inorganics are likely to be a concern.

For WTP managers and operators, disposal of WTP residuals into a sewer system or to a wastewater treatment facility is usually very attractive, cost-effective, and offers significant benefits from a regulatory and operational standpoint. This disposal method, however, has not been universally accepted by the wastewater community. Provided that the water treatment facility complies with the receiving wastewater utility's pretreatment and discharge requirements, the liability for proper disposal of the water treatment residuals is transferred from the water utility to the wastewater utility. Operationally, a WTP has no further responsibility for its waste residuals once they have been transferred to a wastewater utility; routine maintenance of the sewer system and/or operation of the wastewater treatment facility is the responsibility of the wastewater utility.

Costs associated with this option include the capital expenditures necessary to intercept, tie in, or transport residuals; user fees imposed by the receiving wastewater utility to recover conveyance; treatment and ultimate disposal operation and maintenance (O&M) costs; and other costs. These costs are often less than other options for directly handling WTP residuals. It must be noted that in some cases, the receiving WWTP benefits from increased removal of suspended solids and/or biochemical/chemical oxygen demand (BOD/COD).

22.5.4 Landfilling

As a drinking water utility evaluates its treatment plant residuals management options, landfilling will most likely be considered. In nearly every case, landfilling falls into two categories: co-disposal and monofilling. In most instances, WTP residuals will be classified as nonhazardous solid waste.

Under federal regulations, criteria have been established for the design and operation of nonhazardous solid waste landfills. The requirements apply to landfills that receive only WTP residuals (monofills), as well as landfills that accept solid waste other than household waste (e.g., industrial waste). These criteria, which are performance based and do not include any specific design criteria, address seven landfill design and operation areas: flood plains, endangered species, surface water, groundwater, diseases, air, and safety.

Municipal solid waste landfills (MSWLFs) are subject to similar performance-based criteria, but they also include specific design criteria. If a utility disposes of its drinking water residuals in a monofill, then these specific criteria do not apply. If, however, the WTP residuals are co-disposed of with municipal solid waste, including household waste, the requirements established for MSWLFs apply. The specific landfill criteria address six major areas: location restrictions, operating criteria, design criteria, groundwater monitoring and corrective action, closure and post-closure care, and financial assurance criteria.

The federal regulations establish minimum criteria to be applied in all states. In addition, each state may develop its own landfill requirements provided that the criteria at least meet the minimum federal standards. In many states, landfill requirements are more restrictive than federal criteria. Many states have developed unique solid waste landfill programs that reflect specific statewide factors and concerns. Any evaluation of landfilling options for the management of WTP residuals should include a careful examination of state landfill requirements. In many states, the flexibility exists to design and operate landfills according to site-specific conditions and limitations.

22.5.5 Land Application

Land application is an increasingly popular disposal option in the water supply industry due to the escalating regulatory and environmental constraints associated with other disposal options. Land application options for WTP residuals include agricultural use, silvicultural application, application for reclamation to disturbed and marginal lands, and disposal to dedicated lands. In addition to the obvious advantages of waste disposal, this application process may beneficially modify soil properties while recycling residual components. Potential disadvantages of land application of WTP residuals include an increase in the concentration of metals in the soil and possibly in ground water; adsorption of soil phosphorus by water residuals, decreasing the productivity of the soil; excessive application of nitrogen, resulting in the transport of nitrate to ground water; and possible effects caused by the application of poorly crystallized solids of aluminum.

Figure 22.2 provides a simplified planning procedure for evaluation of land application as a disposal option for WTP residuals.

If land application is to be considered a viable option, the residuals cannot be hazardous. Specific federal regulations do not exist for the water industry, and most state regulatory agencies use the same criteria for WTP wastes as they use for other industrial sludges. Although the federal criteria for land application of biosolids generated during the treatment of domestic wastewater are only designed for sewage sludge, they may provide general guidelines for use or disposal of WTP residuals. The guidelines contain numerical values for biosolids application rates that are based on experiments with biosolids and risk assessment algorithms developed using the

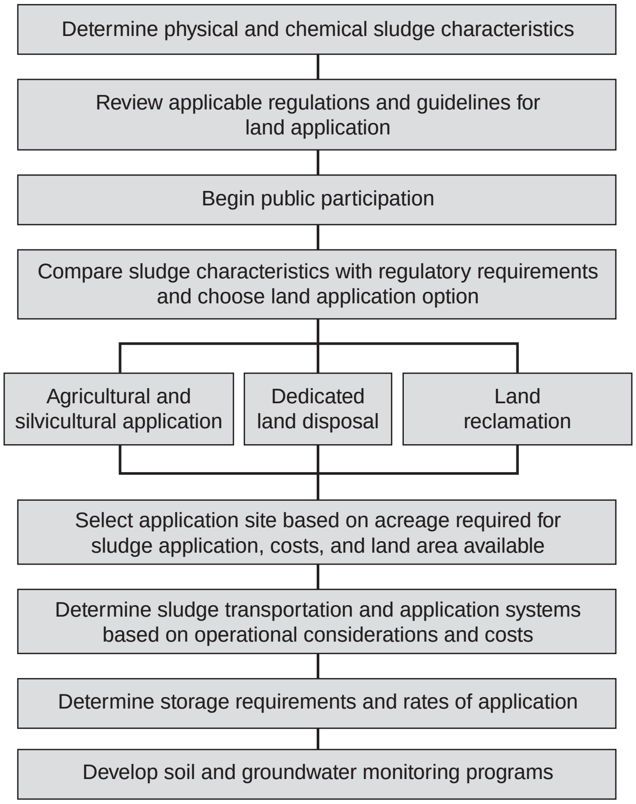


Figure 22.2 Planning procedure for land application (*Source*: US EPA).

biosolids data. These criteria should not be used for determining land application rates of WTP residuals.

Regulations vary from state to state but typically require that residuals be tested for total metals concentrations, nutrients, and pH. The application rate of WTP sludge may be selected to limit hydraulic loading, nitrogen content, or metal content.

22.6 SELECTION OF MANAGEMENT PLANS

To develop a residuals management plan, the WTP manager may start with a broad array of residuals processing and disposal options that can be narrowed through considering specific residuals characteristics and associated regulatory requirements. A focus on available disposal options further narrows the management alternatives.

Table 22.3 illustrates the need for practical disposal options and treatment processes that will take into account economic and noneconomic factors of concerns to the community. The technical criteria used to select the final management plan differ from user to user; economic, cultural, social, and environmental factors are also site specific and are typically included in any final selection.

Developing a successful residuals management plan requires an understanding of the value of residuals characterization and the regulatory requirements, tailoring the treatment options to the requirements of the available disposal alternatives, and then developing rational evaluation criteria. Figure 22.3 illustrates a generic decisions process for developing a residuals management plan.

Table 22.3 Primary elements of a residuals management plan

Primary elements of a residuals management plan
Type of water treatment plant
Residual characteristics
Regulatory requirements
Treatment options
Disposal options
Social/cultural
Environmental
Economic

Source: US EPA.

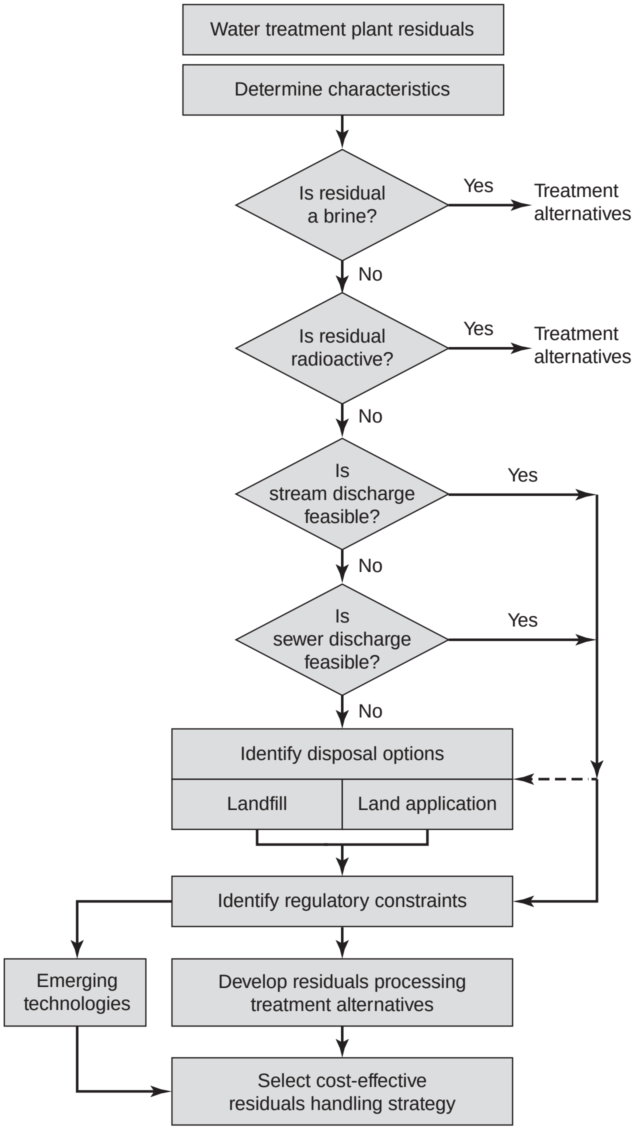


Figure 22.3 Decision process for developing residuals management plan (*Source*: US EPA).

EXAMPLE 22.1 SIMPLE WASTE MANAGEMENT SYSTEMS FOR HANDLING/TREATING/RECYCLING SPENT FILTER BACKWASH WASTEWATER, WASTE SLUDGE, AND FILTER-TO-WASTE AT CONVENTIONAL WATER TREATMENT PLANTS

A conventional WTP shown in Figs. 22.1 and 22.4 involves the use of screening, pumping, chemical feeding, rapid mixing, coagulation-flocculation, sedimentation (or flotation), filtration, corrosion control, and disinfection. The WTP is commonly used for treating surface water or GWUDI (groundwater under direct influence of surface water). After the impurities/contaminants are removed from raw water, the treated/finished water becomes suitable for consumption. The waste sludge from the sedimentation clarifier (or flotation clarifier) and the filter backwash wastewater from the filters are the regulated wastes requiring proper handling and disposal. The filter-to-waste water is unregulated. What is the simplest waste handling system for a conventional WTP assuming that (a) waste sludge in slurry form or cake form may be hauled away for landfill or chemical recycle and (b) an equalization basin or lagoon is to be used for waste holding, solids settling, water recycling, and sludge drying.

Solution:

There are three kinds of wastes produced from a filtration WTP: (a) the unregulated filter-to-waste water that is produced from filtration units during the final stage of filter backwash, before the filter is placed online for service; (b) the regulated waste sludge streams from the clarifiers (sedimentation or flotation); and (c) the regulated spent filter backwash wastewater. Figure 22.4 shows the flow diagram of the simplest waste management processes. Equalization is typically used for spent filter backwash and waste sludge, but may be used for other unregulated recycle streams such as filter-to-waste. Pumps may be required to convey the recycle streams from the equalization tank. Two case histories are listed below as typical examples of creative waste management:

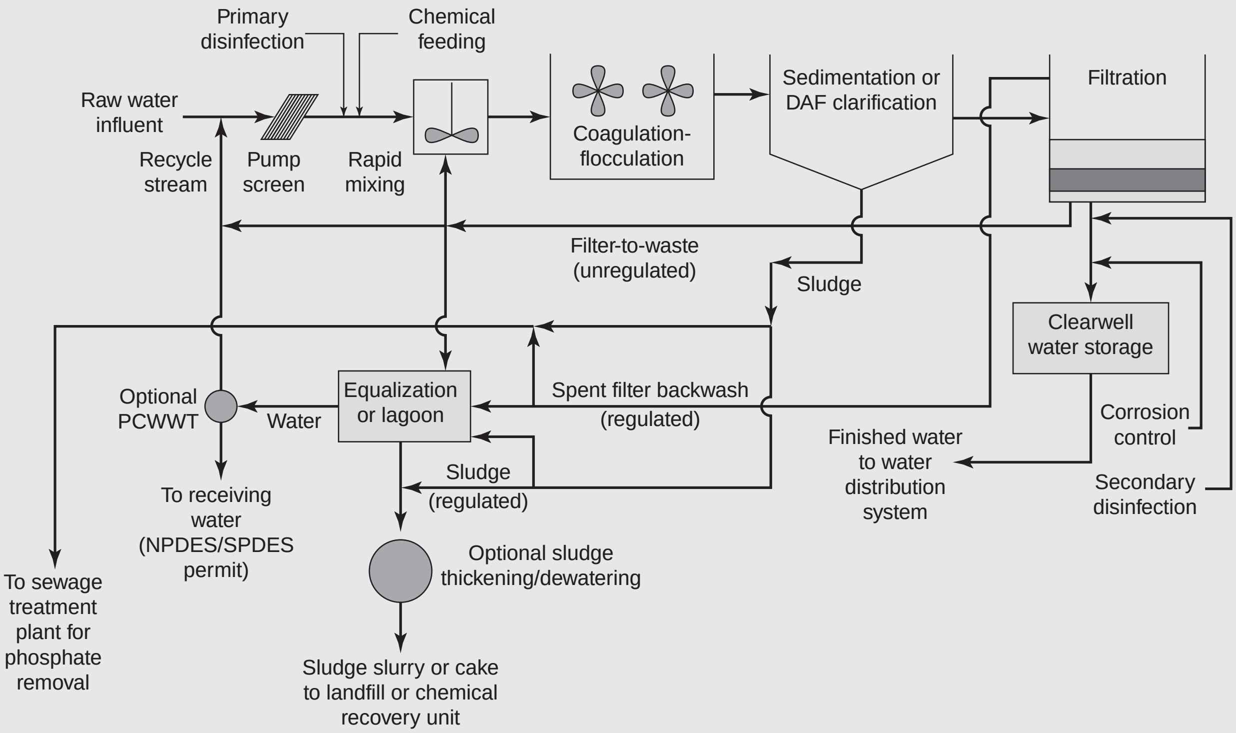


Figure 22.4 Simple waste management alternatives for a typical conventional water treatment plant (*Source*: US EPA).

1. Pittsfield WTP, Massachusetts, is a typical flotation-filtration plant consisting of presedimentation (reservoir), screening, pump, chemical feeding, primary disinfection, rapid mixing, flocculation, dissolved air flotation clarification, automatic backwash filtration (sand), secondary disinfection (chlorination), corrosion control, clearwell, and water storage and distribution. Pittsfield WTP discharges its spent filter backwash wastewater and waste sludge to the Pittsfield WWTP for phosphate removal. Practically speaking, the WTP waste is recycled as phosphate removal chemical at a WWTP. With permission from the government, the water portion of the wastes (including the filter-to-waste) is recycled to the plant for production of drinking water. This option is feasible only when there is a municipal WWTP which needs phosphate removal and has enough hydraulic capacity to handle the WTP wastes (Wang, 1991).

2. Albany WTP, New York, is a typical conventional sedimentation-filtration plant consisting of presedimentation (reservoir), screening, pumping, spray aeration (VOC, taste and odor control), chemical feeding (coagulants, oxidants, PAC), primary disinfection (chlorination), rapid mixing, flocculation, sedimentation, filtration (sand), corrosion control, secondary disinfection (chlorination), clearwell, water storage, UV, and distribution. Albany WTP discharges its filter-to-waste, waste sludge, and spent filter backwash wastewater to four huge lagoons for flow equalization, solids settling, and sludge drying. The supernatant of the lagoon is discharged to a receiving stream with NPDES/SPDES permit. Every 10 years or so, a lagoon will be full of dry sludge. The dried sludge is then dredged and hauled away for landfill. No mechanical sludge thickening and dewatering units are needed. The huge lagoons serve as the waste treatment units. This option is feasible only when there is ample land space available for construction of large lagoons.

EXAMPLE 22.2 VARIOUS WASTE MANAGEMENT OPTIONS AND AN EXAMPLE FOR A COMPLETE WASTE TREATMENT AND RECYCLE SYSTEM FOR A CONVENTIONAL WATER TREATMENT PLANT

In Example 22.1, two very simple WTP waste management processes are illustrated. Not so many municipalities are fortunate to have compatible WWTP nearby to reuse WTP waste or to have large lagoons to perform all-in-one functions—waste collection, flow equalization, solids settling, water discharge, and sludge thickening/drying. Common WTP waste management methods can be various combinations of the following sub-methods:

1. The unregulated filter-to-waste:
   (a) Recycle the unregulated filter-to-waste to rapid mixing basin.
   (b) Recycle the unregulated filter-to-waste to the plant intake.
   (c) Discharge the unregulated filter-to-waste to presedimentation reservoir or receiving water.
   (d) Discharge the unregulated filter-to-waste to an equalization basin or lagoon.
2. The regulated clarifier sludge:
   (a) Discharge the regulated clarifier sludge (sedimentation clarifier, DAF clarifier) to a sludge thickener (DAF thickener, gravity thickener) from where the thickened sludge is hauled away for sludge drying bed disposal or long-term lagoon storage.
   (b) Discharge the regulated clarifier sludge (sedimentation clarifier, DAF clarifier) to a sludge thickener (DAF thickener, gravity thickener) from where the thickened sludge is hauled away for WWTP phosphate precipitation.
   (c) Discharge the regulated clarifier sludge (sedimentation clarifier, DAF clarifier) to a sludge thickener (DAF thickener, gravity thickener) from where the thickened sludge is discharged to a sludge dewatering unit (filter press, centrifuge) from where the dewatered sludge is hauled away for landfill disposal.
   (d) Discharge the regulated clarifier sludge (sedimentation clarifier, DAF clarifier) to a sludge thickener (DAF thickener, gravity thickener) from where the thickened sludge is discharged to a sludge dewatering unit (filter press, centrifuge) from where the dewatered sludge is hauled away to WWTP for phosphate precipitation.
3. The regulated spent filter backwash:
   (a) Discharge the regulated spent filter backwash to an equalization basin or lagoon, from where the supernatant is recycled to the plant intake.
   (b) Discharge the regulated spent filter backwash to an equalization basin or lagoon, from where the supernatant is discharged to a receiving water with NPDES/SPDES permit.
   (c) Discharge the regulated spent filter backwash to an equalization basin or lagoon, from where the supernatant is discharged or pumped to a physicochemical wastewater treatment (PCWWT) unit (coagulation-sedimentation, coagulation-DAF, microsand, membrane filtration, etc.) from where the treated water is recycled to the presedimentation reservoir or plant intake.
   (d) Discharge the regulated spent filter backwash to an equalization basin or lagoon, from where the supernatant is discharged or pumped to a PCWWT unit (coagulation-sedimentation, coagulation-DAF, microsand, membrane filtration, etc.) from where the treated water is combined with the dewatering unit effluent (water portion) for further PCWWT before it is recycled to the presedimentation reservoir or plant intake.
   (e) Discharge the regulated spent filter backwash to a WWTP for removal of phosphate.

(f) Combine the regulated spent filter backwash and the regulated thickener effluent (water portion) in an equalization basin for subsequent treatment in a PCWWT unit (coagulation-sedimentation, coagulation-DAF, microsand, membrane filtration, etc.), from where the PCWWT effluent is combined with the water effluent of dewatering unit (filter press or centrifuge) for a second PCWWT before the water is recycled to the presedimentation reservoir or plant intake.

(g) Recycle the supernatant of gravity thickener (water portion) or the subnatant of DAF thickener (water portion) to the plant intake.

(h) Recycle the supernatant of gravity thickener (water portion) or the subnatant of DAF thickener (water portion) to presedimentation reservoir.

(i) Discharge the supernatant of gravity thickener (water portion) or the subnatant of DAF thickener (water portion) to a receiving water with NPDES/SPDES permit.

(j) Combine the thickener effluent (water portion) with the spent filter backwash in an equalization basin for further treatment in a PCWWT unit (coagulation-sedimentation, coagulation-DAF, microsand, membrane filtration, etc.), from where the PCWWT effluent is recycled to the plant intake or presedimentation reservoir.

(k) Combine the thickener effluent (water portion), spent filter backwash, and the dewatering unit effluent (water portion) together in an equalization basin for further PCWWT, and the PSWWT effluent is recycled to presedimentation reservoir or plant intake.

The functions of sludge thickening and sludge dewatering are both for sludge volume reduction. When both thickening and dewatering are used, the number of sludge treatment steps doubles, the overall sludge handling and treatment costs may actually decrease. A water engineer must design various feasible waste treatment systems (such as those shown in Figs. 11.9, 11.10, 11.11, 11.15, etc.) and conduct cost analyses for comparison, in order to find an optimized waste treatment system for a specific community. The best waste treatment system for Community A may not be the best waste treatment system for Community B because many local factors must be considered. In addition, if the raw water contains toxic substances, after proper water treatment, the finished water is potable and drinkable, but the toxic substances are now concentrated in the collected waste sludge and spent filter backwash wastewater. Under this situation, the waste management method may become complex. Recommend a flow diagram of a complete conventional WTP consisting of (a) a water treatment train consisting of screening, pumping, raw water reservoir (presedimentation), chemical feeding, rapid mixing (flash mixing), coagulation/flocculation, sedimentation, filtration, corrosion control, secondary disinfection, and clearwell; and (b) a waste treatment train consisting of gravity thickening and plate frame press (filter press) dewatering for sludge treatment and equalization flocculation clarification for wastewater treatment.

Solution:

Select the following sub-methods from the above list:

1c. Discharge the unregulated filter-to-waste to presedimentation reservoir or receiving water.

2c. Discharge the regulated clarifier sludge (sedimentation clarifier, DAF clarifier) to a sludge thickener (DAF thickener, gravity thickener) from where the thickened sludge is discharged to a sludge dewatering unit (filter press, centrifuge) from where the dewatered sludge is hauled away for landfill disposal.

3f. Combine the regulated spent filter backwash and the regulated thickener effluent (water portion) in an equalization basin for subsequent PCWWT unit (coagulation-sedimentation, coagulation-DAF, microsand, membrane filtration, etc.), from where the PCWWT effluent is combined with the water effluent of dewatering unit (filter press or centrifuge) for a second PCWWT before the water is recycled to the presedimentation reservoir or plant intake.

Using the sub-methods 1c, 2c, and 3f, a complete flow diagram of a conventional WTP including both potable water treatment train and the waste management train can be constructed. Four assumptions are made for the construction of the flow diagram shown in Fig. 22.5:

1. Sedimentation (instead of dissolved air flotation) is used for water clarification.
2. Gravity thickener (instead of DAF thickener) is used for sludge thickening.
3. Plate frame press or filter press (instead of centrifuge) is used for dewatering.
4. Two-stage coagulation and flocculation (instead of other PCWWT) is used for treatment of wastewater.

Figure 22.5 illustrates the conventional WTP with the above specified waste sludge and wastewater treatment facilities. The most common water treatment units, such as screening, pumping, chemical feeding, corrosion control, primary and secondary disinfection, water storage, and water distribution, are not shown in Fig. 22.5 for the purpose of simplicity.

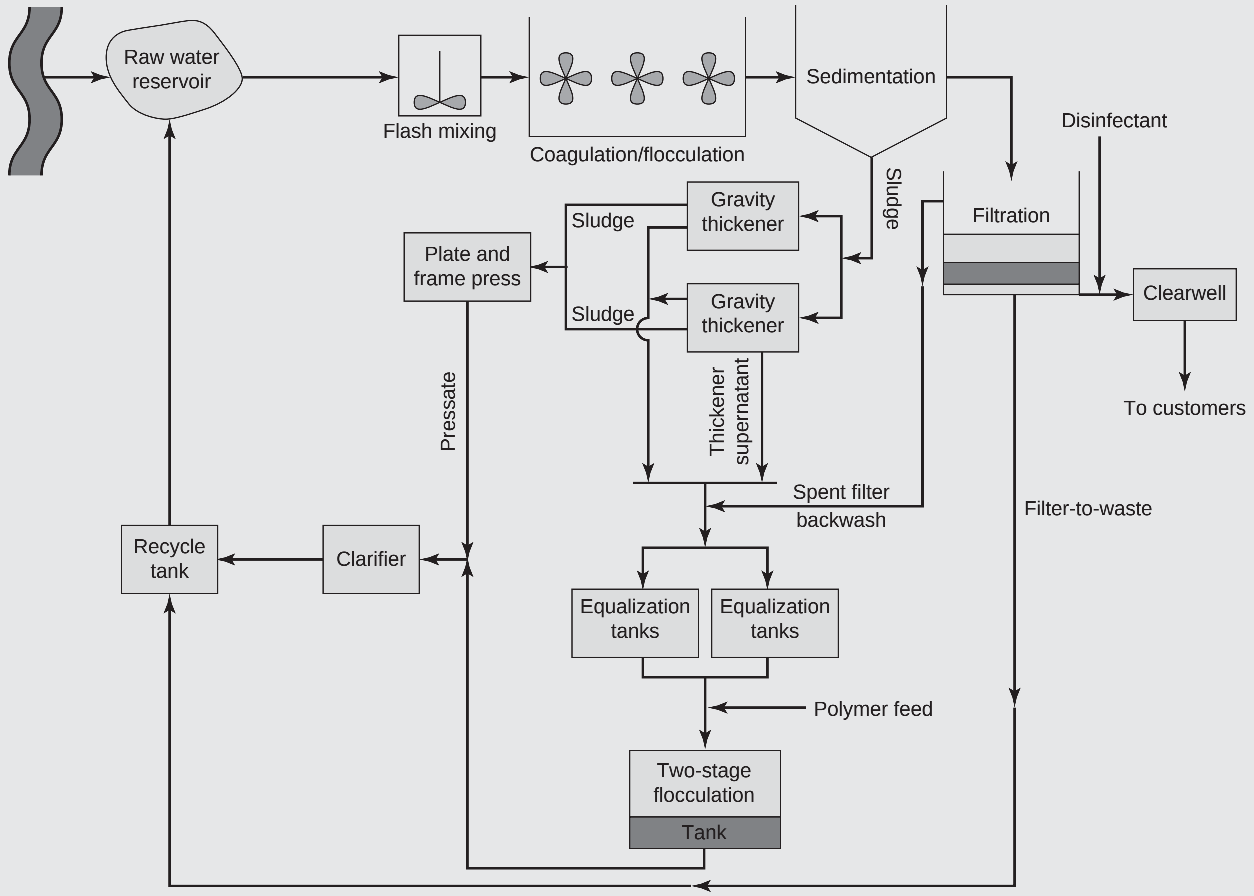


Figure 22.5 Extensive waste management plan for a typical conventional water treatment plant (*Source*: US EPA).

22.7 SAFETY AND ACCIDENT PREVENTION

The best way to maintain the safety of all personnel in a community water supply system and prevent problems and accidents is to minimize hazards and "*design them out*" early in the design process (see Chapter 23). Three case studies from water distribution systems were discussed earlier (see Chapter 6); in this section four more case studies pertinent to WTPs are elucidated. The majority of these cases have been adopted from investigations carried out by NIOSH (National Institute for Occupational Safety and Health), the federal agency responsible for occupational safety and health research.

22.7.1 Safety in Valve Vault Design

A 35-year-old male water system operator (victim) was asphyxiated after entering a valve vault at a municipal WTP. The victim was assigned to turn on a water line valve serving a nearby tree farm. The valve was located at the WTP inside an underground valve vault that "always had normal air." The victim entered the valve vault through a ground-level manhole without testing or ventilating the vault atmosphere. A coworker, who had last seen the victim an hour earlier, checked the manhole and saw the victim lying on his back at the bottom. The victim did not respond to any calls. Other workers summoned from the plant building and local fire department personnel removed the victim. The vault atmosphere was subsequently found to be oxygen deficient. There were no witnesses to the incident, but evidence suggests that the victim lost consciousness and fell from the ladder railings to the bottom of the vault. NIOSH investigators determined that, in order to prevent future similar occurrences, employers should

1. Recognize that confined space atmospheres are dynamic environments subject to unexpected changes and address those dynamics in all written and practiced safe work procedures and subsequent worker training.

2. Develop and implement a comprehensive confined space entry program.
3. In addition, municipalities should ensure that police, as well as fire and rescue personnel, are trained in confined space entry and rescue procedures.

Hazard description: Water transmission systems frequently have below-grade vaults that contain large system valves. Utility workers enter the valve vaults to operate valves, inspect or repair equipment, or during construction. Valve vaults are confined spaces when the entry is through a hatch or manhole and down a ladder. When there is possibility of a hazardous atmosphere (oxygen deficiency, flammable or toxic atmosphere), they become permit-required confined spaces. All permit-required confined space entries require atmospheric testing, control of engulfment, ventilation, a written permit, retrieval equipment, standby attendant, communications plan, and emergency plan.

What is a confined space? The water industry has many locations classified as confined spaces, such as pipelines, manholes, tanks and reservoirs, vaults, and similar. These spaces can be entered by a worker, but they are hard to get in or out of, especially in an emergency. They lack fresh air and are not designed for continuous human occupancy. When they contain or may contain a hazard, they are classified as *permit-required confined spaces*, and they require strict controls and procedures for employees to enter them safely. The hazards may include lack of oxygen (if less than 19.5%), flammable or explosive atmospheres (if over 10% of the lower explosive limit or LEL), other toxic chemicals in the atmosphere (such as hydrogen sulfide or carbon monoxide), or physical hazards (such as moving equipment). In the water industry, engulfment is also a serious hazard. When safety procedures are not followed, serious injuries or fatalities can quickly result.

Prevention through design strategies: Prevention through design (PtD) is the effort by design engineers to make safer workplaces through implementing safety at the project design stage (see Chapter 23). Valve vaults can be designed to make them not confined spaces by improving access—through doors or large hatches and stairways to lower levels—and by providing adequate mechanical ventilation (see Fig. 22.6).

When there is not enough room to include stairways, the valve vault remains classified as a confined space; however, there are strategies that improve safety for workers, such as

1. Eliminate the need for workers to enter the vault for valve operation by moving the valve operator above grade.
2. Locate gauges and controls above grade.
3. Install a mounting sleeve for a retrieval device next to the hatch, to facilitate lifting out workers in an emergency.

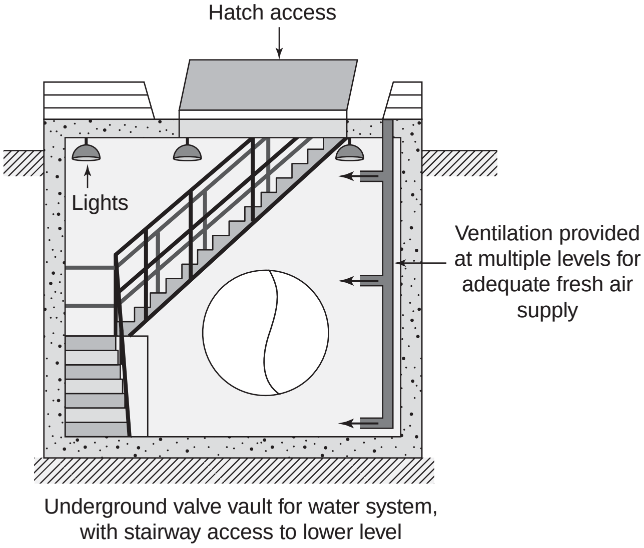


Figure 22.6 Safety in valve vault design (After NIOSH).

22.7.2 Safety in Chemical Handling

Employees at a treatment plant were required to mix polymer for use in the treatment process. The dry polymer powder came in 55 pound bags on pallets. The employees had to lift the bags, place them on a conveyor table, slit the bags open, and then shake the bags to empty the polymer into a hopper above the mixing tank. Employees were having frequent back and shoulder injuries from manipulating the heavy bags, plus polymer dust spread in the workers' faces and around the floor. When the polymer dust got wet, the floor became extremely slippery, and employees were also having fall injuries.

Hazard description: Repeated lifting and handling of heavy bags, in awkward positions, contributes to injuries of the muscles and tendons and joints, known as musculoskeletal injuries.

Prevention through design strategies: Polymer can be purchased as dry powder or liquid, and in bags or large totes. The totes are moved by a hoist system to a fixed location and then pumped, or lifted to discharge directly into a mixing tank (see Fig. 22.7).

When the treatment plant replaced the 55 pound bags of powdered polymer with a tote system moved by a hoist, the employee musculoskeletal injuries were eliminated. With less polymer spilling on the floor, the fall injuries were also significantly reduced.

22.7.3 Safety in the Placement of Valves

A water utility worker was required to manually operate a valve in a newly built treatment plant pump station. The valves in this facility were approximately 5–6 ft (1.5–1.8 m) above the floor, with some of the valve handles in a horizontal position and some in a vertical position. There was no stable

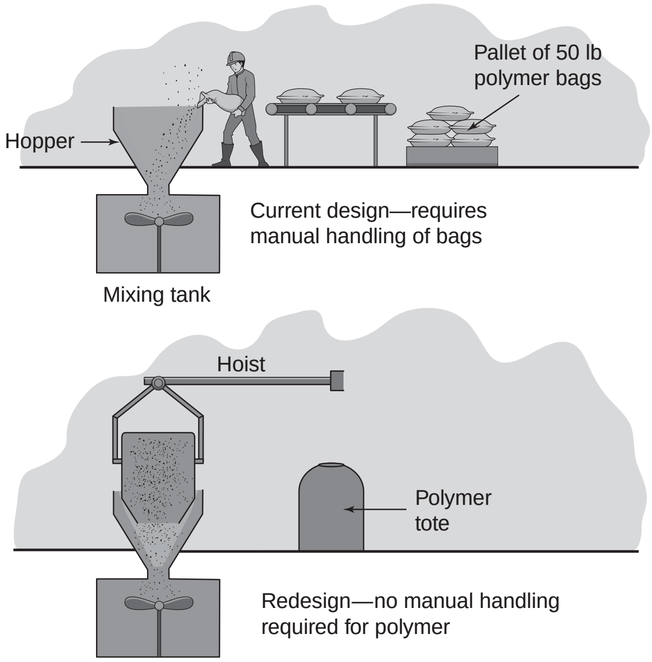


Figure 22.7 Safety in chemical handling (After NIOSH).

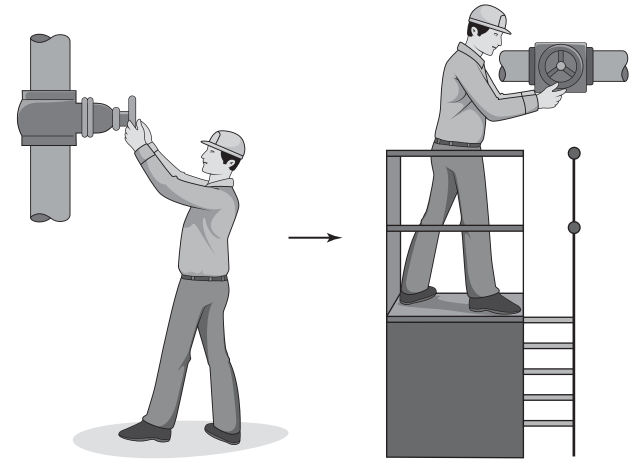
location for the employee to stand to easily reach the valve handle, so he stood on a nearby pipe. The round pipe surface was difficult to stand on, and the employee fell, suffering serious back injuries.

In this same water utility, another pump station had valve handles that were only a 1 or 2 ft (0.30–0.60 m) off the floor. While there had been no serious injuries at this pump station, workers found it very difficult to get maximum leverage while leaning over to turn the valve handle.

Hazard description: WTPs, pump stations, pipelines, and associated facilities use countless valves to control water flow. Large valves are typically operated with electrically powered operators; however, the valves may be operated by hand in certain situations. Also, older equipment and smaller valves may be strictly hand operated.

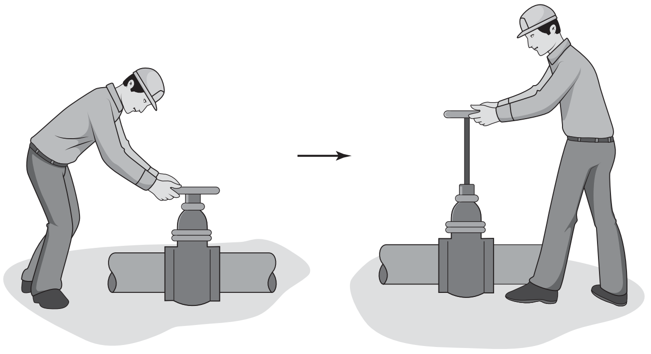
To gain mechanical advantage, valve operation requires many turns of the handwheel by the worker. Workers are able to generate maximum sustained effort when they are working close to their body and the handwheel is between waist and shoulder height.

Prevention through design strategies: Design strategies for valves include bringing the valve handle closer to the worker or bringing the worker closer to the valve (see Fig. 22.8). However, the best approach is to have electrically powered valves whenever possible.



Poor practice—valve handle too high

Good practice—provide platform for operator



Poor practice—valve handle too low

Good practice—extend valve stem

Figure 22.8 Safety in the placement of valves (After NIOSH).

22.7.4 Safety in Water System Perimeter Fall Protection

A water agency built a new pump station, using green building design approaches, including a “green” or living roof. The pump station was located in a residential neighborhood and efforts had been made to design a structure that would blend well with the nearby homes and trees. The 15-ft (4.5-m) high concrete roof was covered with a living layer, consisting of a waterproof liner, topped with soil and native plants, and a watering system. There was a low, 6 in. berm (150 mm) around the roof edge. No roof access was provided for workers. When a number of the plants near the roof edge needed to be replaced, the site supervisor questioned how employees could safely access the roof and work near the edge.

Hazard description: Although living roofs are new, the safety hazards associated with accessing equipment located in high, unprotected locations are common in water facilities. Utility workers need to access equipment during operations, maintenance, installation of new electronics, or simply changing light bulbs. They need safe access to the elevated work locations and fall protection to prevent falls while doing their tasks. In the living roof situation, the supervisor suggested a simple extension ladder leaned against the building, but there was nothing to tie the ladder to. Once on the roof, workers were at risk of a serious or fatal fall when working near the edge.

Prevention through design strategies: PtD is the effort by design engineers to make safer workplaces through implementing safety at the project design stage:

- Roofs (or other elevated locations) that workers need to regularly access must be provided with safe means to access, such as installed ladders.
- Once on the roof, workers can be protected by perimeter railings, parapets at least 40 in. (1.0 m) high, or by fall arrest/fall restraint systems that prevent the worker from falling to the ground or from getting too close to the roof edge.
- For employee access to elevated work locations such as valves, or equipment requiring calibration, install platforms.
- Alternatively, equipment such as light fixtures should be located where lift devices, such as scissor lifts, can maneuver.

Engineers for the pump station with the living roof considered installing a horizontal cable system for workers in harnesses to connect to, but decided to install a guardrail system to give maximum mobility to care for the plants (see Fig. 22.9).

22.8 EMERGENCY RESPONSE PLAN

Safe and reliable drinking water is vital to every community. Emergency response planning is an essential part of managing a drinking water system. The purpose of this section is to provide guidance on developing or revising emergency response plans (ERPs) for community drinking water systems. An ERP is a documented plan that describes the actions that a community water system (CWS) would take in response to various major events. A major event refers to

- Credible threats, indications of terrorism, or acts of terrorism
- Major disasters or emergencies such as hurricanes, tornadoes, storms, earthquakes, fires, flood, or explosion regardless of cause
- Catastrophic incidents that leave extraordinary levels of mass casualties, damage, and disruption severely affecting the population, infrastructure, environment, economy, and government functions

On June 12, 2002, the Public Health Security and Bioterrorism Preparedness and Response Act of 2002 (referred to as the Bioterrorism Act) was signed into law. In the Bioterrorism Act, Congress recognizes the need for drinking water systems to undertake a more comprehensive view of water safety and security. The Act amends the Safe Drinking Water Act and specifies actions CWSs and the US Environmental Protection Agency (US EPA) must take to improve the security of the nation's drinking water infrastructure.

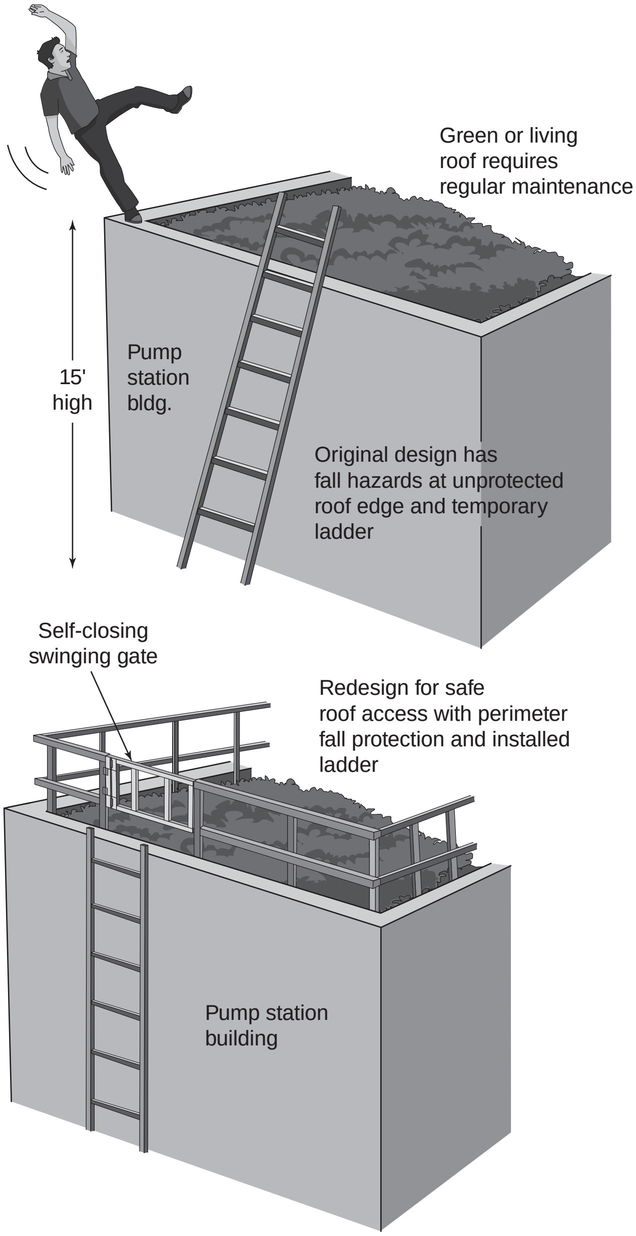


Figure 22.9 Perimeter fall protection (After NIOSH).

Protecting public health is the primary goal of community drinking water systems, and having an up-to-date and workable ERP helps achieve this goal in any crisis situation. The Bioterrorism Act requires CWSs to either prepare or revise an ERP that incorporates the results of its vulnerability assessment (VA). The ERP must include plans, procedures, and identification of equipment that can be implemented or utilized in the event of a terrorist or other major disasters on the CWS. The ERP also must include actions, procedures, and identification of equipment which can obviate or significantly lessen the impact of attacks or other disasters on the public health and the safety and supply of drinking water provided to communities and individuals.

The US EPA Guidance Document describes five steps that will assist CWS in developing or revising their ERPs. These steps are

1. *Actions prior to developing an ERP*: Actions the CWS needs to complete before it could successfully develop or revise its ERP.
2. *Emergency response plan core elements*. Completion of emergency response plan core elements, which are universal to any ERP.
3. *Putting the ERP together and ERP activation*: Steps and issues need to be addressed once all core ERP elements are in place and put together into a single comprehensive plan. Additionally, the CWS needs to understand the types of events that will trigger use of the plan. An effective ERP now needs to address intentional acts of terrorism as well as other emergencies and natural disasters. Planning for these events makes developing, updating, and deciding to activate the ERP more challenging than in the past.
4. *Action plan*. Action plans (APs) are developed and used to tailor emergency response actions to specific incidents or events. Under the Bioterrorism Act, the CWS is required to address the findings of a VA in an ERP. An AP identifies the steps to take to address specific vulnerabilities and respond to a given incident.
5. *Next steps*. Steps to take after completing an ERP, for example, submitting a certification to US EPA, conducting training, and updating the plan.

The ERP may contain sensitive information, so the CWS should consider steps that it needs to take to ensure the security of the ERP. Sensitive information should be placed in appendices or in sections that are not readily available to unauthorized personnel. The ERP, however, should be easily accessible to authorized personnel and should be easily identifiable during a major event. Steps taken to limit access by unauthorized persons should consider local and state Freedom of Information Act (FOIA) laws. Alternatively, the CWS can opt to make its ERP general in nature so that everyone can use it and not include specific information about system vulnerabilities.

A secure copy of the ERP should be maintained in an off-premises location in the event that the primary copy cannot be accessed.

22.9 ACTIONS PRIOR TO DEVELOPING AN ERP

Before beginning to develop or revise an ERP, there are two steps that the CWS needs to take. First, CWS needs to have completed a VA as required under the Bioterrorism Act. The findings from the VA will be addressed in the ERP through specific action plans (APs). Second, the CWS should identify and coordinate with first responders and ERP partners who will help and assist the CWS during a major event. As required by the Bioterrorism Act, partners should include, to the extent possible, Local Emergency Planning Committees (LEPCs) established under the Community Right-to-Know Act such as local law enforcement departments, fire departments, health departments, local environmental agencies, hospitals, broadcast and print media, community groups, and nearby utilities. Other partners could include state and federal agencies and laboratories. Figure 22.10 shows local entities (in the shaded area) as well as state and federal entities (in the white area) that may assist the CWS during a major event. US EPA strongly recommends that CWSs consult with

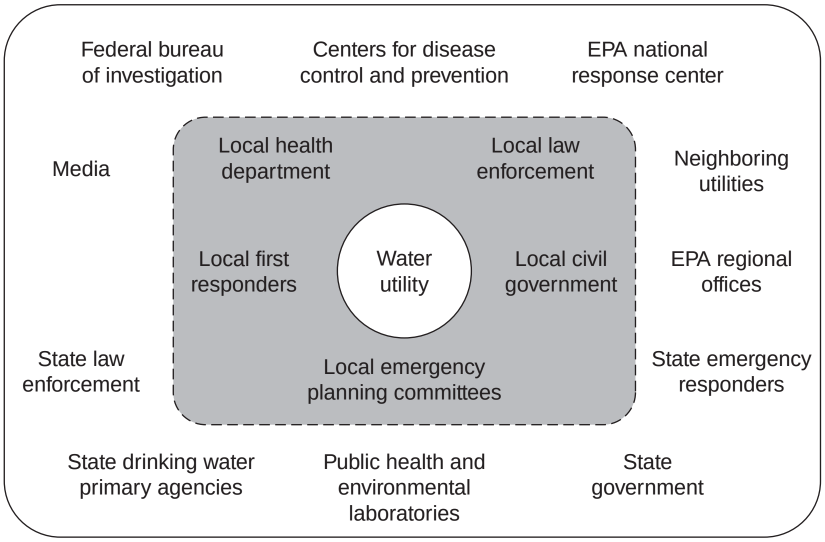


Figure 22.10 Overview of emergency response plan partners (*Source*: US EPA).

local and state entities as they develop their ERPs. The purpose of the consultation is to form partnerships and seek advice. Through these partnerships, each party knows and understands its role and responsibilities in emergency situations. These partnerships help everyone respond better to an emergency.

22.10 EMERGENCY RESPONSE PLAN CORE ELEMENTS

Core elements form the basis, or foundation, for responding to any major event. US EPA has identified eight core elements common to an ERP that CWS should plan to utilize or bring to bear during water emergencies. These are

1. System-specific information
2. CWS roles and responsibilities
3. Communication procedures: who, what, and when
4. Personnel safety
5. Identification of alternate water sources
6. Replacement equipment and chemical supplies
7. Property protection
8. Water sampling and monitoring

22.10.1 System-Specific Information

During a major event, the CWS needs to have basic technical information readily available for its personnel, first responders, repair contractors/vendors, the media, and others. The information needs to be clearly documented and readily accessible so the staff can find and distribute it quickly to those who may be involved in responding to the major event. The location of critical documents, such as distribution maps, detailed plan drawings, site plans, source water locations, and operations manuals, should be identified and readily available during a major event. The CWS should have located and reviewed much of this information while conducting its VA, so the ERP should only need to identify it.

Basic information that may be presented in an ERP includes

1. Public water system identification, owner, administrative and alternate administrative contact persons
2. Population served and service connections
3. Water distribution map
4. Pressure boundary map
5. Overall process flow diagrams
6. Site plans and facility "as-built" engineering drawings: pumping and storage facilities, reservoirs and retention facilities, water treatment facilities, booster pump stations, pressure-regulating valve sites, distribution system process and instrumentation diagrams, equipment and operations specifications, emergency power and light generation, and maintenance supplies
7. Operating procedures and system descriptions, backup systems, and interconnections with other systems
8. Supervisory control and data acquisition (SCADA) system/process control systems operations
9. Communications system operation
10. Site staffing rosters and employees' duties and responsibilities
11. Chemical handling and/or storage facilities and release impact analyses (i.e., chemical releases into air or water)

22.10.2 CWS Roles and Responsibilities

The CWS should designate an emergency response leader and alternate leader. The leader will be the main point of contact and decision-maker during a major event. This person will have responsibility for evaluating incoming information, managing resources and staff, and deciding on appropriate response actions. This person will also have the responsibility of coordinating emergency response efforts with first responders.

The leader and the alternate leader need to be reachable 24 hours a day, 7 days a week. A clear chain of command, or command structure, should also be established so that personnel and staff know their individual roles and responsibilities. If the CWS have the resources and staff, it may consider forming an emergency team that uses a well-defined command structure. At a minimum, your ERP should include the following basic information.

22.10.3 Communication Procedures (Who, What, and When)

Appropriate and timely communication is essential during an emergency. The ERP should identify clear communication channels for CWS staff and personnel, external non-CWS entities, and the public/media. As part of the ERP, the CWS should maintain internal and external notification lists that contain information on all appropriate entities to be contacted, including their names, titles, mailing addresses, e-mail addresses, all applicable land line and cellular phone numbers, and pager numbers. These lists should be updated as necessary. In a major event (e.g., a terrorist attack), it may not be possible to use normal channels of communication. Provisions need to be made for an efficient and fail-safe form of communication to be available during conditions when the use of normal means may not be possible. Communication procedures with the public and media may already be part of the CWS day-to-day operations, but these procedures need special attention during a major event in order to provide the public and media with timely, accurate, and complete information. Critical users (e.g., hospitals) and commercial and industrial customers such as those that incorporate water into their product (e.g., bottling and canning companies) should also be on the contact list. Some of these users should be given priority notification due to their public health mission and because they may serve customers considered "sensitive subpopulations" (e.g., senior residential housing, child care centers, medical facilities). Specific notification procedures should be developed for these groups.

22.10.4 Personnel Safety

Protecting the health and safety of everyone in their CWS as well as the surrounding community is a key priority during an emergency. During an emergency, personnel may be at risk of harm, injury, or even death. This section of the ERP should provide direction to personnel on how to safely implement a variety of response actions. When considering personnel safety the following factors should be taken into account:

1. *Evacuation planning*: Develop a CWS evacuation policy and procedures.
2. *Evacuation routes and exits*: Designate primary and secondary evacuation routes and ensure that they are clearly marked, well lit, unobstructed at all times, and unlikely to expose evacuating personnel to additional hazards.
3. *Assembly areas and accountability*: Obtaining an accurate account of personnel requires planning and practice. Designate assembly areas where personnel should gather after an evacuation and specify procedures for taking a head count and identifying personnel.
4. *Shelter*: In some major events, the best means of protection is to take shelter either within the CWS or away from the CWS in another building.
5. *Training and information*: Train staff and personnel in evacuation, shelter, and other safety procedures.
6. *Emergency equipment*: Consider developing written procedures for using and maintaining the emergency response equipment. This should apply to any emergency equipment relevant to a response involving a toxic chemical, including all detection and monitoring equipment, alarms and communications systems, and personal protective equipment not used as part of normal operations.
7. *First aid*: Discuss proper first aid and emergency medical treatment for employees and others who are onsite at the CWS. This should include standard safety precautions for victims as well as more detailed information for medical professionals.

22.10.5 Identification of Alternate Water Sources

The CWS should consider the amount of water needed to address short-term (hours to days) and long-term (weeks to months) outages. As part of the ERP, the CWS should identify the alternate water supplies available during both types of outages. To do this, the CWS needs to have a comprehensive understanding of its current water supply, its water distribution system, and its water system demand requirements. It should clearly understand the location and capabilities of other regional CWS, including available excess capacity and ease of connection to its distribution system. In addition, it should also understand the interconnection agreements that partners have in place and potential issues that could arise if multiple CWS are affected.

Short-term outages might be due to contamination or electrical power outages. If the CWS has been contaminated, a public health notification such as "boil water," "do not drink," or "do not use" may be issued by the drinking water primacy agency. If a "boil water" notice is issued, no alternative water source is needed. If a "do not drink" order is issued, then the suspect water can still be used for other activities that do not involve ingestion of the water. In this situation, it will only be necessary to provide an alternate drinking water supply for consumption and related activities such as food preparation.

A "do not use" order is much more restrictive. The CWS will need sufficient alternate water sources to supply water for consumption, hygiene, and emergency needs. A "do not use" notice may also have implications with respect to water used for firefighting. Although a prohibition on use of water for firefighting is likely to occur only if the water is contaminated with certain substances, an alternate source of firefighting water, such as a pond, river, or stream, may be necessary in this event.

As part of the ERP, the CWS should consider the potential effects of a power outage. The utility could be without power due to a major event, and it may take several days for power to be restored. The plan should include contingencies for backup power generation and alternative power sources.

As part of the ERP, the CWS should also identify agencies or private companies that could provide water supplies (bottled or bulk) in the event of a major event and establish mutual aid agreements with surrounding communities, industries, contractors, and related utilities as appropriate. Possible short-term alternate water supply options include (but are not limited to) the following: bottled water provided by outside sources; (b) bottled water provided by local retailers; (c) bulk water provided by certified water haulers; (d) bulk water transported or provided by military assets (i.e., National Guard or US Army Corps of Engineers); (e) bulk water provided by neighboring water utilities by truck or via pipeline; (f) bulk water from hospitals, universities, and local industry that maintain backup water supplies for consumption; (g) interconnections with nearby public water systems; (h) water treated by plant and hauled to distribution centers (i.e., in the case of water distribution system contamination); (i) water pumped from surface water sources, treated at the plant or nearby plants, and hauled to distribution centers; (j) water for firefighting from federal agencies such as the US ACE and FEMA; and (k) water from unaffected wells owned by local citizens and businesses.

If the CWS will need extensive cleaning, or if portions of the system have been destroyed, the CWS will need a long-term alternate water supply. The following are examples of possible long-term water supply options:

1. Connection of the water distribution system to an existing municipal or private water supply (assumes existing WTP and distribution system is intact and useable)
2. Connection of the water distribution system with a new uncontaminated groundwater or surface water source (assumes existing WTP and distribution system is intact and useable)
3. Development of new water distribution system (assumes existing WTP and source water is uncontaminated and useable)
4. Development of oversized community storage facilities to compensate for loss of existing system capacity

22.10.6 Replacement Equipment and Chemical Supplies

The ERP should identify equipment that can significantly lessen the impact of a major event on public health and protect the safety and supply of drinking water. The CWS should maintain an updated inventory of current equipment (e.g., pumps), repair parts, chemical supplies for normal maintenance and operations, and information on mutual aid agreements.

Based on the findings of its VA, the CWS should identify how and where to find the equipment, repair parts, and chemicals that would be needed to respond adequately to a particular vulnerability. The CWS should consider establishing mutual aid agreements with other CWS to address any deficiencies. These agreements should identify the equipment, parts, and chemicals available to the CWS under the agreement.

22.10.7 Property Protection

Protecting CWS facilities, equipment, and vital records is essential to restoring operations once a major event has occurred. The ERP should identify measures and procedures that are aimed at securing and protecting the CWS following a major event. Items that should be considered include (a) "lock down" procedures, (b) access control procedures, (c) establishing a security perimeter following a major event, (d) evidence protection measures for law enforcement (should the major event also be declared a crime scene), (e) securing buildings against forced entry, and (f) other property protection procedures and measures.

22.10.8 Water Sampling and Monitoring

Water sampling and monitoring should be an integral part of the ERP and not an afterthought. How else can the CWS determine whether the drinking water that it supplies is safe for public consumption and use? During the stage of forming partnerships the CWS should consult with the State Drinking Water Primacy Agency on the issues of water sampling and monitoring. In the ERP the CWS will need to identify and address special water sampling and monitoring issues that may arise during and after a major event. Some water sampling and monitoring issues to consider include (a) identifying proper sampling procedures for different types of contaminants, (b) obtaining sample containers, (c) determining the quantity of required samples, (d) identifying who is responsible for taking samples, (e) identifying who is responsible for transporting samples (in times of sensitive situations), (f) confirming laboratory capabilities and certifications, and (g) interpreting/monitoring of laboratory results.

22.11 PUTTING THE ERP TOGETHER AND ERP ACTIVATION

The next goal is to put all the collected information together and produce a single comprehensive ERP document that is accessible to appropriate personnel and that can be updated or modified as the need arises. The ERP document may be organized into overall ERP polices, general ERP procedures, any mutual aid agreements, reference documents, and APs.

How a CWS ultimately organizes and documents its ERP is up to it and can depend on whether the CWS is developing an ERP from scratch or is revising an existing ERP. Other federal and state requirements may also influence how a CWS organizes and documents its ERP. The Bioterrorism Act requires that a copy of the ERP be kept for 5 years after the CWS send its ERP certification to US EPA. A secure copy of the ERP should also be maintained in an off-premises location in the event that the CWS copy cannot be accessed.

Knowing when to activate or set the ERP in motion is as important as having a prepared and documented ERP. In the past, emergency response mostly dealt with emergencies such as natural disasters and accidents. The definition of a

Figure 22.11 Summary of potential threat warnings (*Source*: US EPA).

"major event" now includes a major disaster or other emergency as well as a terrorist attack. Being prepared to respond to a terrorist attack requires special attention.

CWS should pay attention to any "threat warning." The Homeland Security Advisory System contains five threat condition levels. Low Condition (Green) is declared when there is a low risk of terrorist attacks. Guarded Condition (Blue) is declared when there is a general risk of terrorist attacks. Elevated Condition (Yellow) is declared when there is a significant risk of terrorist attacks. High Condition (Orange) is declared when there is a high risk of terrorist attacks. Finally, Severe Condition (Red) reflects a severe risk of terrorist attacks. US EPA has issued supplemental guidance for water utilities to increase security based on threat conditions described by the five-tiered Homeland Security Advisory System.

A "threat warning" is an occurrence or discovery that indicates a threat of a malevolent act and triggers an evaluation of the threat. These warnings should be evaluated in the context of typical CWS activity and previous experience in order to avoid false alarms. The threat warnings presented in Fig. 22.11 are intended to be inclusive of those most likely to be encountered, but this listing is by no means comprehensive of all possibilities.

Once a threat warning is received, the threat decision (or threat evaluation) process begins. The emergency leader or his alternate should be notified immediately because they will be involved in this decision process and make decisions about who else (e.g., other emergency responders) should be involved. The threat decision process is considered in three successive stages: "possible," "credible," and "confirmed." Upon confirmation of an incident, CWS should fully implement its ERP. The ERP should contain action plans that address specific major events, and these action plans should be implemented immediately. The emergency response leader should work through the threat decision process and implement the ERP as needed.

22.12 ACTION PLANS

Action plans (APs), also known as response guidelines, are tailored ERPs that address specific major events. APs describe response actions to take for events that a CWS thinks might occur at its facility based on the specific vulnerabilities identified in its VA. An AP should provide a quick approach for responding to a specific major event and it complements actions already initiated under the ERP. CWS may only need one to two pages to cover specific response information since it has already addressed basic emergency response steps in the core elements of its ERP. An AP should be an accessible (i.e., "rip and run") document that can be detached and taken to the field by the ER leader or his alternate. An AP should include the following basic information: (a) any special notification requirements, (b) special response steps to be taken upon ERP activation, and (c) recovery actions to bring the CWS back into operation.

The Bioterrorism Act requires that prepared or revised ERPs incorporate the results of completed VAs. During the VA process, CWS should have determined its high-priority vulnerabilities. An AP defines the specific actions it would take to respond to events where its high-priority vulnerabilities have been compromised. In addition, US EPA recommends that the CWS develops APs for certain high-consequence events regardless of whether these are among its high-priority vulnerabilities. Events and threats of events that should be considered in APs include the following:

1. Contamination of the drinking water
2. Structural damage/physical attack
3. SCADA, computer, or cyber attack
4. Intentional hazardous chemical release (e.g., release of chlorine or ammonia from storage)

Two example APs for the events listed above are included in Examples 22.3 and 22.4. It should be noted that these simplified examples are for discussion purposes only and that the CWS should develop "APs" specific to the needs of its water system and surrounding community. APs should be easily accessible to authorized personnel and should be easily identifiable during a major event. However, the CWS may want to limit access to APs containing sensitive information and specifics related to intentional events. Again, steps taken to limit such access should consider local and state FOIA laws. Alternatively, the CWS can opt to make its APs general in nature so that everyone can use them and place sensitive information in the ERP appendices, or in sections that are not readily available to unauthorized personnel. The CWS may wish to develop individual APs for other major events that are not terrorist related. A similar approach to that described above could be used to plan for natural disasters and other significant events.

EXAMPLE 22.3 ACTION PLAN FOR WATER SYSTEM CONTAMINATION

Formulate an action plan (AP) in case of contamination of a water system listing the special actions and notifications to be taken for the following threat conditions:

- Threat warning received
- Is the threat possible? (Stage 1)
- Is the threat credible? (Stage 2)
- Has the incidence been confirmed? (Stage 3)

Solution:

Action Plan
Water System Contamination
Threat Warning Stage

Threat Warning Received	Special actions and notifications to be taken: • Notify ER Lead or Alternate ER Lead • Record and document all information pertaining to the threat warning • Do not disturb site if the threat warning could be a possible crime scene • Return to normal operations if no further action is required (i.e., the threat warning can be explained) • Begin the "Threat Decision Process" if the threat warning cannot be explained

Threat Decision Process Stage

Is the Threat Possible? (Stage 1)	Special actions and notifications to be taken: • Notify local law enforcement • Notify State Drinking Water Primacy Agency • Evaluate threat warning and make decisions in consultation with State Drinking Water Primacy Agency and local law enforcement • Initiate basic precautionary measures: 1. Alert staff and personnel about threat warning 2. Prepare additional notification lists if the situation escalates to the "Is the Threat Credible?" stage

If the threat is not possible, then return to normal operations. Otherwise, proceed to "Is the Threat Credible" stage.

Is theThreat credible? (Stage 2)	Special actions and notifications to be taken: • Activate notification and personnel safety portions of ERP • Evaluate whether the threat is credible in consultation with assisting agencies • Visually inspect physical evidence and determine whether there is a change in normal system operating parameters (i.e., chlorine residuals, turbidity, odor, color, pH.) • Conduct actions and testing as recommended by monitoring and sampling experts

If the threat is not credible, then return to normal operations. Otherwise, proceed to "Has the Threat been Confirmed" stage.

Has the Incident Been Confirmed? (Stage 3)	Special actions and notifications to be taken: • Initiate full ERP activation • Follow State Incident Command System • Isolate portion of system or backflush • Shut down system if obvious or confirmed contamination warrants • Issue public notice and issue follow-up media press releases • Continue sampling and water monitoring • Assess need to remediate storage tanks, filters, sediment basins, solids handling, etc.

Source: US EPA.

EXAMPLE 22.4 ACTION PLAN FOR WATER SYSTEM STRUCTURAL DAMAGE/PHYSICAL ATTACK

Formulate an action plan (AP) in case of contamination of a water system listing the special actions and notifications to be taken for the following threat conditions:

- Threat warning received
- Is the threat possible? (Stage 1)
- Is the threat credible? (Stage 2)
- Has the incidence been confirmed? (Stage 3)

Solution:

ACTION PLAN

Structural Damage/Physical Attack to Water System or Facility(ies)

Threat Warning Stage

Threat Warning Received	Special actions and notifications to be taken: • Notify ER Leader or Alternate ER Leader • Record and document all information pertaining to the threat warning • Do not disturb site if the threat warning could be a possible crime scene • Return to normal operations if no further action is required (i.e., the threat warning can be explained) • Begin the "Threat Decision Process" if the threat warning cannot be explained

Threat Decision Process Stage

Is the Threat Possible? (Stage 1)	Special actions and notifications to be taken: • Notify local law enforcement • Notify State Drinking Water Primacy Agency • Evaluate threat warning and make decisions in consultation with State Drinking Water Primacy Agency and local law enforcement • Initiate basic precautionary measures: 1. Alert staff and personnel about threat warning 2. Heighter security at critical facilities 3. Prepare additional notification lists if the situation escalates to the "Is the Threat Credible?" stage

If the threat is not possible, then return to normal operations. Otherwise, proceed to "Is the Threat Credible" stage.

Is theThreat Credible? (Stage 2)	Special actions and notifications to be taken: • Activate notification and personnel safety portions of ERP • Physically secure water system facilities • Evaluate whether the threat is credible in consultation with assisting agencies

If the threat is not credible, then return to normal operations. Otherwise, proceed to "Has the Threat been Confirmed" stage.

Has the Incident Been Confirmed? (Stage 3)	Special actions and notifications to be taken: • Initiate full ERP activation • Follow State Incident Command System • Deploy damage assessment team • Isolate damaged facility from rest of water system • Coordinate alternative water supply, as needed, or consider alternate (interim) treatment schemes • Issue public notice and issue follow-up media press releases • Repair damaged facilities • Assess need for additional protection/security measures

Source: US EPA.

22.13 NEXT STEPS

Once a CWS prepares or revises its ERP, it is required under the Bioterrorism Act to submit a written certification stating that the plan has been completed. It is important to note that an ERP is a "living" document that should be updated periodically (i.e., at least annually or if there is a major change to the CWS configuration). The ER leader and appropriate CWS management staff should approve the ERP and identify the time period for routinely updating the ERP (e.g., annually). Updates should occur if there are changes in CWS staff, internal and external contacts, and roles and responsibilities of anyone involved in response, or there are changes made in infrastructure.

The CWS should make sure that its staff is trained on their ERP responsibilities. Training can include briefing sessions, classroom sessions, or mock exercises. CWS should also remember to do "refresher" training on a regular basis. Training should include testing of the ERP. Drills and exercises that challenge the information in the ERP should be conducted at least annually. There are many sources (state, federal, and industry specific) that describe what should be included in emergency training. These typically include the following four types of training:

1. *Orientation sessions*: Orientation sessions work well for basic instruction and explaining ERP procedures. Written tests may be employed to ensure some level of comprehension by the attendees.
2. *Table-top workshop*: Table-top workshops involve developing scenarios that describe potential problems and provides certain information necessary to address the problems. The idea is to present staff and emergency response officials with a fabricated event, have them verbally respond to a series of questions, and then evaluate whether the responses match what is written in the ERP.
3. *Functional exercises*: The functional exercise is considered the most effective training tool, next to a real emergency, because a team of simulators is trained to develop a realistic major event. By using a series of prescripted messages, the simulation team sends information in to personnel assigned to carry out the ERP procedures. Both the simulators and personnel responding to the simulation are focused on carrying out the procedures to test the validity of the ERP.
4. *Full-scale drills*: These are the most costly and time-consuming training programs but can be extremely effective. In a full-scale drill, emergency response personnel and equipment are mobilized to a scene, an emergency scenario is presented, and they respond as directed by the ERP.

The bottom line is that time, resources, and personnel need to be dedicated to accomplishing the training. Use the training to identify lessons learned, debrief staff of lessons learned to enhance future response and recovery efforts, and update plans to incorporate lessons learned.

PROBLEMS/QUESTIONS

22.1 What is the purpose of using the following water treatment processes: (a) clarification (either sedimentation or flotation), (b) coagulation/filtration, (c) water softening, (d) membrane separation, (e) ion exchange, and (f) granular activated carbon?

22.2 What steps need to be considered when developing a comprehensive water treatment residuals management plan?

22.3 What are the four major types of residuals produced from water treatment processes?

22.4 What are the common residual treatment processes? Briefly mention the purpose of each process?

22.5 What are the disposal methods for water treatment residuals?

22.6 What are the primary elements of a residuals management plan?

22.7 What is the best way to maintain the safety of all personnel in a community water supply system and prevent problems and accidents?

22.8 What are the strategies that improve safety for workers in confined spaces such as valve vaults?

22.9 What are the five steps that will assist a community water system in developing their emergency response plan? Briefly explain each step.

22.10 What are the core elements of an emergency response plan?

22.11 Discuss the possible alternative water sources that a community water system need to address short-term and long-term water outages during an emergency?

22.12 Discuss the types of personnel training that a community water system should undertake in order to make sure that its staff is trained on their ERP responsibilities.

22.13 Formulate an action plan (AP) in case of cyber attack on SCADA or the operational computer of a water system listing the special actions and notifications to be taken for the following threat conditions:

- Threat warning received
- Is the threat possible? (Stage 1)
- Is the threat credible? (Stage 2)
- Has the incidence been confirmed? (Stage 3)

22.14 Formulate an action plan (AP) in case of hazardous chemical release from water system facilities listing the special actions and notifications to be taken for the following threat conditions:

- Threat warning received
- Is the threat possible? (Stage 1)
- Is the threat credible? (Stage 2)
- Has the incidence been confirmed? (Stage 3)

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Chapter 23

Prevention through Design and System Safety

Prevention through Design (PtD) is a collaborative initiative in the United States based on the belief that the best way to prevent work-related injuries and illnesses is by anticipating and “designing out” potential hazards and risks at the “drawing board” or as early as possible in the design phase of new processes, structures, facilities, equipment, or tools, and organizing work to take into consideration construction, maintenance, production, and decommissioning operations. The PtD initiative is a positive catalyst to the creation and dissemination of business tools, case studies, demonstration projects, and good engineering practices centered on design solutions that reduce worker injuries, illnesses, and costs.

Many practicing engineers view the ideas behind PtD as a necessity in today’s highly competitive business environment and PtD concepts are maturing in a number of businesses worldwide. These leaders understand that it is less costly to anticipate and minimize workplace hazards and risks early in the design process than to retrofit changes after workers get hurt. Besides the cost of retrofitting, not designing for prevention can lead to increased chemical exposures, ergonomic hazards, explosions, fires, falls, amputations, and so on. PtD helps plan for success by integrating occupational safety and health solutions with key business processes. It also increases efficiency and cost-effectiveness.

The PtD concept is not new. The engineering profession has long recognized the importance of preventing safety and health problems with the designs its members create. Although the PtD initiative today is broad in scope, one of the building blocks that led to its recognized value for businesses is the development of a process called “system safety.” The remainder of this chapter focuses on system safety and how it applies to water and wastewater engineering. Key examples are introduced here and in other chapters of the book where applicable.

23.1 INTRODUCTION TO SYSTEM SAFETY

Systems are analyzed to identify their hazards and those hazards are assessed as to their risks for a single reason: to support management *decision-making*. Management must decide whether system risk is acceptable. If that risk is not acceptable, then management must decide what is to be done, by whom, by when, and at what cost.

Management decision-making must balance the interests of all *stakeholders*: employees at all levels of the company, customers, suppliers, the public, and the stockholders. Management decision-making must also support the multiple goals of the enterprise and protect all of its resources: human, equipment, facility, product quality, inventory, production capability, finance, market position, and reputation.

System safety originated in the aircraft and aerospace industries. *Systems engineering* was developed shortly after World War II. It found application in US nuclear weapons programs because of the complexity of these programs and the perceived costs (risks) of non-attainment of nuclear superiority. Systems engineering seeks to understand the integrated whole rather than merely the component parts of a system, with an aim toward optimizing the system to meet multiple objectives. During the early 1950s, the RAND Corporation developed *systems analysis methodology* as an aid to economic and strategic decision-making. These two disciplines were used in the aerospace and nuclear weapons programs for several reasons: (a) schedule delays for these programs were costly (and perceived as a matter of national security); (b) the systems were complex and involved many contractors and subcontractors; (c) they enabled the selection of a final design from among various competing designs; and (d) there was intense scrutiny on the part of the public and the funding agencies. Over the years, the distinction between systems engineering and systems analysis has blurred. Together, they form the philosophical foundation for system safety. That is, safety can—and should—be managed in the same manner as any other design or operational parameter.

System safety was first practiced by the US Air Force (USAF). Historically, most aircraft crashes were blamed on pilot error. Similarly, in industry, accidents were most commonly blamed on an unsafe act. To attribute an aircraft crash to pilot error or an industrial accident to an unsafe act places very little intellectual burden on the investigator to delve into the design of the system with which the operator (pilot or worker) was forced to coexist. When the USAF began developing intercontinental ballistic missiles (ICBMs) in the

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Water Engineering: Hydraulics, Distribution and Treatment, First Edition. Nazih K. Shammas and Lawrence K. Wang.

1950s, there were no pilots to blame when the missiles blew up during testing.

Because of the pressure to field these weapon systems as quickly as possible, the USAF adopted a concurrent engineering approach. This meant that the training of operations and maintenance personnel occurred simultaneously with the development of the missiles and their launch facilities. Remember that these weapon systems were far more complex than had ever been attempted and that many newly developed technologies were incorporated into these designs. Safety was not handled in a systematic manner. Instead, during those early days, safety responsibility was assigned to each subsystem designer, engineer, and manager. Thus safety was compartmentalized, and when these subsystems were finally integrated, interface problems were detected too late.

The USAF describes one incident in a design manual: an ICBM silo was destroyed because the counterweights, used to balance the silo elevator on the way up and down in the silo, were designed with consideration only to raising a fueled missile to the surface for firing. There was no consideration that, when you were just testing and not firing in anger, you had to bring fueled missile back down to defuel. The first operation with a fueled missile was nearly successful. The drive mechanism held it for all but the last 5 ft (1.5 m), at which point gravity took over and the missile dropped down. Very suddenly, the 40-ft (12.2-m)-diameter silo was altered to about a 100 ft (30.5 m) diameter.

The investigations of these losses uncovered deficiencies in management, design, and operations. The USAF realized that the traditional (reiterative) "fly–crash–fix–fly" approach could not produce acceptable results (because of cost and geopolitical ramifications). This realization led the USAF to adopt a system safety approach, which had the goal of preventing accidents before their first occurrence.

Beyond mere regulatory compliance, companies and utilities are realizing that waiting for accidents to occur and then identifying and eliminating their causes is simply too expensive, whether measured in terms of the costs of modification, retrofit, liability, lost market share, or tarnished reputation.

The principal advantage of a system safety program—compared with a conventional or traditional industrial safety program—is that early in the design stage, the forward-looking system safety program considers the hazards that will be encountered during the entire life cycle. The industrial safety program usually considers only the hazards that arise during the operational phases of the product or manufacturing system.

The system safety techniques allow the analysis of hazards at any time during the life cycle of a system, but the real advantage is that the techniques can be used to detect hazards in the early part of the life cycle, when problems are relatively inexpensive to correct. System safety stresses the importance of designing safety into the system, rather than adding it on to a completed design. Most of the design decisions that have an impact on the hazards posed by a system must be made relatively early in the life cycle. System safety's early-on approach leads to more effective, less costly control or elimination of hazards. System safety looks at a broader range of losses than is typically considered by the traditional industrial safety practitioner. It allows the analyst (and management) to gauge the impact of various hazards on potential "targets" or "resources," including workers, the public, product quality, productivity, environment, facilities, and equipment.

System safety relies on analysis, and not solely on past experience and standards. When designing a new product, no information may be available concerning previous mishaps; a review of history will have little value to the designer. Because standards writing is a slow process relative to the development of new technology, a search for—and review of—relevant standards may not uncover all of the potential hazards posed by the new technology.

System safety is broader than *reliability*. Reliability asks the question, "Does the component or system continue to meet its specification, and for how long?" System safety asks the broader question, "Was the specification correct, and what happens if the component meets (or doesn't meet) the specification?" Reliability focuses on the failure of a component; system safety recognizes that not all hazards are attributable to failures and that all failures do not necessarily cause hazards. System safety also analyzes the interactions among the components in a system and between the system and its environment, including human operators.

When new processes are developed, the designer seldom begins with a blank canvas. Rather, there is a mixture of retained knowledge, combined with new technology that is fashioned into the new design. The retained knowledge (lessons learned) and new technology drive the safety program planning, hazard identification and analyses, and the safety criteria, requirements, and constraints. The designer's "up stream" knowledge of the safety issues allows for the cost-effective integration of safety, health, and environmental considerations at all points of the product life cycle. Knowledge will be gained as the product/process life cycle moves forward. This knowledge or "lessons learned" can be applied at earlier stages of the product life cycle, leading to changes in design, materials, manufacturing methods, inspection, and so on. This approach to continuous process improvement is shown graphically in Fig. 23.1.

Figure 23.1 provides a schematic description of the system safety approach as it is successfully used in various settings, including the design of utilities, semiconductor manufacturing facilities, chemical and food processing plants, air and ground transportation systems, and consumer products. Many modern systems are software-controlled. This has resulted in increasing recognition of the importance of integrating software safety efforts within the system safety program.

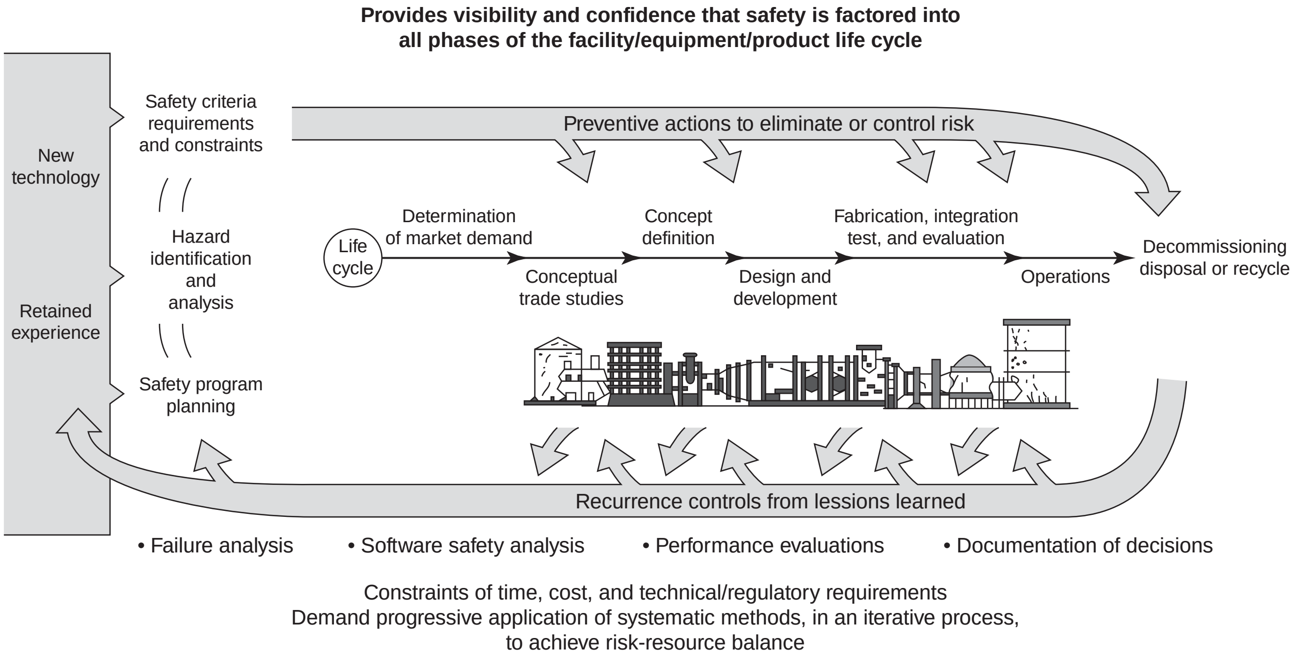


Figure 23.1 Schematic of system safety approach (After NIOSH).

23.2 NATURE AND MAGNITUDE OF SAFETY AND HEALTH PROBLEMS

Many graduates of civil engineering programs frequently find themselves in responsible positions as employees of or consultants to construction project participants, having little or no background or training in safety and health. Their knowledge on this important topic may be less than adequate for providing the optimal professional service in design, management, or supervision. Traditionally, the emphasis in civil engineering curricula has been on the safety of a facility as designed or constructed and little attention has been given to public safety and health, and occupational safety and health during the construction, operation, and maintenance processes. Most of the knowledge on work site safety and health has been gained, and to a large extent continues to be acquired, through experience on the job. This chapter is designed to fill an important gap in the civil engineering curriculum, affording opportunities for teaching and learning the significance of and the principles and practice of safety and health in construction, operation, and maintenance.

The construction industry in the United States is a $846 billion (in 2015 USD) per year industry. About 90% of the total is new construction work, while 10% is maintenance and repairs. Over the years, construction has been a major contributor to the national economy. Its share of the gross national product (GNP) is about 5%. The construction industry employs more than 5 million people annually (3 million full-time jobs). This number represents more than 5% of the national labor force. There are also more than a million contractors and subcontractors engaged in construction work.

Construction projects are conceived, planned, designed, and built by a team usually consisting of the owner, designer-architect/engineer (A/E), and constructor (general contractor and subcontractors). A fourth entity, the construction manager (CM), may also be involved. In addition, support roles are played by material and equipment suppliers, financiers, insurers, regulators, and consultants. Workers carry out the construction tasks.

The construction industry is large but diffuse and fragmented, generally consisting of small units. A number of very large construction firms also exist, each doing more than USD 2 billion business annually, often for very large corporate clients whose construction bills run into billions of dollars. The industry undertakes diverse projects ranging in size from single-family residential units to multibillion-dollar power and treatment plants. The work may encompass general building construction (residential, non-residential, institutional, industrial), heavy construction (highways, public works infrastructure, utilities), and industrial construction (major industrial complexes such as refineries, and power plants).

The construction industry is uniquely different from manufacturing and other industries, in that

1. Its products are site produced and one-of-a-kind. With the exception of perhaps the housing sector, each project can be considered a “first, full-scale prototype.”
2. The construction project is highly dependent on local conditions: site, climate, local codes and regulations, and so forth.

3. The work is very labor-intensive involving large forces and energies. The tasks are very variable; mass production and standardization are very difficult to achieve.
4. The industry brings together diverse entities such as designers, engineers, planners, developers, skilled craftspeople, laborers, suppliers, distributors, managers, lawyers, and regulators. These entities are joined together for a single project usually under complex arrangements and are then dispersed on completion of the work. The workforce is transient in most cases.

Because of the inherent diversity and complexity of construction projects, the industry has traditionally suffered low productivity, a high degree of disputes, litigation, and business failures, and a poor record of site safety. The national as well as state statistics clearly point out that construction work is dangerous, and significant problems need to be addressed concerning safety and health in the construction industry. Although some improvement in certain indicators is apparent, the situation is not where it should be. The National Forum on Construction Safety and Health Priorities identified seven critical issues that need attention from the industry:

1. Accident surveillance
2. Construction team interface
3. Engineering needs
4. Legal/legislative aspects
5. Occupational health
6. Primary safety exposures
7. Education and training

Concerns about *accident surveillance* include proper and meaningful data collection, accident reporting requirements and procedures, data analysis, and contractual aspects, including liability issues. A vastly improved system of accident surveillance is needed to help contractors and regulatory agencies acquire the data significant to the understanding and solution of the problems. Training of the pertinent personnel on the effective implementation of all surveillance-related tasks is essential.

With respect to the *construction team interface*, the overriding problem revolves around the lack of a real understanding of the roles and responsibilities for safety and health by the project participants, that is, the owner, designer, consultant, contractor, and workers. Safety and health aspects in the construction industry are not managed in the same manner as cost, quality, schedule, and productivity, mainly due to the lack of appropriate education and training in this area. The industry needs to fully understand the economic benefits of effective safety and health programs, and owners should screen contractors based on their safety and health records.

Engineering needs deal with the application of engineering principles and skills to accident prevention, particularly with respect to construction erection systems, the construction process, equipment design, and accident data. Many construction accidents and failures can be prevented by proper engineering. Sound structural design rationale is needed for temporary erection systems and criteria for acceptable risk in various situations and environments. Layout controls and operating procedures for construction equipment should be standardized based on ergonomics principles and on an effective accident data collection and analysis system.

The role and effectiveness of *the Occupational Safety and Health Administration* (OSHA) is recognized as the single most important area of concern in regard to legal and legislative aspects. It is suggested that OSHA is not particularly effective in accident prevention due to its focus on regulatory compliance and penalties, and not on providing the appropriate education, training, and incentives for the contractor to work safely. This issue is being rectified through OSHA's "Prevention through Design" initiative.

In terms of *occupational health*, the primary concern is over the applicability of the OSHA hazard communication standard to the construction industry, although its benefits are readily acknowledged. There is a need for a medical screening and monitoring system tailored to the requirements and potential exposures of the job. The need for effective safety and health programs with clearly assigned responsibilities emphasizing the health aspects and hazard control aspects of a particular project requires that a model safety and health program be developed at the project level.

In terms of *primary safety exposure* the root causes of accidents include (a) lack of company programs and written rules; (b) lack of basic, specific, and continuing training; (c) lack of planning, organization, motivation, communication, and control; (d) lack of a sound value system, for example, attitude, loyalty, and sensitivity; (e) poor work practices and improper procedures; and (f) lack of accountability and responsibility. The key factors for solutions to these problems can be identified as effective education and training, and the implementation and enforcement of well-designed safety and health programs.

Education and training are, by far, the most important issue and need in the construction industry at all levels, from owners, architects, design engineers, and construction management personnel, all the way through to the site supervisors and foremen and on to the workers. Lack of adequate education and training is a part of many problems related to construction safety and health, and the need for it is strongly emphasized for devising effective solutions to the problems. An awareness of safety and health needs has to be fostered by training and must be phased into the educational system very early, such as in grade schools and vocational training centers. The earlier mentioned national forum particularly recommended that safety and health education become a required part of the engineering curriculum in the United States.

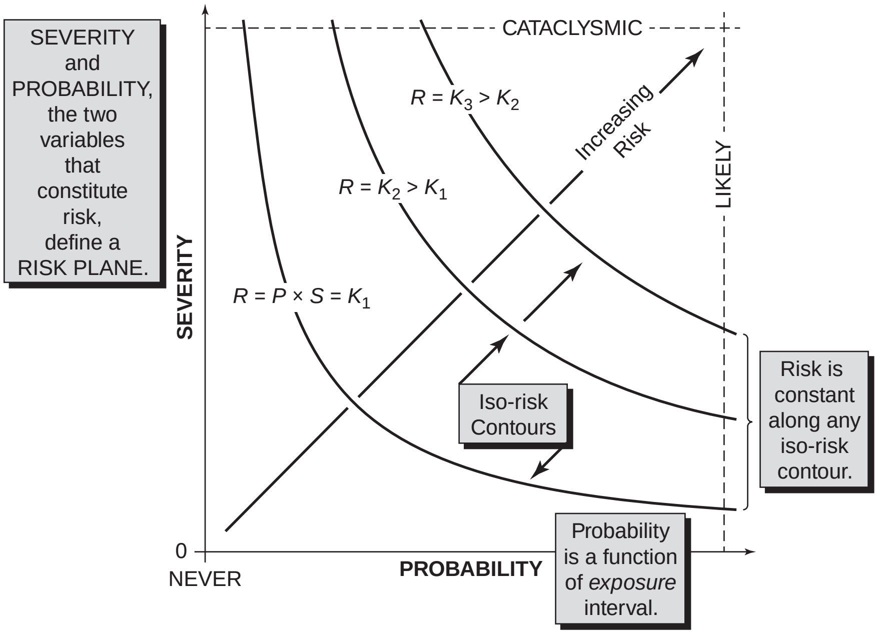


Figure 23.2 Iso-risk contours (After NIOSH).

23.3 RISK ASSESSMENT MATRIX

23.3.1 Description

A *risk assessment matrix* is a tool that is used to conduct subjective risk assessments for use in hazard analysis. The definition of risk and the principle of the iso-risk contour are the basis for this technique.

The risk posed by a given hazard to an exposed resource can be expressed in terms of an expectation of loss, the combined severity and probability of loss, or the long-term rate of loss. *Risk* (*R*) is the product of *severity* (*S*) and *probability* (*P*) (loss events per unit time or activity). Note that the probability component of risk must be attached to an exposure time interval.

The severity and probability dimensions of risk define a risk plane. As shown in Fig. 23.2, *iso-risk contours* depict constant risk within the plane. The concept of the iso-risk contour is useful to provide guides, conventions, and acceptance limits for risk assessments (see Fig. 23.3).

Risk should be evaluated for the worst credible case, not worst conceivable case, or conditions. Failure to assume credible (even if conceivable is substituted) cases may

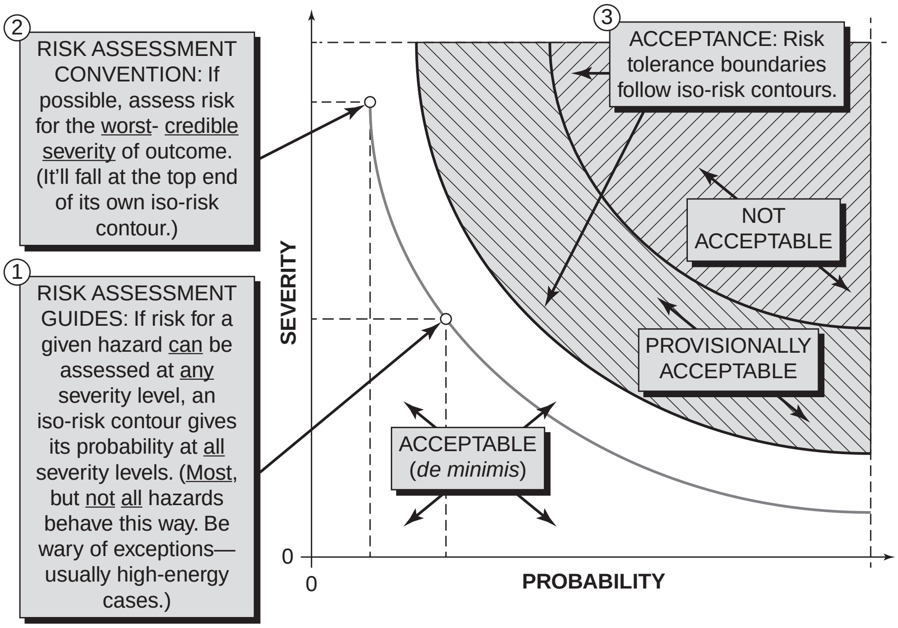


Figure 23.3 Iso-risk contours usage (After NIOSH).

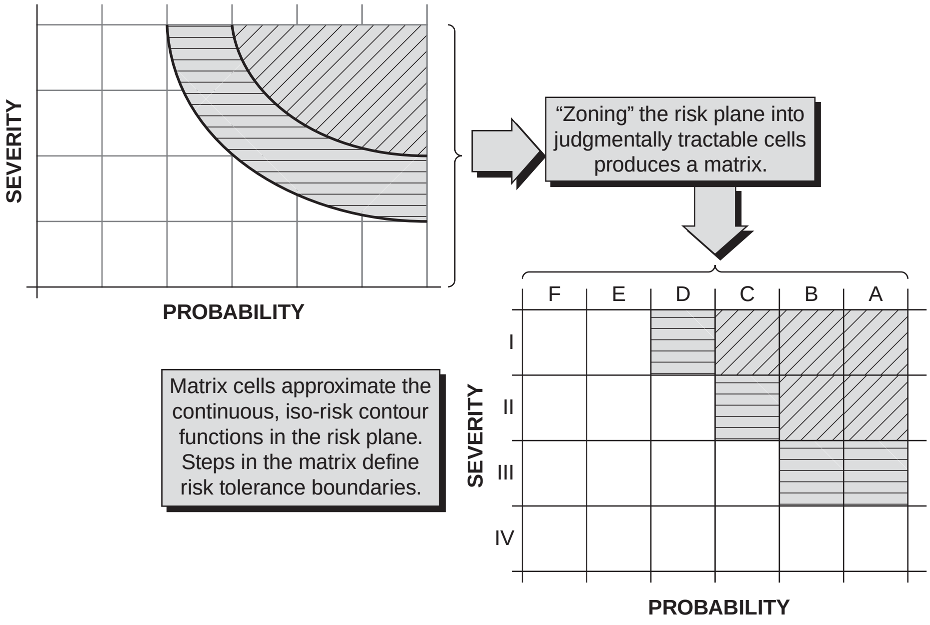
result in an optimistic analysis, which is a non-viable analysis.

The risk assessment matrix is typically used during the design and development phase, but may also be used in the conceptual trade studies phase. This technique is used as a predetermined guide or criterion to evaluate identified hazards. These risks are expressed in terms of severity and probability. Use of this tool allows an organization or firm to institute and standardize its approach for performing hazard analyses.

23.3.2 Procedures

The procedures for developing a risk assessment matrix are as follows:

1. Categorize and scale the subjective probability levels for all targets or resources, such as frequent, probable, occasional, remote, improbable, and impossible. Note that a *target* or *resource* is defined as the "what" that is at risk. One typical breakout of targets or resources is personnel, equipment, downtime, product loss, and environmental effects.
2. Categorize and scale the subjective severity levels for each target or resource, such as catastrophic, critical, marginal, and negligible.
3. Create a matrix of consequence severity versus the probability of the mishap (the event capable of producing loss). Approximate the continuous, iso-risk contour functions in the risk plane with matrix cells (see Fig. 23.4). These matrix cells fix the limits of risk tolerance zones. Note that management—not the analyst—establishes and approves the risk tolerance boundaries. Management will consider social, legal, and financial impacts when setting risk tolerance
4. The following hints are helpful for creating the matrix:
   (a) Increase adjacent probability steps by orders of magnitude. The lowest step, "impossible," is an exception (see Fig. 23.5).
   (b) Avoid creating too many matrix cells. Because the assessment is subjective, too many steps add confusion with no additional resolution (see Fig. 23.6).
   (c) Avoid discontinuities in establishing the risk zones, that is, make sure every one-step path does not pass through more than one zone (see Fig. 23.7).
   (d) Establish only as many risk zones as there are desired categories of resolution to risk issues, that is, (1) unacceptable, (2) accepted by waiver, and (3) routinely accepted (see Fig. 23.8).
   (e) Link the risk matrix to a stated *exposure period.* When evaluating exposures, a consistent exposure interval must be selected, otherwise risk acceptance will be variable. An event for which the probability of occurrence is judged as remote during an exposure period of 3 months may be judged as frequent if the exposure period is extended to 30 years. For occupational applications, the exposure period is typically 25 years. All stakeholders (management or the client) who participate in establishing the risk acceptance matrix must be informed of any changes to the



"Zoning" the risk plane into judgmentally tractable cells produces a matrix.

Matrix cells approximate the continuous, iso-risk contour functions in the risk plane. Steps in the matrix define risk tolerance boundaries.

Figure 23.4 Risk contours to risk matrix transformation (After NIOSH).

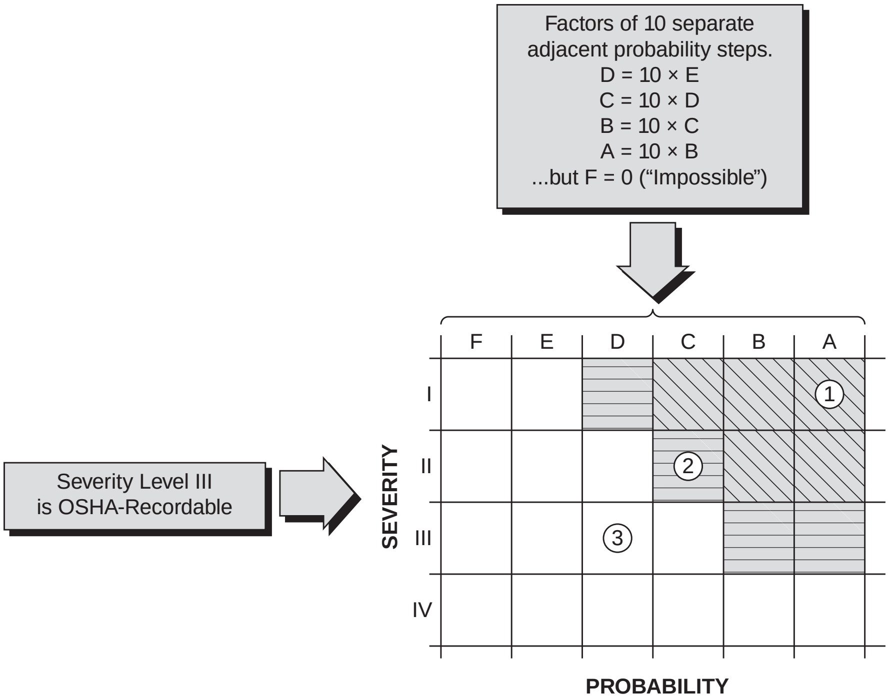


Figure 23.5 Useful conventions (After NIOSH).

exposure interval for which the matrix was calibrated.

5. Calibrate the risk matrix by selecting a cell and attaching a practical hazard scenario to it. The scenario should be familiar to potential analysts or characterize a tolerable perceivable threat. Assign its risk to the highest level severity cell just inside the acceptable risk zone. This calibration point should be used as a *benchmark* to aid in evaluating other, less familiar risk.
6. An example for developing a risk assessment matrix is given in Example 23.1. Examples of strategies to manage harmful energy flow are listed in Table 23.1

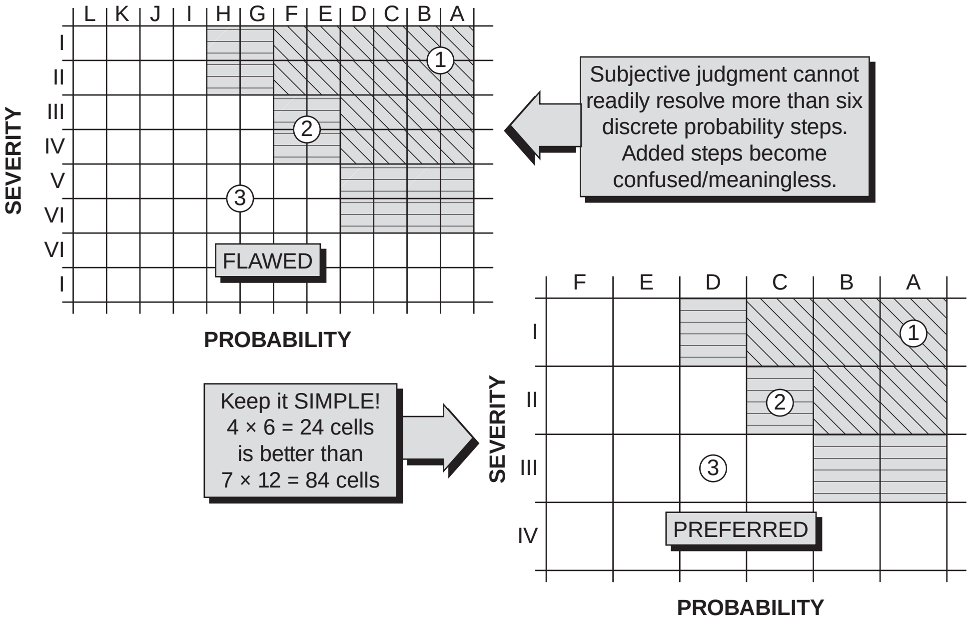
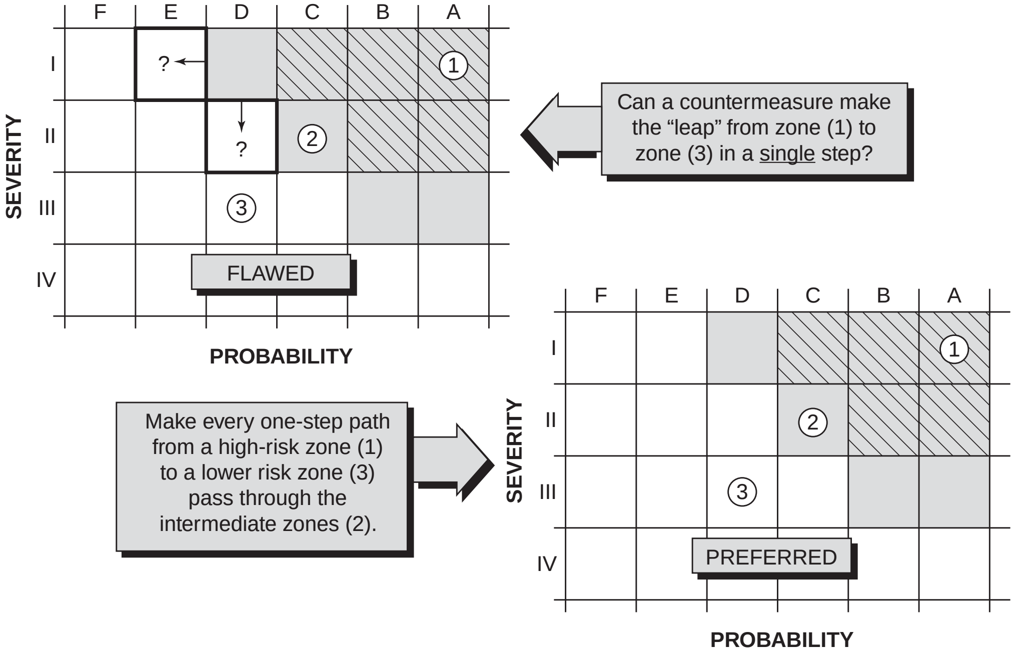


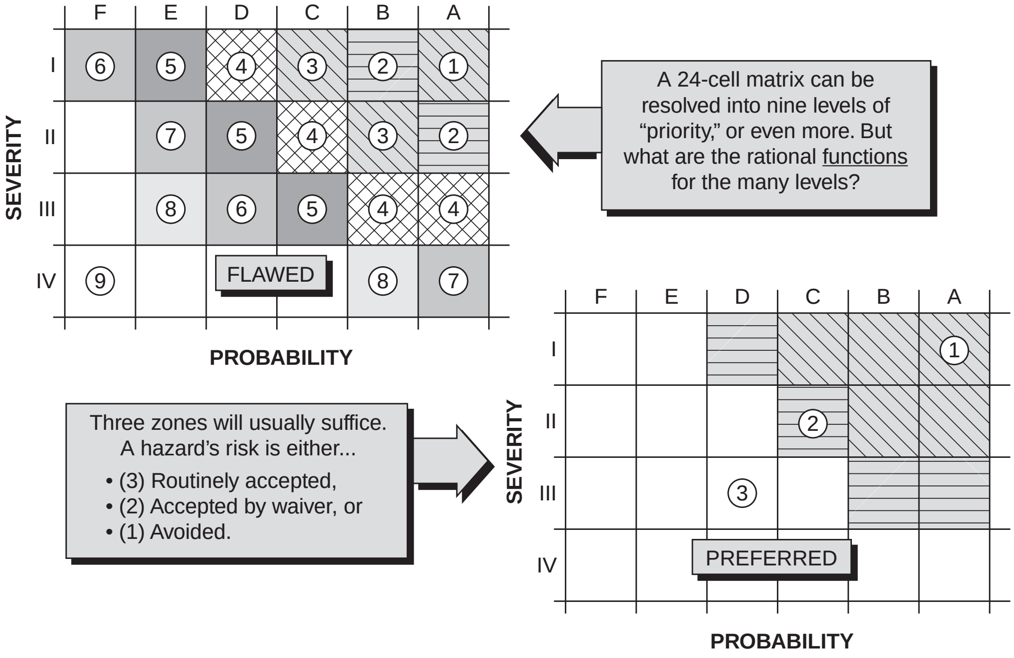
Figure 23.6 Creation of matrix cells (After NIOSH).



Can a countermeasure make the "leap" from zone (1) to zone (3) in a <u>single</u> step?

Make every one-step path from a high-risk zone (1) to a lower risk zone (3) pass through the intermediate zones (2).

Figure 23.7 Avoidance of discontinuities (After NIOSH).



A 24-cell matrix can be resolved into nine levels of "priority," or even more. But what are the rational <u>functions</u> for the many levels?

Three zones will usually suffice. A hazard's risk is either...

- (3) Routinely accepted,
- (2) Accepted by waiver, or
- (1) Avoided.

Figure 23.8 Number of risk zones (After NIOSH).

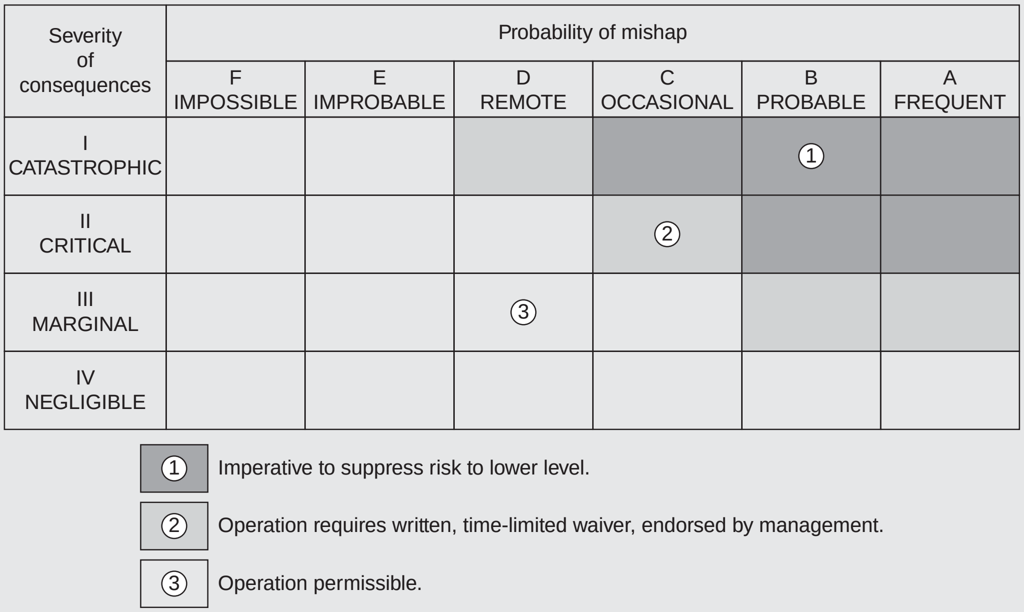
Table 23.1 Examples of strategies to manage harmful energy flow

Strategy	Examples
Eliminate energy concentrations	• control/limit floor loading • disconnect/remove energy source from system • remove combustibles from welding site • change to nonflammable solvent
Limit quantity and/or level of energy	• store heavy loads on ground floor • lower dam height • reduce system design voltage/operating pressure • use small (er) electrical capacitors/pressure accumulator • reduce/control vehicle speed • monitor/limit radiation exposure • substitute less energetic chemicals
Prevent energy release	• heavy-wall pipe or vessels • interlocks • tagout—lockouts • double-walled tankers • wheel chocks
Modify rate of energy release	• flow restrictors in discharge lines • resistors in discharge circuits • fuses/cucuit interrupters
Separate energy from target in time and/or space	• evacuate explosive test areas • impose explosives quantity—distance rules • install traffic signals • use yellow no-passing lines on highways • control hazardous operations remotely
Isolate by imposing a barrier	• guard rails • toe boards • hard hats • face shields • machine tool guards • dikes • grounded appliance frames/housing • safety goggles
Modify target contact surface or basic structure	• cushioned dashboard • fluted stacks • padded rocket motor test cell interior • Whipple plate meteorite shielding • breakaway highway sign supports • foamed runways
Strengthen potential target	• select superior material • substitute forged part for cast part • "harden" control room bunker • cross-brace transmission line tower
Control improper energy input	• use coded keyed electrical connectors • use match-threaded piping connectors • use back flow preventors

Source: NIOSH.

EXAMPLE 23.1 DEVELOPMENT OF A RISK ASSESSMENT MATRIX

Figure 23.9 shows a typical risk assessment matrix. Interpret the severity and probability steps for this matrix.



Severity of consequences	Probability of mishap					
	F IMPOSSIBLE	E IMPROBABLE	D REMOTE	C OCCASIONAL	B PROBABLE	A FREQUENT
I CATASTROPHIC			░	▓	▓ ①	▓
II CRITICAL				░ ②	▓	▓
III MARGINAL			③		░	░
IV NEGLIGIBLE						

▓ ① Imperative to suppress risk to lower level.

░ ② Operation requires written, time-limited waiver, endorsed by management.

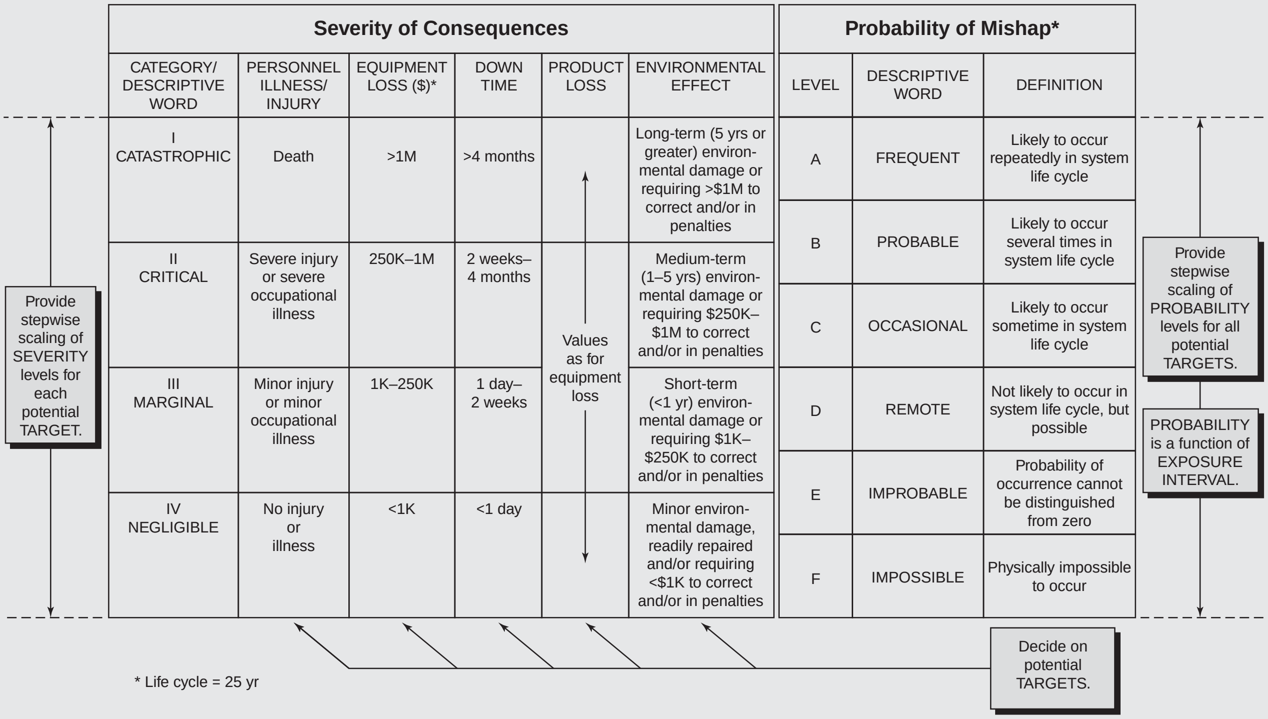
③ Operation permissible.

NOTE: Personnel must not be exposed to hazards in Risk Zones 1 and 2.

Figure 23.9 Typical risk assessment matrix (life cycle = 25 yr) (After NIOSH).

Solution:

Figure 23.10 shows sample interpretations of the severity and probability steps for this matrix.



Severity of Consequences					
CATEGORY/ DESCRIPTIVE WORD	PERSONNEL ILLNESS/ INJURY	EQUIPMENT LOSS ($)*	DOWN TIME	PRODUCT LOSS	ENVIRONMENTAL EFFECT
I CATASTROPHIC	Death	>1M	>4 months	Values as for equipment loss	Long-term (5 yrs or greater) environmental damage or requiring >$1M to correct and/or in penalties
II CRITICAL	Severe injury or severe occupational illness	250K–1M	2 weeks–4 months		Medium-term (1–5 yrs) environmental damage or requiring $250K–$1M to correct and/or in penalties
III MARGINAL	Minor injury or minor occupational illness	1K–250K	1 day–2 weeks		Short-term (<1 yr) environmental damage or requiring $1K–$250K to correct and/or in penalties
IV NEGLIGIBLE	No injury or illness	<1K	<1 day		Minor environmental damage, readily repaired and/or requiring <$1K to correct and/or in penalties

Probability of Mishap*		
LEVEL	DESCRIPTIVE WORD	DEFINITION
A	FREQUENT	Likely to occur repeatedly in system life cycle
B	PROBABLE	Likely to occur several times in system life cycle
C	OCCASIONAL	Likely to occur sometime in system life cycle
D	REMOTE	Not likely to occur in system life cycle, but possible
E	IMPROBABLE	Probability of occurrence cannot be distinguished from zero
F	IMPOSSIBLE	Physically impossible to occur

* Life cycle = 25 yr

Figure 23.10 Severity and probability interpretations (After NIOSH).

23.3.3 Advantages and Limitations of the Risk Matrix

The risk assessment matrix has certain advantages:

1. The risk matrix provides a useful guide for prudent engineering.
2. The risk matrix provides a standard tool of treating the relationship between severity and probability in assessing risk for a hazard.
3. Subjective risk assessment avoids unknowingly accepting intolerable and senseless risk, allows operating decisions to be made, and improves resource distribution for mitigation of loss resources.

The risk assessment matrix has the following limitations:

1. The risk assessment matrix can only be used if hazards are already identified; this tool does not assist the analyst in identifying hazards.
2. Without data, this method is subjective and is a comparative analysis only.

23.4 FAILURE MODES, EFFECTS, AND CRITICALITY ANALYSIS

23.4.1 Description

A *failure modes and effects analysis* (FMEA) is a forward logic (bottom-up), tabular technique that explores the ways or modes in which each system element can fail. It also assesses the consequences of each of these failures. In its practical application, its use is often guided by top-down screening to establish the limit of analytical resolution. A *failure modes*, *effects*, *and criticality analysis* (FMECA) also addresses the criticality or risk of individual failures. Countermeasures can be defined for each failure mode, and consequent reductions in risk can be evaluated. FMEA and FMECA are useful tools for cost/benefit studies to implement effective risk mitigation and countermeasures.

It is important to remember that the analytical techniques discussed in this chapter complement (rather than supplant) each other. It has long been sought, but there is no “Swiss army knife” technique that answers all questions and is suitable for all situations.

23.4.2 Application

An FMEA can call attention to system vulnerability to failures of individual components. Single-point failures can be identified. This tool can be used to provide reassurance that the cause, effect, and associated risk of component failures have been appropriately addressed. These tools are applicable within systems or at the system/subsystem interfaces and can be applied at the system, subsystem, component, or part levels.

These failure mode analyses are typically performed during the design and development phase. The vulnerable points identified in the analyses can aid management in making decisions to allocate resources in order to reduce vulnerability.

23.4.3 Procedures

Procedures for preparing and performing FMECAs are presented next. Procedures for preparing a FMEA are the same with the exception of steps 8 through 12 being omitted:

1. Define the scope and boundaries of the system to be assessed. Gather pertinent information relating to the system, such as requirement specifications, descriptions, drawings, components, and parts lists. Establish the mission phases to be considered in the analysis.
2. Partition and categorize the system into convenient and logical elements to be analyzed. These system elements include subsystems, assemblies, subassemblies, components, and piece parts.
3. Develop a numerical coding system that corresponds to the system breakdown (see Fig. 23.11)
4. Identify resources of value to be protected, such as personnel, facilities, equipment, productivity, mission or test objectives, and environment. These resources are potential targets.
5. Identify and observe the levels of acceptable risk that have been predetermined and approved by management or the client. These limits may be the risk matrix boundaries defined in a risk assessment matrix.
6. By answering the following questions, the scope and resources required to perform a classic FMEA can be reduced, without loss of benefit:
   - Will failure of the *system* render an unacceptable or unwanted loss?
     If the answer is no, the analysis is complete. Document the results. (This has the additional benefit of providing visibility of non-value-added systems, or it may correct incomplete criteria used for the FMEA.) If the answer is yes, ask the following question for each subsystem identified in step 2:
   - Will failure of this *subsystem* render an unacceptable or unwanted loss?
     If the answer for each subsystem is no, the analysis is complete. Document the results. If the answer is yes for any subsystem, ask the following question for each assembly of those subsystems identified in step 2:

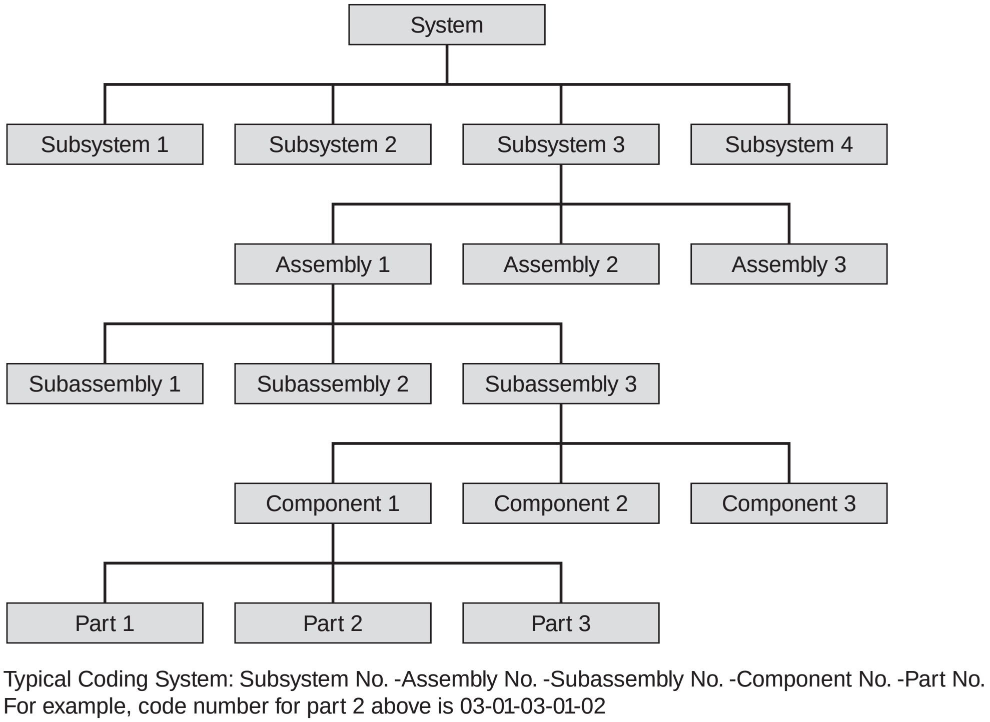


Figure 23.11 Example of system breakdown and numerical coding (After NIOSH).

- Will failure of this *assembly* render an unacceptable or unwanted loss?
  If the answer for each assembly is no, the analysis is complete. Document the results. If the answer is yes for any assembly, ask the following question for each component of those assemblies identified in step 2:
- Will failure of this *subassembly* render an unacceptable or unwanted loss?
  If the answer for each subassembly is no, the analysis is complete. Document the results. If the answer is yes for any subassembly, ask the following question for each component of those subassemblies identified in step 2:
- Will failure of this *component* render an unacceptable or unwanted loss?
  If the answer for each component is no, the analysis is complete. Document the results. If the answer is yes for any component, ask the following question for each part of those components identified in step 2:
- Will failure of this part render an unacceptable or unwanted loss?

7. For each element (system, subsystem, assembly, subassembly, component, or part) for which failure would render an unacceptable or unwanted loss, ask and answer the following questions:
   - What are the failure modes for this element?
   - What are the effects (or consequence) of each failure mode on each target?
8. Assess worst credible case (not the worst conceivable case) severity and probability for each failure mode, effect, and target combination.
9. Assess the risk of each failure mode using a risk assessment matrix. The matrix should be consistent with the established probability interval and force or fleet size for this assessment.
10. Categorize each identified risk as acceptable or unacceptable.
11. If the risk is unacceptable, then develop countermeasures to mitigate it.
12. Then reevaluate the risk with the new countermeasure installed.
13. If countermeasures are developed, determine if they introduce new hazards or intolerable or diminished system performance. If added hazards or degraded performance are unacceptable, develop new countermeasures and reevaluate the risk.
14. Document your completed analysis on an FMEA or FMECA worksheet. The contents and formats of these worksheets vary among organizations. Countermeasures may or may not be listed.

Figure 23.12 represents a flowchart for FMEA or FMECA. Table 23.2 presents a sample FMEA worksheet

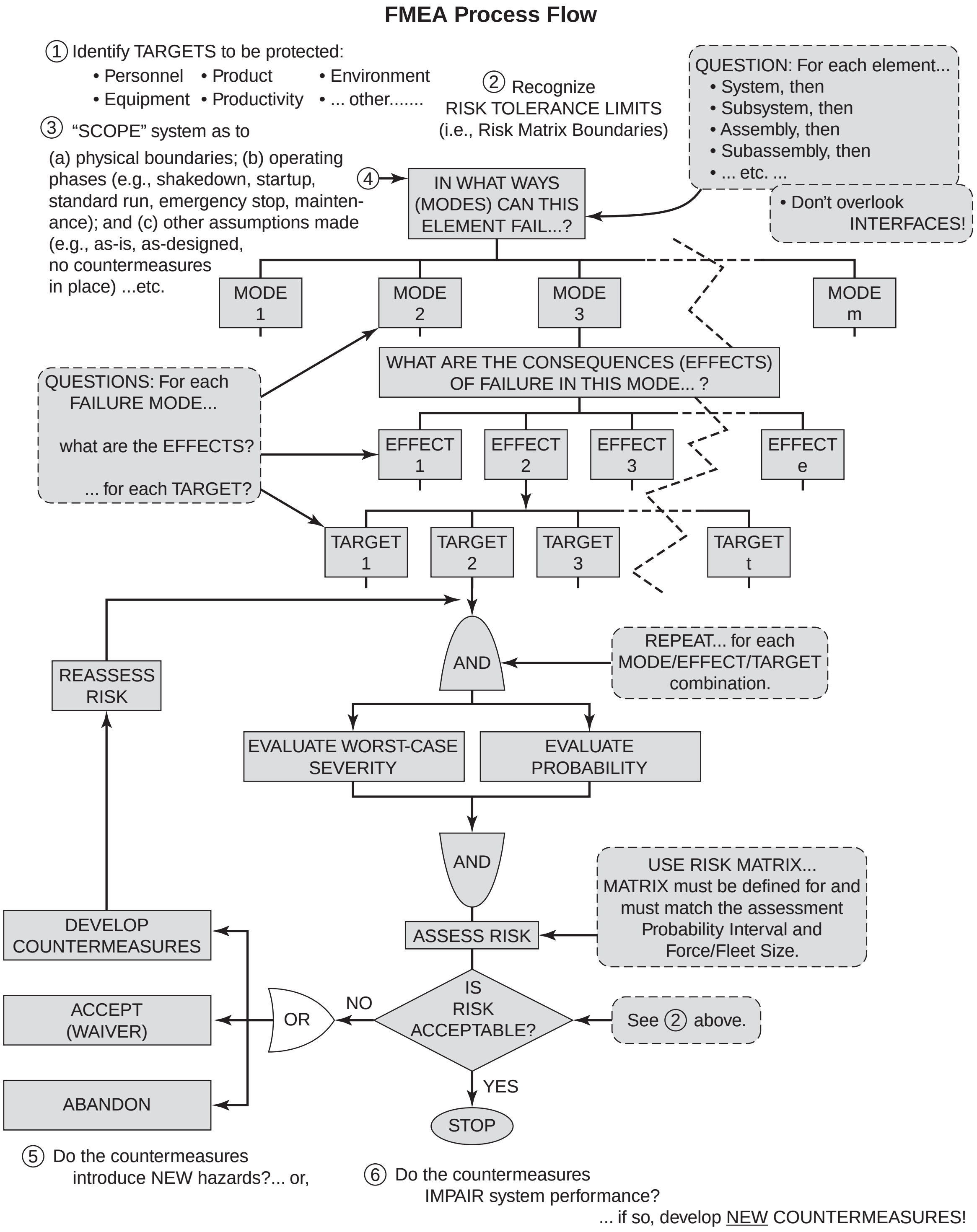


Figure 23.12 Failure modes, effects, (and criticality) analysis process flowchart (After NIOSH).

Table 23.2 Typical failure modes, effects, and criticality analysis worksheet

<table>
<tr><td colspan="3">FMEA NO: ____________
PROJECT NO: ____________
SUBSYSTEM NO: ____________
SYSTEM NO: ____________
PROB. INTERVAL____________</td><td colspan="3">FAILURE MODES, EFFECTS, AND CRITICALITY ANALYSIS WORKSHEET</td><td colspan="5">SHEET ____ OF ____
DATE ____________
PREPARED BY: ____________
REVIEWED BY: ____________
APPROVED BY: ____________</td></tr>
<tr><td colspan="11">TARGET/RESOURCE CODE: P-PERSONNEL/E-EQUIPMENT/T-DOWNTIME/R-PRODUCT/D-DATA/V-ENVIRONMENT</td></tr>
<tr><td rowspan="2">Id. No.</td><td rowspan="2">Item/ Functional Identification</td><td rowspan="2">Failure Mode</td><td rowspan="2">Failure Cause</td><td rowspan="2">Failure Event</td><td rowspan="2">Target</td><td colspan="3">Risk Assessment</td><td rowspan="2">Action Required/ Comments</td></tr>
<tr><td>Sev</td><td>Prob</td><td>Risk Code</td></tr>
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</table>

Source: NIOSH.

EXAMPLE 23.2 ASSESSMENT OF RISK

Assess the risk for an automated mountain climbing rig.

Solution:

A sample FMECA is illustrated in Fig. 23.13. The system being assessed is an automated mountain climbing rig. Table 23.3 illustrates the breakdown and coding of the system into subsystem, assembly, and subassembly elements. An FMECA worksheet for the control subsystem is presented in Table 23.4.

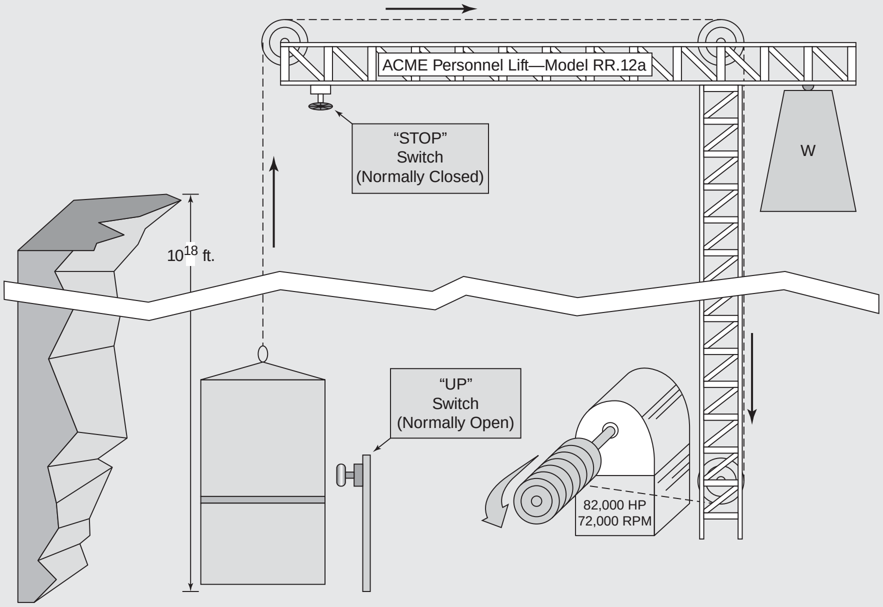

Figure 23.13 System for Example 23.2 (After NIOSH).

Table 23.3 System breakdown and coding for Example 23.2

Subsystem	Assembly	Subassembly
Hoist (A)	Motor (A-01)	Windings (A-01-a) Inboard bearing (A-01-b) Outboard bearing (A-01-c) Rotor (A-01-d) Stator (A-01-e) Frame (A-01-f) Mounting plate (A-01-g) Wiring terminals (A-01-h)
	Drum (A-02)	
External power source (B)		
Cage (C)	Frame (C-01) Lifting lug (C-02)	
Cabling (D)	Cable (D-01) Hook(D-02) Pulleys (D-03)	
Controls (E)	Electrical (E-01)	START Switch (E-01-a) FULL UP LIMIT Switch (E-01-b) Wiring (E-01-c)
	Operator (E-02)	

Source: NIOSH.

Table 23.4 Failure modes, effects, and criticality analysis for Example 23.2

FMEA NO: ____________ PROJECT NO: ____________ SUBSYSTEM NO: Controls SYSTEM NO: Mountain Climbing Rig PROB. INTERVAL: 30 years	**FAILURE MODES, EFFECTS, AND CRITICALITY ANALYSIS WORKSHEET**	SHEET ____ OF ____ DATE ____________ PREPARED BY: ____________ REVIEWED BY: ____________ APPROVED BY: ____________
TARGET/RESOURCE CODE: P-PERSONNEL/E-EQUIPMENT/T-DOWNTIME/R-PRODUCTS/D-DATA/V-ENVIRONMENT		

Id. No.	Item/ Functional Identification	Failure Mode	Failure Cause	Failure Event	Target	Risk Assessment			Action Required/ Comments
						Sev	Prob	Risk Code	
E-01-a	Start Switch	Switch fails closed.	Mechanical failure or corrosion.	Cage will not move.	P E T	IV IV IV	C C C	3 3 3	
E-01-b	Full Up Switch	Switch fails open.	Mechanical failure or corrosion.	Cage does not stop.	P	II	A	1	
E-02	Wiring	Cut, Disconnected.	Varmint invasion, faulty assembly	No response a switch. Start switch fails open. Stop switch fails closed. Cage stays in safe position.	P E T	IV IV IV	D D D	3 3 3	

Source: NIOSH.

23.4.4 Advantages and Limitations

Performing FMEA and FMCEA has the following advantages:

1. They provide an exhaustive, thorough mechanism to identify potential single-point failures and their consequences. An FMECA provides risk assessments of these failures.
2. Results can be used to optimize reliability, optimize designs, incorporate "fail-safe" features into the system design, obtain satisfactory operation using equipment of "low reliability," and guide in component and manufacturer selection.
3. An FMECA can be a very thorough analysis suitable for prioritizing resources to higher risk areas if it can be performed early enough in the design phase.

The following limitations are imposed when performing FMEAs and FMECAs:

1. The analyses are costly in terms of man-hour resources, especially when performed at the parts-count level within large, complex systems.
2. Probabilities or the consequences of system failures induced by coexisting, multiple-element faults or failures within the system are not addressed or evaluated.
3. Although systematic and guidelines/check-sheets are available for assistance, no check methodology exists to evaluate the degree of completeness of the analyses.
4. These analyses depend heavily on the ability and expertise of the analyst for finding all necessary modes.
5. Human error and hostile environments frequently are overlooked.
6. Failure probability data are often difficult to obtain for an FMECA.
7. If too much emphasis is placed on identifying and eliminating single-point failures, then focus on more severe system threats (posed by coexisting failures/faults) may be overlooked.

23.5 ENGINEERING AND DESIGN FOR SAFE CONSTRUCTION

23.5.1 Construction Failures

Tragic construction failures, especially those that involve fatalities and substantial economic losses, often receive wide media coverage and bring public outcry for preventive actions. Such incidences have frequently led to federal investigations, and controversy and debate within the civil engineering and construction community, and have prompted legislative initiatives. The following are a few of the most infamous examples of construction failures, which have caused great losses and generated significant public and professional interest.

23.5.1.1 L'Ambiance Plaza Condominium Collapse On April 23, 1987, L'Ambiance Plaza, a 16-story apartment building in Bridgeport, CT, collapsed during construction. The lift slab method was being used in this project to erect the structure. The collapse resulted in 28 fatalities and 18 injuries, which made L'Ambiance the most catastrophic construction accident of the 1980s. An investigation was performed by the National Bureau of Standards (NBS; now called NIST—the National Institute for Standards and Technology) to determine the most probable cause of this collapse. This investigation concluded that the most probable technical cause of the collapse was the failure of some lifting assembly components, designed and erected by the lift slab contractor and its subcontractor.

Several subsequent forensic investigations were also conducted on this accident identifying design deficiencies and suggesting various alternative failure mechanisms. Many experts involved in this work believe that the fragmentation of responsibility, the delegation of structural design to the contractors, and poor communication between the parties were the underlying reasons that led to the collapse of L'Ambiance Plaza.

23.5.1.2 Willow Island Cooling Tower Collapse In April 1978, the hyperbolic cooling tower of the Pleasants Power Station near Willow Island, WV, partially collapsed during construction, resulting in the death of 51 construction workers. The patented jump from scaffolding, which was supported by the partially completed concrete shell, collapsed as the connecting anchors pulled out of the partially matured concrete, 1 day after the pouring. This accident brought to the nation's attention the potential hazards of temporary structures and the potential limitations of construction materials.

23.5.1.3 Kansas City Hyatt Regency Walkway Collapse Although this failure did not occur during construction, an important lesson may be learned from the errors that led to it. On July 17, 1981, the second and fourth floor hanging walkways spanning across the atrium of the Hyatt Regency Hotel in Kansas City, MO, collapsed and fell to the ground floor during a social event. Described as "the most devastating structural collapse ever to take place in the United States," this failure took the lives of 114 people and injured 200 more. According to NBS, which performed an investigation on the failure, the cause of the collapse was inadequate connection details between the steel box beams and the hanging rods.

It was found that without the consent of the engineer of record, these critical details were altered by the steel fabricators in such a way that the main source of support for the second floor walkway was transferred from the roof to the fourth floor walkway, creating an overload situation. This flawed revision was not noticed by the engineer of record, who checked the shop drawings only for conformance to the original design. This failure raised the question of responsibility for the shop drawings and the design of structural details, causing the engineering profession to start reconsidering the traditional practices with regard to structural details, particularly those that do not conform to the American Institute for Steel Construction (AISC) standards.

23.5.2 Causes of Construction Failures

Prevention of construction failures, first and foremost, requires an understanding of the causes. Many efforts have been directed toward this end by engineers, forensic scientists, the legal community, and legislators. In the early 1980s, this problem was addressed by the Investigations Subcommittee of the House Committee on Science and Technology through a study aimed at determining the underlying causes of the construction failures and the possible related shortcomings in technology and education. This investigation was based on extensive interviews with construction engineers focusing on the understanding of the actions that take place during construction. The findings of this study were summarized as follows:

1. The organization of a construction project is unique since it brings together many parties for a limited period of time.
2. There is little evidence of the presence of consistent methods of maintaining overall project quality.
3. The structural engineer in charge of design does not perform all of the design work; important details are implemented by steel fabricators.
4. The structural engineer, who is the logical person to inspect a structure, is discouraged from doing so.
5. Recent engineering graduates are well trained in mathematics and computers, but are relatively unsophisticated about the behavior of structures.
6. Long-span structures require careful quality control, and there is a need for new approaches emphasizing dynamic behavior analysis.

Eldukair and Ayyub (1991) conducted a comprehensive study of the causes of construction/structural failures. They reviewed 604 cases that occurred in the United States between 1975 and 1986. They discovered the following:

1. Nearly 57% of failures resulted in collapse and 4% resulted in other unsafe conditions.
2. A critical percentage (44%) of the structural failures occurred during the construction phase.
3. Errors in design and construction phases of the projects were the most predominant sources of failures. Design errors occurred in 51% of the total failure cases; 57% of the total failure cases recorded deficiencies in construction procedures.
4. Structural designers were responsible for the errors in 40% of the cases. Contractor's site staff was accountable for errors in 60% of the total failure cases.
5. Deficiencies in the construction procedures primarily involved inadequate construction methods, ineffective evaluation of laws and safety regulations, and inadequate planning and supervision.
6. Errors in management practices had a tremendous effect on the performance of project activities, the schedule, and safety. These included deficiencies in work responsibilities, deficiencies in the communication process, and insufficient work cooperation. Errors in defining work responsibilities were dominant (30%) as part of the deficiencies in management practices.
7. A great majority of the cases (86%) indicated that reinforced concrete elements were involved in the failures, and the highest number of failed members (34%) was slabs and plates.
8. The primary causes of most failures were poor erection procedures and inadequate connection elements leading to inadequate load behavior, followed by unclear contract information, and contravention of information. A small percentage of failures were attributed to unforeseen events.
9. Secondary causes of failures were led by environmental effects (mostly poor weather conditions), followed by poor supervision and control, poor communications, poor material and equipment usage, poor workmanship, and other minor technical and nontechnical factors.

The study found that the cases examined caused 416 total deaths and 2515 total injuries. The total direct costs of damage associated with the studied structural and construction failures in 1986 were estimated at USD 3.5 billion (6.3 billion in 2012 USD). The hidden indirect costs were excluded from the estimate because of the difficulty of determining.

23.5.3 Classification of Causal Factors

The above-mentioned studies and others have shed considerable light on some of the common causes of construction failures. Based on these considerations, it is possible to summarize the causal factors governing construction failures into five major classifications:

1. Natural disasters
2. Engineering and design considerations
3. Material properties and performance

4. Construction methods and procedures
5. Contract administration and project management

23.5.3.1 Natural Disasters Natural phenomena such as earthquakes, hurricanes, violent storms, and tornados often cause enormous destruction to buildings and other structures during construction or service, leading to loss of life and property damage of significant magnitude. Although most structures can be designed and constructed to withstand such overwhelming forces of nature, some damage is often unavoidable (hence the term “act of God”). However, a natural disaster may also reveal a latent deficiency in design, construction techniques, or code requirements.

An example from 1992 is Hurricane Andrew, which resulted in the costliest disaster in US history. This violent storm with gusty winds exceeding 150 mph (240 km/h) devastated many buildings, while revealing critical deficiencies in construction practices in the South Florida area. This fact became particularly evident in residential construction, where roof failures caused by inadequate connections resulted in the total destruction of many wooden frame houses. Experts pointed to cost-cutting measures by contractors as a contributing factor for the great damage caused by Hurricane Andrew.

Such disasters may expose critical deficiencies in structures, raising concerns for public safety. The engineering community should recognize the threat of natural disasters in geographic areas susceptible to them and propose corrective measures through code revisions to eliminate or mitigate the potential failures.

23.5.3.2 Engineering and Design Considerations Engineering and design for construction involve complex processes, which blend technical expertise with skillful project organization and management. In addition, with respect to design and construction knowledge, an understanding of government regulations and building codes is necessary, along with effective communications with various groups involved in the construction process.

An important prerequisite to safe design and engineering is the consideration of *constructability*. The constructability task force of the Construction Industry Institute (CII) has defined constructability as the optimum integration of construction knowledge and experience in planning, design, procurement, and field operations to achieve overall project objectives. This approach will lead to greater productivity, lower costs, and improved quality and safety.

According to CII, the design process must integrate construction needs from the conceptual stage to final delivery of contract documents. Constructability analysis of the design can identify conflicts and shortcomings that may complicate construction. It is therefore beneficial to integrate such an analysis into the project execution plans starting with the preliminary design stage. Meaningful input from the constructors, CMs, or constructability consultants can be very beneficial in the early stages of a project. Several critical decisions are made during the schematic design stage that will determine many subsequent construction activities. Factors such as availability of raw materials and skilled labor, costs, and the overall project schedule can be incorporated into these decisions. It might be assumed that the project schedule is construction sensitive, and the general construction methods might consider the impact of seasonal weather conditions, site characteristics, drainage, and soil conditions.

Constructability reviews can be applied in more depth during the design development and construction document preparation phases by incorporating the following objectives and principles:

- Design configurations should enable efficient construction and eliminate spatial constraints, which complicate proper assembly, especially for large structural elements.
- Sequence of design should follow construction logic and take into account feasible construction time. For example, builders need to realize that certain slabs that are designed for service loads may not support the construction loads.
- Complicated construction details should be minimized, especially if they require extensive on-site assembly. Standardization and preassembled design should be maximized.
- Construction task interdependencies that would bring many trades together should be minimized.
- Flexibility for field adjustments during implementation should be allowed, and tight tolerances should be eliminated. An example of this would be certain structural members that have adequate depth to support the service loads not allowing for the necessary connections.
- The site layout should consider accessibility, and the contract documents should contain provisions for safe access for workers and storage materials and equipment.
- The construction team should provide input and advice on access needs, major equipment placement, and storage requirements.
- Potential conflicts between underground work and heavy equipment movement should be eliminated.
- The design and erection sequence should consider the stability of incomplete structural frames during construction.
- If the project needs to be completed during adverse weather conditions, design procurement should facilitate an early enclosure of the structure to minimize

the exposure to harsh weather. Specifications should promote the use of weather-compatible materials and limit field work by using preassembled modules.

These principles imply that the consideration of constructability issues during design can eliminate difficulties in execution of construction tasks and avoid unsafe conditions on the jobsite. Also, all document conflicts and discrepancies of information caused by the possible lack of coordination between different sections of the construction documents should be eliminated. Such conflicts and gaps may go unnoticed and create constructability problems (such as incompatibilities between structural members and mechanical elements) when the project has reached an advanced stage.

23.5.3.3 Material Properties and Performance Many structural/construction failures are caused by material deficiencies. Materials, in and of themselves, may not be defective; however, their inappropriate selection or utilization in the construction process may lead to failures. Understanding the nature and properties of the materials, their structural behavior, and their strength and durability performance in the service environment is vital to the successful performance of the structure, and the avoidance of failures. For instance, freeze/thaw effects and shrinkage problems in concrete are well known. However, strength development (maturity) is somewhat less well understood. The collapse of the Willow Island cooling tower mentioned earlier was primarily attributable to incomplete curing of the concrete, resulting in insufficient material strength at the time of formwork removal.

Building products have been continually evolving and their changing attributes may present a challenge to the designers. New materials, as well as technological advancements in traditional materials, create the possibility of not fully understanding their potential limitations. For instance, the performance of plastics, which are gaining acceptance in the construction industry, has not been fully tested for all service conditions. Improved properties of steel, concrete, and wood can enhance structural performance, but may also present some problems. For instance, higher strength steel facilitates greater space availability, but it may offer reduced ductility.

The effects of environmental conditions on materials are another important concern. For example, metals under certain conditions are subject to corrosion, despite conventionally adequate safeguards and quality control. Galvanic action between certain metals is a common cause of corrosion. Metal fatigue may create cracks that weaken the material and facilitate chemical reactions promoting corrosion. Contrary to common belief, concrete does not always provide adequate protection to reinforcing steel. Moisture can reach the steel through the cracks (and sometimes pores) in concrete causing corrosion, which can lead to total deterioration. Chlorides also promote corrosion in steel. They can come from deicing salts or from chloride additives used in high-bond mortar.

Incompatibility of materials that come in contact in the constructed form may also cause deterioration. The different materials that form the composite members of a structural member may have incompatible tolerances, strengths, and ductilities. These differences can produce unanticipated stress redistributions. Incompatibility may also lead to differential movements resulting from temperature and moisture gradients. Different materials have varying coefficients of expansion. For instance, brick, limestone, and concrete have uniquely different coefficients, and under the same temperature gradient, they may expand and contract at different rates, leading to cracks.

Some of the most important decisions regarding design and constructability reviews involve material selection. Engineers and architects must specify the materials, approve substitutions (as may be required by value engineering analysis), and test the quality of the material during construction. These tasks must take into account all exposures, compatibility, and the limits of performance. In the selection of materials, manufacturer's design and performance data must be scrutinized, and qualitative information should be clearly documented.

23.5.3.4 Construction Methods and Procedures In present practice, the construction methods and procedures by which a facility or project is erected are considered to be the responsibility of the contractor. In a great majority of construction projects, constructors are proficient and possess sufficient knowledge and experience in their work. However, some construction situations may require sophisticated design decisions at the time of erection, and some constructors may attempt to make such decisions on their own despite a possible lack of capacity to do so. This may adversely affect the project safety.

Temporary structures cover those structures temporarily erected to assist in the construction operations and are central to shoring, formwork and slip forms, scaffolding, ramps and platforms, bracing and guying, cofferdams, and so on. Contract documents prepared by the engineers may specify the design of the temporary structures or may require that the contractor develop the necessary temporary structures. In the latter case, the contractor generally assumes responsibility for their integrity.

Scaffold failure is a common type of accident associated with temporary structures as demonstrated by many case histories. A scaffold resting on concrete blocks collapsed in Chicago killing 12 workers. Insufficient strength of the concrete blocks was determined as the cause of collapse. Fourteen injuries were reported on another accident in New Jersey when a scaffold collapsed pulling over a 12-ft (3.7-m) high masonry wall to which it was tied. Overloading with construction materials, coupled with poor base support of the planks was the apparent cause of failure. In a gymnasium remodeling project in Illinois, the roof collapsed when the scaffold failed to support the roof after the supporting beams were removed. The court decided that the designers were at fault in that they did not determine the scaffold's ability to support the roof since the scaffold was intended to be used as a shoring system. These cases underscore the importance of sound design and construction of scaffolds.

In a legal dispute, opposing parties may present expert witnesses (often engineers) who may testify about the competence of services as compared to that of another reasonable/prudent engineer in the same community. It is the expert's duty to perform the research necessary to establish the facts about the case and be totally impartial, although he or she is paid by a plaintiff or defendant. Thus, when experts disagree about given issues, as is commonly the case, the disagreement should stem from differing judgments and not from their acting as advocates for their clients.

Privity indicates direct contractual relationships. In the past, lack of privity protected design engineers from liability for third-party claims (mainly for injuries from construction accidents). It was understood that engineers were not obligated to protect the workers from safety hazards since they had no contract with them. However, increasingly, the engineer's liability is determined by the tort concept of foreseeability. In this context, if a party is negligent in the performance of its duties, it may be held liable to any party who suffered harm that was foreseeable at the time duties were undertaken.

The doctrine of *strict liability* generally applies to manufacturers of a product. It is far more stringent than the doctrine of professional liability, since negligence does not have to be proved. However, the following must be shown:

1. There was a defect in the product
2. The defect existed at the time the product was transferred to the purchaser/user
3. The defect caused or contributed to an injury
4. The product failed in normal use

Claimants frequently assert that the engineer should also be held strictly liable in tort for injuries caused by allegedly defective designs of inherently dangerous structures. However, several courts have rejected the strict liability doctrine in application to engineers, viewing the science of engineering and construction as inexact, in that engineers are not expected to guarantee the satisfactory results of their services.

23.5.3.5 Contract Administration and Project Management Contract administration encompasses all activities from the beginning to the end of a construction project relating to the execution of the project tasks as provided for by the contract documents. Shop drawings and change orders are the two most important aspects that may impact construction safety. They are discussed in the following paragraphs.

Shop drawings are documents prepared by various subcontractors who will perform specific construction tasks. They usually show the specific components of the constructed system and the details of the erection procedure. Shop drawings have two critical purposes. First, they constitute the contractor's interpretation of the design documents. Second, they provide the freedom to contractors (especially in complex projects) to propose construction details. This is particularly necessary where it is not economically feasible to include all the necessary details on the contract documents. Note that, in many cases, suppliers and subcontractors may have greater expertise in materials and construction methods and familiarity with some new technologies than the designer. This freedom generally results in a competitive bid since the contractor is not forced to comply with specific methods in which he may not be totally proficient. Shop drawings are reviewed by both the contractors and the designers. They are reviewed more critically by the responsible designers if the case concerns structural details. Usually shop drawings receive precedence over contract documents and, thus, become the sole source of information and guidance for the construction team. Any errors or omissions on shop drawings can lead to inadequate construction which may result in failures.

Deficiencies in shop drawing submittal and processing practices can jeopardize safe and efficient construction. Three crucial factors must be observed to minimize such problems, namely,

1. Efficient planning and time management
2. Adequate shop drawing reviews
3. Good communication between the designer and the contractor

Effective communication is perhaps the most critical factor in preventing failures. In practice, there is often some overlap of design responsibility between the engineer/design professional and the contractor/subcontractor. This is especially common in certain areas like steel connections or precast concrete construction. It is important that the designers develop excellent communications with the contractors and clearly express their intentions. Also, the contract documents must clearly define the extent of the contractors' responsibilities to eliminate any misunderstandings.

The Hyatt Regency walkway collapse generated considerable debate with regard to the inherent dangers of the traditional practices concerning shop drawing reviews. In approving the shop drawings, design engineers generally do not assume responsibility for the accuracy of the work represented in the drawings. They simply verify that the information submitted is in conformance with the design. In fact, the reviewed shop drawings usually contain a statement that defines the limits of the engineer's professional liability in terms of the scope of evaluation and the resulting action expected from the review. On the other hand, some contractors view the engineer's approval as assumption of responsibility for the outcome of the shop drawing. The fact that structural engineers for the Hyatt Regency walkways were found liable indicates that the engineer's role and responsibility goes beyond just reviewing the shop drawings to check for conformance to the design concept.

A change order is essentially a communication tool that is issued by designers to the contractors/subcontractors during the construction phase to alter the course of a project with owner's approval. The necessity for a change order may arise from unexpected site conditions, unavailability of materials or equipment, potential design flaws, changes in owner's requirements, and other relevant factors.

Change orders may have profound impacts on the project budget and schedule, often imposing additional demands and liability for cost of the related design and construction. This fact, coupled with time constraints and pressures during construction, can cause the designers to pay insufficient attention to the preparation of change orders. Yet, change orders become a part of the contract documents, and any errors or omissions on them actually can become design flaws, which might have an impact on the safety of the constructed facility and the site personnel. Change orders should contain clearly written instructions supported by complete and accurate drawings to avoid confusion and poor guidance on the project changes.

23.6 CONSTRUCTION SAFETY AND HEALTH MANAGEMENT

Managers responsible for construction projects must recognize that safety and health must be managed in the same way as cost, schedule, quality, and productivity for effective loss control. Management of safety and health deals with managerial decisions and actions taken at all levels to produce an organizational setting in which employees are trained, motivated, and supervised to perform work free from injury and illness. Development and implementation of a company safety and health program is key to good management. Safety and health programs are most effective when they are designed to meet the specific and individual needs of each company. It may even be useful to develop one for each project.

23.6.1 Safety and Health Program Elements

Depending on the sophistication of the contracting firm, the nature of the project, and owner requirements, a safety and health program can be relatively plain and simple, or highly detailed. In this regard, there is not a unique set of elements for a company safety and health program. The key elements believed to be common to successful programs are listed and summarized here.

Research has shown that safety starts at the top. Companies in which the chief executive has made a strong commitment to safety and health and communicates this concern to employees by word and deed have better safety records than companies for which this is not true. Although chief executives do not normally supervise construction workers on the jobsite, the image they project and the behavior they display on and off the jobsite can have a great impact on the company's safety performance. This can be achieved by

1. Creating an organizational culture in which safety and health are top priority
2. Holding line managers accountable for the safety and health of their subordinates
3. Focusing on and providing staff support and other necessary resources to help line managers meet their safety and health goals.

A written safety and health policy statement issued by top management is the best way to show the commitment to safety and health, to start the communication of goals and expectations, and to set the ground rules for the delivery of a successful program.

Clear goals for the safety and health program need to be set at the corporate as well as the project level. These goals need to be communicated to all employees. Goals may include zero fatalities and permanent disability causing injuries, prevention of major fires, and vehicle accidents and full compliance with OSHA standards. The goals set for the program should be measurable to assess whether they are being successfully met.

An organizational and administrative structure should be established to achieve the best results from a safety and health program through line management and support staff. This is possible through setting an effective chain of communication between all levels. There is no set rule as to the proper location of safety and health responsibility in an organization; it is company specific, depending on the need. It is common, however, to have a safety and health department at both the home office of a company, and at the individual project level. Usually, the home office serves a staff function, while the field office provides a line function under the project superintendent.

It is important to recognize that safety on the project is everybody's responsibility. Following the commitment of top management comes the assignment of safety duties. Employees at all levels must share in these duties because each employee has a safety duty to every other employee. Safety assignments must be understood by all employees. The right of each employee to a safe workplace must be explained and delineated, along with the duty to help keep it that way. It is highly desirable to put the assignments in writing. These might include individual responsibilities for program accomplishment and/or compliance for the company's president, management, jobsite superintendents, safety staff, foremen, craft personnel, subcontractors, suppliers, A/Es, owner personnel, and visitors. Accountability should also be addressed stating how incentives are distributed or disciplinary actions will be taken.

Maintaining an effective communication network at the jobsite is key to successful safety and health management. Managers can accomplish this by the chain-of-command system, direct contacts, and group meetings. The chain-of-command system alone has drawbacks since it can lead to distorted or "filtered" information through the hierarchy, which may provide misleading data on which to act. Managers who walk through the job frequently and talk directly with the site personnel can create a medium of effective two-way communication and convey their safety and health priorities. It is better when they share information with workers and foremen and listen to their feedback, rather than just issuing orders. This way the integrity of the chain-of-command system is not undermined. It is also important for follow-up action to be taken to address the issues discussed with the site personnel. Otherwise, the interest in participation may dwindle.

The safety supervisor/director plays an important role in the implementation of the safety program. These professionals are usually in staff positions rather than line positions, because safety is a staff responsibility. They provide support to the line managers who actually control safety and health performance. Among their duties are

1. Serving as consultants to management on technical and organizational aspects of safety and health
2. Developing and participating in the training and worker orientation programs
3. Assisting in safety and health planning, monitoring (jobsite supervision), and recordkeeping
4. Keeping the organization up-to-date on safety and health regulations and related matters

23.6.2 Project Safety Rules

All company safety rules should be published in a written form. Written rules are more easily enforced than unwritten ones and provide readily available guidance for operational safety. These rules should include the penalties for noncompliance. Each current employee, as well as any new employee, should be furnished a copy of the safety rules.

Each contractor should have rules for the basic types of operations it performs. In some cases there should be rules for the different company divisions or crews. Also, these rules should be modified as site conditions or owner requirements dictate. These work practices may be simple, 1-page handouts for specific operations such as erecting scaffolds, excavating trenches, or operating specific pieces of equipment. Depending on the situation, the safety rules may also be very involved based on a hazard analysis for critical tasks. In any event, there should be a list of general rules for everyone, such as those shown in Table 23.5. Rules must be concise and easy to understand. They are instructions to field personnel for safe working procedures; therefore, they must be in a format that can be easily implemented in the field.

23.6.3 Training and Worker Orientation

Training and worker orientation are necessary elements of an effective safety and health program. There are two types of training:

1. Training for specific tasks
2. General training in accident avoidance and prevention

It is desirable to integrate these efforts and train employees in safety and health as part of craft training. There are several ways to provide employee training. Discussions at weekly toolbox meetings are an effective training method. Many insurance companies and trade associations, such as the Associated General Contractors (AGC), provide "toolbox talks" to assist employers in the selection and delivery of safety topics. Private subscription services are also available to provide new topics on a regular basis.

Many insurance companies conduct safety seminars on the jobsite as part of their services associated with the workers' compensation coverage. This type of training can be utilized effectively before employees start work or on specific hazardous operations. Local safety councils and trade associations may provide free or low-cost training programs. These programs are tailor-made to meet specific training requirements. Many academic institutions also offer extensive safety courses. Finally, safety consultants provide both training and training program evaluation services. All training efforts must be carried out by qualified persons who are capable of stimulating and motivating the trainees.

23.6.4 New Worker Orientation

A significant component of the company safety and health training program is new worker orientation. New construction workers, and workers starting on an entirely new activity without previous experience, are particularly susceptible to being injured. The objectives of the orientation program are to relieve beginner anxieties, indoctrinate the workers to the company's safety program, and teach safe work practices, which include the use of personal protective equipment. Each employee should be handed a set of company safety rules during orientation and should be encouraged to read and understand them thoroughly.

Table 23.5 General safety rules

All of our safety rules must be obeyed. Failure to do so will result in strict disciplinary action being taken.

1. Keep your mind on your work at all times. No horseplay on the job. Injury or termination, or both, can be the result.
2. Personal safety equipment must be worn as prescribed for each job, such as safety glasses for eye protection; hard hats at all times within the confines of the construction area, and gloves when handling materials. Safety shoes are highly recommended for protection against foot injuries.
3. Shirts and long-legged pants must be worn to prevent sunburn and to protect against acid burns, steam burns, weld splatter, and cuts. Minimum clothing for the upper body is a “T”-shirt.
4. If any part of your body should come in contact with an acid or caustic substance, rush to the nearest water available and flush over the affected part. Secure medical aid immediately.
5. Watch where you are walking. Don’t run.
6. The use of illegal drugs or alcohol or being under the influence of same on the project shall be cause for termination. If you take or are given strong prescription drugs that warn against driving or using machinery, let your supervisor know about them.
7. Do not distract the attention of fellow workers. To do so may cause injury.
8. Sanitation facilities have been or will be provided for your use. Defacing or damaging these facilities is forbidden.
9. A good job is a clean job and a clean job is a safe one. So keep your working area free from rubbish and debris.
10. Do not use a compressor to blow dust or dirt from your clothes, hair, face, or hands.
11. Never work aloft if you are afraid to do so, are subject to dizzy spells, or if you are apt to be nervous or sick.
12. Never move an injured person unless it is absolutely necessary. Further injury may result. Keep the injured persons as comfortable as possible and utilize jobsite first-aid facilities until a doctor arrives.
13. Know where firefighting equipment is located and learn how to use it.
14. Learn to lift correctly with the legs not the back. If the load is too heavy, GET HELP. 20% of all construction-related injuries result from lifting materials.
15. Riding on loads, fenders, running boards, sideboards, and gates or with your legs hanging over the ends or sides of trucks will not be tolerated.
16. Do not use power tools and equipment until you have been properly instructed in safe work methods and become authorized to use them.
17. Be sure that all guards are in place. Do not remove, displace, damage, or destroy any safety device or safeguard furnished or provided for use on the job, nor interfere with the use thereof.
18. Do not enter an area that has been roped off or barricaded.
19. If you must work around power shovels, cranes, trucks, and dozers, make sure operators can always see you.
20. Never oil, lubricate or fuel equipment while it is running or in motion.
21. Rope off barricade danger areas.
22. Keep away from the edge of cuts, embankments, trenches, holes, and pits.
23. Trenches must be shored or sloped to comply with the most stringent requirements. Keep out of trenches or cuts that have not been properly sloped or shored. Excavated or other material shall not be stored nearer than 3 ft (1 m) from the edge of any excavation.
24. Use the “four and one” rule when using a ladder. One foot of base for every 4 ft of height (25 cm of base for every 1 m of height).
25. Always secure the bottom of the ladder with cleats and/or safety feet. Lash off the top of ladder to avoid shifting.
26. Ladders must extend 3 ft (1 m) above a landing for proper use.
27. Defective ladders must be properly tagged and removed from service.
28. Keep ladder base free of debris, hoses, wires, material, and so on.
29. Build scaffolds according to manufacturers’ recommendations.
30. Scaffold planks must be cleated or secured to prevent them from sliding.
31. Use only extension cords of the three-prong type. Check the electrical grounding system daily.
32. The use of safety belts with safety lines when working from unprotected high places is mandatory. Always keep your line as tight as possible.
33. Tar kettles must be kept at least 25 ft from buildings or structures and never on roofs.
34. Open fires are prohibited.
35. Know what emergency procedures have been established for your jobsite. (Location of emergency phone, first-aid kit, stretcher location, fire extinguishers locations, evacuation plan, etc.)
36. Notify your supervisor of unlabeled or suspect toxic substances immediately and avoid contact.

Source: NIOSH.

23.6.5 Accident Investigation and Recordkeeping

Accident investigations can highlight problem areas, and help detect patterns of unsafe acts or conditions, which should be addressed in preventive efforts. In this regard, accident reports make excellent training tools. Every accident, including those without injury should be investigated as part of the company safety and health program. The recordkeeping requirements are also an important element of the safety and health program. A thorough and continuous recordkeeping system for recordable cases, accident investigations, safety training programs, test results, and so on provides managers with significant information and management capability for good decision-making.

23.6.6 Safety Budget and Audits

A company truly interested in safety will include a budget covering safety program expenses, such as costs of safety personnel, training, protective equipment, and first-aid station maintenance. An advanced program will include these safety costs showing them as part of direct labor. An enhanced program will be able to demonstrate these costs as part of operational or task performance, such as a percentage of each cubic yard of concrete poured. Contractors should be required to show that they are prepared to allocate resources for safety supervision, joint consultation, training, safety equipment, and other necessary safety program assets in relation to the size and nature of the project.

Safety and health audits measure the program's effectiveness. Audits include both field inspections and overall program evaluation. One of the better tools for enhancing safety programs is a system of frequent on-site inspections. These inspections, or field audits, provide immediate and tangible evidence of the performance level of a safety program. They also provide for direct observation of any present or developing site hazards. On-site inspections keep safety at the forefront and ensure a high degree of compliance with the rules. Safety audits should be an ongoing activity throughout the project.

Inspections may be performed by owner representatives, site superintendents, foremen, or the company safety staff. The advantages of management's personnel performing the audits are as follows: (a) management is primarily responsible for safety, (b) managers control work performance, and (c) supervisors should know the hazards on the jobsite. The main disadvantage is the possibility of a conflict of interest. There may be a tendency to overlook or minimize some situations that might imply negligence on management's part. This problem can be alleviated when safety professionals perform the audits.

Numerous items can be targeted for inspection and jobsite observation. From a hazard standpoint, these may include housekeeping and sanitation, fire prevention, electrical installations, hand and power tools, ladders, scaffolding, cranes, hoists and derricks, heavy equipment, motor vehicles, barricades, handling and storage of materials, excavation and shoring, demolition, hazardous materials, welding and cutting, personal protective equipment, concrete and masonry construction, and steel erection.

A comprehensive safety and health program audit may include the following key elements:

1. Degree of management commitment
2. Presence and effectiveness of the policy statement
3. Program goal setting
4. Definition of safety responsibilities
5. Experience modification rating (over 2–3 yr)
6. Management supervisory meetings
7. Preplanning for jobsite safety
8. Effectiveness of training and orientation programs
9. Accident investigation activities
10. Recordkeeping
11. Substance abuse policy
12. Safety budget
13. Field performance audits

Incorporation of safety and health in construction contracts can make a significant and positive impact on the project's safety and health performance. One effective way of doing this is to establish target safety and health standards and criteria for the project.

Inclusion of safety and health in construction contracts places the required emphasis on their significance and encourages the contractor to pay systematic attention to safety and health. However, safety is usually not a pay item in the contract, and its costs are lumped into the "cost of doing business" or the project overhead. At the National Forum on Construction Safety and Health Priorities, it was strongly suggested that safety and health be included in the contracts as a bid item just like excavation, borrow material, and concrete. This will make owners pay for this item separately and it will provide the resources to contractors to invest in safety and health appropriately, delivering improved performance.

23.7 REQUIREMENTS FOR SAFETY IN CONSTRUCTION PROJECTS

23.7.1 Falls

Falls may occur near excavations or manholes when workers are entering or leaving the area, or when they simply may have not noticed the opening. In sewers, fall hazards may lead to drowning if the person experiences unconsciousness. The most effective means of fall protection on such locations are guard rails with toe boards, covers for openings, and hazard communication signs. Toe boards and screens are to be used to safeguard lower elevations from falling objects. Materials should not be placed near edges, where such protection is not provided. Prohibiting signs can warn workers against entering a fall hazard zone.

23.7.2 Excavation and Trenching

Excavation and trenching operations are essential to many types of construction projects. Foundations, drains, sewers, and underground utilities are part of a great majority of construction projects, requiring excavation or trenching. Excavation is the removal of soil and rock from the original location. The behavior of soil during excavation is dependent on its composition and the environmental conditions. Soil composition varies, from sand, which flows easily, through silt to clay, which is cohesive. Water is often present in soil to some degree and affects soil behavior. Many types of soils cannot support their own weight during excavation. Therefore, some form of support is often required. The exact method of support will depend on the soil type, as well as site and groundwater conditions. In shallow trenches, or excavations with depths less than 5 ft, sloping or shoring may not be required if the soil is cohesive. For deeper trenches, support will be required as decided by a qualified person.

23.7.2.1 Excavation and Trenching Hazards Detailed information on excavation and trenching hazards has been presented by the Center for Excellence in Construction Safety (CECS). The primary hazard in excavation operations is the possibility of *cave-ins*, or earth slides, burying workers in the trenches. Many of these accidents result in death. Persons buried beneath the soil become unable to breath and ultimately suffocate. Complete burial is, in fact, not necessary to cause death. The pressure of surrounding soil will force air from lungs and prevent further breathing. The likelihood of rescuing buried workers in such cases is usually not very high because heavy equipment cannot be used to uncover victims for fear of causing further injuries. Furthermore, sending rescuers inside a caved-in trench exposes more persons to the same danger.

The four important factors that contribute to cave-ins are as follows:

1. *Weight*: Weight is generated by the soil itself, construction equipment and vehicles, or other objects.
2. *Amount of water in soil*: Excessive moisture or lack of moisture can weaken the soil leading to collapse.
3. *Vibrations*: Vibrations are caused by activities such as vehicular traffic, blasting, and pile driving.
4. *Soil composition*: The composition and structure of the soil affects its stability during the excavation process. Mechanical failure due to removal of lateral support is also a critical factor in cave-ins.

Water, either by rain or percolation, can affect the lateral resistance of soil. Water can enter the dry soil through cracks and voids and cause softening and sudden loss of strength. In cold climates freezing and thawing can affect soil strength. Water can also increase the weight of soil and act as a lubricant contributing to slippage. Vehicle traffic, especially large trucks and heavy construction equipment, can cause soil displacement and instability. Other hazardous conditions are created by placing excavated earth, pipes, and shoring equipment too close to the trench to form additional (surcharge) loads. Impacts during unloading of these materials can lead to cave-ins as well.

Certain types of excavation hazards may be particular to a specific location, such as a trenching operation at or near previously excavated ground; excavation reduces the inherent strength of the soil. Therefore, starting a new excavation on previously excavated ground increases the possibility of a cave-in. Where new excavation is planned close or parallel to a previously excavated trench, it must be recognized that the ground between them may be particularly susceptible to collapse. Hazards are also caused by differing strata within a soil cut or by pockets of weak soil at the site. Existing vegetation such as large trees extracts the soil moisture through the roots and may contribute to loss of soil strength.

One of the greatest hazards encountered during excavation is the collapse of adjacent structures, which can result in catastrophes and fatal injuries. In construction projects involving proposed additions and renovations, a new trench is commonly excavated close to an existing foundation wall. If the soil is removed to a depth that is equal to or greater than the depth of the existing footing, and proper shoring is not provided to support the existing structure, the loss of bearing support of the excavated soil may cause a partial or full collapse of the existing structure.

Although cave-in failures are generally the major concern in excavation operations, other hazards may also be encountered. Excavations are performed for a variety of purposes and they involve a variety of activities. Physical injury from falls or falling objects, exposure to buried electrical cables, and hazards associated with confined spaces are also of major concern, and should not be overlooked.

Excavation cave-ins are preventable if certain precautionary measures are adopted. It is important to recognize that visual inspection of soil stability is not sufficient to ensure the long-term safety of an excavation because one cannot anticipate the changes in the weather, rainfall, or future unsafe practices. Therefore, soil analyses must be conducted by a competent person, for example, a geotechnical engineer, who will determine the adequate support system depending on the nature of the excavation, the soil, and groundwater conditions.

23.7.2.2 Shoring *Shoring* is the support system designed to prevent the lateral movement of soil, which can lead to cave-ins. It is commonly used for deep excavations, those that are 5 ft (1.5 m) or deeper. Various shoring systems are available to prevent excavation cave-ins. Aluminum hydraulic shoring systems have been developed as an economic method to ensure the safety of workers. Their light weight and adaptability to varying conditions make them very attractive. Other shoring systems include vertical shores for compact soils (“skip shoring”), stringer systems providing horizontal support for intermittent or solid sheathing for relatively unstable soils, and manhole braces.

Shoring systems must incorporate certain precautionary features to ensure the safety of workers:

1. Providing effective means of access and egress, such as securely fixed ladders in trenches that are 4 ft (1.2 m) or deeper
2. Erection of temporary barriers to guard against falls
3. Construction of diversion dikes and ditches to provide drainage of the adjacent area, which will prevent water from entering a trench

23.7.2.3 Sloping *Sloping* involves cutting the sidewalls of an excavation to form a safe angle of repose, which represents the angle at which the soil settles to a natural state with no tendency to further settle to a shallower form. The angle of repose varies with the specific soil conditions; however, it cannot exceed 45%. Sloping is not a practical approach for deep excavations because it requires excessive removal and replacement of soil. However, sloping can be effectively used in conjunction with shoring. Typically, the top of the trench is sloped in such cases to allow for easy installation of the shoring system at lower depths.

23.7.2.4 Trench Shields *Trench shields* (boxes) are essentially used as “personnel protectors.” Such devices are commonly placed in an already excavated, but unshored, trench or pit. Normally, the trench walls remain intact long enough to complete the construction. If they collapse, however, a properly designed shield must be capable of withstanding the maximum anticipated lateral soil stress at a given depth. This minimizes the potential for worker injury. Trench shields are most often used in open areas away from existing utilities, streets, and buildings that may require a support system. If ground conditions are favorable and the trench zone requires no direct support, the use of a trench shield may be an excellent choice. However, when the job calls for strict compaction and replacement requirements, one must be cautious. If unstable soils slough off as the shield is pulled up the trench line, it could make proper construction a difficult and costly task.

23.7.2.5 Inspections Inspection of excavations by a competent person can assist in assessing the soil stability and the quality of the shoring system. Daily inspections should be performed. Attention should be directed to the following during the inspections:

- Bulges on trench walls, cracks near the edges and walls
- Accumulation of loose rocks fallen from the trench wall
- Water in the excavation area
- Soundness and quality of the shoring materials (e.g., timber that has decayed)
- Improper connections in the shoring system, and other signs of distress

Other safety measures adopted to protect workers against excavation hazards are as follows:

- Keep the excavated soil pile at least 2 ft (0.6 m) from any opening to avoid excessive surcharge pressure.
- Keep all vehicles and equipment away from the trench.
- Be aware of vibrations, overhead utilities, and underground utilities.
- Be aware of toxic fumes. Toxic fumes can seep through the soil and accumulate at the bottom of an excavation. Clean air should be provided in sufficient volume to dissipate the toxic gases.
- While excavating the soil, clear all unnecessary workers from the area.
- Protect the public by installing guardrails, barricades, and warning signs around the excavation area.

23.7.3 Confined Space Entry

Confined spaces are enclosed areas having limited access or egress. They can be storage tanks, boilers, bins, silos, and so on, which are accessible through a manhole. Confined spaces commonly encountered on construction projects include basements, trenches, shafts, bore holes, ducts, pipelines, drains, and sewers. Entry into confined spaces can be hazardous and requires special care. Fatal accidents in confined spaces are common.

The atmosphere in confined spaces may be hazardous due to a lack of oxygen, the presence of toxic agents, or the presence of flammable and explosive gases. Other hazards in confined spaces include moving parts of machines, extreme temperatures, collapses and cave-ins, and sudden flooding. The difficulty of access and egress and the difficulty of moving in confined spaces amplify the impact of these hazards. Some specific hazards are discussed next.

23.7.3.1 Oxygen Deficiency Small drops in oxygen level can cause loss of balance and concentration, breathing difficulties, and fatigue; a continued drop in the oxygen level may result in unconsciousness and death by suffocation. According to the National Institute for Occupational Safety and Health (NIOSH), an oxygen-deficient atmosphere has less than 19.5% available oxygen (O_2), requiring an approved self-contained breathing apparatus (SCBA). Atmospheres with less than 16% O_2 cause faulty judgment and rapid fatigue. Atmospheres below a 6% O_2 level create breathing difficulties and cause death in minutes. Oxygen deficiency is most commonly caused by gas leakage into the confined space, oxidation due to corrosion or bacterial growth, and exhaustion of the oxygen supply due to inhalation and combustion.

23.7.3.2 Other Atmospheric Hazards Presence of toxic gases in sufficient quantities is another source of hazard. Hydrogen sulfide is a toxic gas commonly produced by the decomposition of organic matter and is often present in the sewer systems. Hydrogen sulfide is both toxic and explosive. Another highly explosive gas, methane, is also often present in sewer lines. Other lethal gases are carbon dioxide, which is naturally present in soil, and carbon monoxide, which is usually produced by internal combustion engines. Leakage can cause chemical vapors to poison a confined space atmosphere; ammonia and chlorine are typical toxic fumes that may be encountered. Nitrous fumes may be left in a confined space immediately after explosive operations. Very small quantities of certain vapors and gases can cause fires and explosions. Whereas methane and hydrogen sulfide are explosive gases, petroleum and vapors of solvents such as acetone and toluene can cause fire.

23.7.3.3 Hazard Control in Confined Spaces Good advance planning and adequate training in safe methods of work in confined spaces will enable workers to enter these areas safely. Confined spaces training will vary according to the nature of the activity and the confined space. The training program should consider the role of supervisors, workers, persons outside the confined spaces, and rescue personnel. A permit-to-work system, in which each step is planned and authorized, is required for confined space entry. Other alternatives to entry should be considered for confined space operations wherever possible. If entry to the space is unavoidable, then appropriate breathing apparatus should be considered. The use of mechanical or forced ventilation is the first choice. Before entry into a confined space, the space should be withdrawn from service and be isolated from electrical and mechanical sources.

23.7.3.4 Monitoring, Protective Equipment, and Communications Personnel planning to work in confined spaces should check for leaks and test the atmosphere for oxygen and flammable, explosive, and toxic gases. Once begun, the operation should be constantly monitored. Personal protective equipment such as breathing apparatus, harnesses, lifelines, and rescue equipment must be provided and used as necessary. Personnel must be informed about the details of activities and the required communications. A trained person should be in attendance and in continual verbal communication with the workers in confined spaces throughout the operation.

23.7.3.5 Rescue Operations Rescue operations may be needed when a person is injured, or collapses in a confined space, becoming unconscious. Rescuers must wear breathing apparatus and safety harnesses that are attached to a lifeline before entering the space. The rescue equipment must include lifting tools such as tripods and winches to carry the unconscious worker.

23.7.4 Heavy Construction Equipment

Heavy construction equipment is an integral part of construction operations. The machinery is used for many demanding tasks that are beyond human capability. Earth-moving equipment such as bulldozers, scrapers, motor graders, and front-end loaders are typical of the heavy construction equipment that performs cutting, transporting, and grading tasks.

Bulldozers are used to strip soil in layers. They move and grade earth/rock material for distances less than 300 ft (91.4 m). The various types include bulldozers and angle dozers. Scrapers also strip in layers. However, they also load the earth into bowls, and haul, spread, and partially compact it. Types are crawler-tractor-pulled scrapers and wheel-tractor-pulled scrapers. They represent a compromise between the best backing and best hauling machines. Motor graders cut, shape, and grade. They have a blade that can be set at different angles. They can be quite versatile when fitted with rippers, backslopers, snow plows, and so on. Front-end loaders are used to transport bulk materials, to load trucks, to excavate earth, and so on. Types are crawler and wheel loaders. Other types of heavy construction equipment such as hydraulic excavators, trenching machines, tractors, and rollers are also available.

23.7.4.1 Heavy Equipment Safety Hazards Many of the hazards associated with heavy construction equipment are common to all:

- Poor repairs and service
- Obstructed view during backing
- Striking people and collision with other equipment
- Travelling empty at excessive speeds
- Pinch points between equipment and objects
- Riders falling from equipment or bucket
- Overturning of the equipment
- Unexpected electrical shock
- Failure of lifting mechanisms/operational failures
- Injuries to operators due to ingress/egress difficulties

- Runaway machines (e.g., if wheels were not blocked upon parking or if operator was unable to control the machinery)
- Being struck by limbs of trees or other overhead obstructions, and moving equipment

23.7.4.2 General Safety Precautions Recommendations for general safety precautions associated with heavy construction equipment are as follows:

- Management should carefully select competent operators.
- Rules for operation should be clearly stated.
- All equipment should be kept in good working condition.
- All new machines should be equipped with rollover protection
- Existing machines with rollover protection should be maintained in good working condition.
- Adequate illumination should be provided for night operations.
- Dust must be kept down on all roads.
- Operators must be given clear and specific instructions.
- All personnel should be clear of the work area.
- Unauthorized riding on the equipment should be prohibited.
- Speeds should be consistent with job conditions and OSHA requirements.
- Use a signal person in busy areas.
- Equipment should have an audible reverse signal alarm system that operates automatically with backward movement.
- Repairs should be done when the machine is not running.
- During refueling operations, all personnel in the vicinity should stop all motors and refrain from smoking.
- Personal protective equipment (hard hat, steel toed shoes, gloves, safety glasses, respirators, etc.) should be used at all times.

23.7.4.3 Maintenance and Training Maintenance is extremely important in securing the safety of machinery and equipment. OSHA standards require inspection of all machinery before each use. Results of tests and inspections should be recorded. Frequent inspections should be made on wire ropes and guys, hoists and trolley cables, jib and counterweight jib guy lines, hoist rope anchorage on winding drum, foundations, and structural connections.

Training of workers on all cases is essential and must include a review of potential hazards and familiarization with the equipment.

23.8 OCCUPATIONAL DISEASES

In the past decade, a greater public awareness of occupational diseases and their consequences has created a broad-based involvement with issues related to worker health and safety. Studies related to occupational diseases are no longer restricted to healthcare professionals, and the field of industrial hygiene contributed significantly to this change.

As scientists have increased their ability to analyze minute concentrations, those who study occupational disease have established that materials, biological agents, or energies existing in very low concentrations may be harmful to humans. Exposure to materials that exist as minute contaminants of other industrial raw materials may also represent a risk to worker health.

Engineers who, as a profession, are often responsible for the design and construction of municipal and industrial facilities and the protocols relating to their operation must now become more involved with worker health-related issues. New materials or new or improved processes, or both, that are constantly being introduced require greater awareness of their health effects at the design stage. Engineers should not relegate health considerations to retrofitting practices. Increased public awareness has also had positive results, for example, greater worker awareness of health-related issues, setting new standards of exposure and adjustment of old standards to reflect a more up-to-date understanding of occupational diseases, and recognition of the need to instruct professionals such as engineers in health-related issues. By greater understanding of these issues, engineers will be able to contribute to improving the conditions that influence the well-being of workers.

Occupational diseases are preventable. An understanding of the nature of occupational diseases (identifying the causal agents, quantitatively assessing the human exposure, and learning about the interactions between hazardous agents and the human body) should result in a safer work environment. In this enlightened situation, arrived at partly through the actions of engineers, health effects associated with a specific operation will be considered in the overall engineering design. Being aware of the interaction of a worker with his or her work environment will affect process design, including engineering controls, decisions about safe operating practices, and the use of protective equipment.

23.8.1 System Approach

When investigating the development of an occupational disease, one is concerned with the interface and interaction between the human body and the work environment. An additional and important concern is the nature of the disease process, including the structural and functional changes occurring in the human body that result in or are a result of a disease process (the study of human pathophysiology). Together, all of these disciplines contribute to an understanding of occupational diseases. In this work, a systems approach is used to investigate the behavior of the human body in an occupational setting. This requires a description of the interaction of the human body with potential disease-causing chemical, physical, or biological agents in the environment. Matters are complicated by the fact that only a fraction of a worker's time may be spent in an occupational environment and that exposure to potential disease-causing agents (including chemical agents, physical energies, or biological entities) may take place in the home environment or as a result of nonoccupational activities.

A thorough study of occupational disease requires a multidisciplinary approach. Physical sciences and engineering analyses are useful to describe the interface of the human with the work environment. Life sciences information, including biochemical, biomechanical, toxicological, and pathological descriptions, is needed to assess the physiological effects of occupational exposure. In addition, the social patterns of the workers (i.e., lifestyle considerations) may influence the outcome of exposure.

23.8.2 Complexity of the Issues

The central focus in discussing occupational diseases is the human worker and his or her body a complex system of physiological and psychological (behavioral) components. Just as no two people will behave exactly alike in a work environment, they also will not have the same responses to stimuli. Not only can the range of human responses to physical, chemical, or biological agents be dramatic, so can the range of idiosyncratic behavior and personal hygiene. Cigarette smoking and willingness of a worker to wear protective equipment are two examples of personal habits that may influence the risk of occupational diseases.

A person's work history may also influence his or her potential for developing an occupational disease. Past exposure to disease-causing agents may have resulted in the bodily accumulation of agents that are slow to be removed. Body burdens (the amounts of these agents in various body compartments) may stress normal physiological systems. Alternatively, previous work exposure may have affected homeostasis (the stabilizing tendency in the human body) and, thus, rendered the individual more susceptible to certain occupational diseases. Assessing the effect of work histories on the potential for developing an occupational disease is difficult and is a subject of ongoing research.

Generally, the differences in individual susceptibility to occupational disease can be related to general health, age, sex, race, diet, and heredity. These differences, as well as the limited information relating exposure and human response, significantly complicate the task of setting "safe" levels for potentially harmful agents. For these reasons, limits for occupational exposure cannot be considered as absolute levels of protection. Although limits are established, this is not assurance that individual workers may not show deleterious effects if they have unusual susceptibility.

23.8.3 Scientific Factors

Many dilemmas exist concerning the role of scientific "information" and occupational disease. Most scientific information that relates exposure to diseases comes from three qualitatively different types of investigation:

1. Case studies of accidents or disasters
2. Epidemiological studies
3. Basic and applied scientific research

Accidents and disasters such as occurred in Chernobyl and Hiroshima serve as unparalleled sources of information concerning human exposure to radiation. The human and environmental exposure as a result of the industrial accident in Bhopal, India, is and will remain under investigation for many years. While the moral, ethical, and legal implications of these accidents are being explored, scientists are gathering data from medical records and environmental measurements to assess human and environmental responses to ionizing radiation and widespread isocyanate exposure. Sometimes information from accidents and disasters is claimed to be "gained from experience." Nonetheless, perhaps the most useful outcome of these accidents/disasters is the corrective changes that diminish the probability of a recurrence.

Epidemiological research attempts to relate exposure to harmful agents and the occurrence and distribution of disease or injury in segments of society. Unlike clinical medicine, where the focus is on the diagnosis and treatment of an individual, epidemiology emphasizes the patterns of disease or injury in groups of individuals in order to identify causality. Interpreting causal relationships is made difficult because of systematic errors (called biases) introduced into a study or because individuals are exposed to factors other than a particular environmental agent factors that may influence the outcome of a disease (a *confounding factor*). The epidemiologist's raw data may consist of monitored or estimated patterns of exposure, job titles and death records, or medical history records. The results of a study may be a description of an incidence rate for a particular disease, and it may be standardized to the general population or expressed in a proportional manner. The statistically determined increased risk of developing a disease based on exposure to a specific agent is often sought in these studies. Results of epidemiological investigations have been extremely beneficial in assessing causal relationships. They are, however, often clouded by imprecision because of faults with the data sets and the presence of confounding agents and biases in the studies.

Scientific laboratory research aimed at better understanding the interaction between exposure to an agent and a disease is limited primarily to animal experimentation. For example, laboratory tests seek to identify harmful agents that interfere with reproduction in particular, agents that interfere with transferring genetic information from parent to offspring or that may produce physical defects in the offspring. The great complexity of and physiological differences between human and other animal reproductive systems is such that extrapolating laboratory information from one species to the other should be approached cautiously.

Because conducting animal experiments is expensive in terms of time and money, the usual approach is to subject the animals (usually rats and mice or other small mammals) to very high levels of the substance being investigated. This way a sufficient number of positive (disease-producing) results can be obtained. The extrapolation procedure is subject to criticism, and some of the issues relate to

1. Differences between laboratory test animals and humans
2. Well-controlled administration of test agents in the laboratory versus uncontrolled occupational exposure of humans
3. The often several orders of magnitude difference per mass of body weight of laboratory versus occupational exposure
4. The short-term laboratory exposure versus the chronic (long-term) occupational exposure

Much of what we know about exposure to potentially harmful agents and the development of occupational diseases is obtained from laboratory experiments on animals. Regulatory policy relies heavily on such scientific endeavors. The drawbacks and weaknesses of this approach, however, continue to create a dilemma for medical professionals, other scientists, and policy makers.

23.8.4 Occupational Disease as a Process

Occupational diseases develop out of the complex circumstances involving the pathophysiological response of a human exposed to harmful agents. Despite recent, rapid advances in science and medicine, much of the information describing the etiology (the study of factors that cause diseases) of occupational disease either does not exist or is poorly understood. The large variability among humans in their responses also contributes to this unclear situation. Still other factors have hindered the understanding of occupational diseases: the long latency period of some occupational diseases, the difficulty in establishing causality between exposure and development of a disease, the lack of emphasis on occupational diseases during medical training, and the multifactorial nature of real-life exposure.

From an engineering point of view, the development of an occupational disease may be considered a complex, incompletely understood process involving a contaminated environment (forcing function) acting on a physiological entity (the worker or physiological transfer function) with the potential of producing a pathological condition (a perturbed state). Occupational diseases are usually studied according to functional physiological units (e.g., organs) or according to hazardous agents (or materials). The approach here differs from the standard healthcare-oriented approach; the emphasis is on system behavior including interaction with the local environment.

Exposure to some potentially harmful agent (a chemical substance, a biological material, or energy) becomes an input to the human body (dose) when it crosses the hypothetical surface separating the occupational environment from the internal milieu of the body. This dose now elicits the possibility of two initial responses: normal or altered body processes, each with their inherent defense mechanisms and elimination. The altered body process route may be in response either to previous exposure or to genetic characteristics that predisposed the body to be hyper- or hyposusceptible to a stimulus. The final state of the body that results from the input and physiological and biochemical processing steps is (a) homeostasis in which normal conditions are attained; (b) acclimation, in which changes occur that do not impede normal function; (c) disease that differs qualitatively from acclimation only in that the changes that occur are greater and in that normal function is disrupted; or (d) death.

23.8.5 Potential Hazards

Chemical agents (including particulates, gases, vapors, liquids, and combined forms of these) probably represent the largest category of industrial hazards to health. In addition to these chemical substances, energies (including various forms of radiant energies, mechanical energies associated with physical labor, and biomechanical operation of the human body) may be hazardous. Climatic conditions such as temperature or humidity represent potential hazards. Biological agents (ranging from microorganisms to insects) may also be deleterious. In a negative sense, the deprivation of any normal input required to maintain normal human function (e.g., oxygen deprivation) is itself a hazard. Finally, physiological and sociological stimuli are potential hazards.

23.8.6 Modes of Entry

Any of the potential hazards or circumstances above can affect the internal human body via inhalation, ingestion, irradiation, or information exchange (information that may affect mental health). Where an energy transfer takes place between the worker and the work environment, the form of exchange may result not only from environmental stresses such as climate, light, sound, and heat but from the physical aspects of labor. The physical aspects of labor are part of the worker job interaction, that is, the study of ergonomics.

Hazardous agents usually enter the body via *inhalation*. Our respiratory system is a highly organized mass transfer apparatus; its primary function is oxygen exchange into the blood and carbon dioxide exchange from the blood to the outside environment. This physiological system is effective in removing particulates, gases, and vapors from inspired air. The specific effects and the site of action of these airborne materials are determined by the organization of the respiratory system, including the dynamics of breathing, as well as by the physical, chemical, and physiological properties of the agents being captured.

Ingestion is not considered a major pathway by which foreign agents enter the body; nevertheless, it is worth noting. Either accidently (e.g., poor hygiene) or as a consequence of a contaminated environment (exacerbated by mouth breathing), foreign agents may be ingested. The ingested agents usually wind up in the stomach where they are exposed to an active physiological system characterized by low pH and high digestive enzyme activity.

Transport of agents across the skin to the internal body environment can occur via several distinct pathways. Transcutaneous absorption will depend on the barrier function of the skin and the physicochemical properties of the absorbing agent. Adsorption onto the skin may be a first step that results in the material eventually being transferred to the internal milieu (milieu interne). Abraded or punctured skin opens a direct entry route to the internal environment for hazardous agents. The direct route may provide an entry several orders of magnitude greater than that of adsorption. Biological agents such as ticks or microorganisms can also penetrate the skin and gain entry to the internal environment.

Irradiation (the exposure of the body to either ionizing or nonionizing radiation) may affect the skin surface or may penetrate to deeper layers. This form of energy exchange could be from sunshine or from exposure to radioactive agents.

Information is a psychosocial form of exchange with an environment that is very difficult to quantify. A conscious or subconscious input is received by the worker, and his or her mental health depends in part on information.

The amount of a hazardous agent that reaches the body's internal environment is referred to as the *dose.* For most agents, the notion of a dose may be easy to quantify (e.g., chemical agents); for others, the concept of a dose may be almost meaningless (e.g., as in the case of information). A dose may be referenced to a specific target organ or to a site of exposure, or it may be generalized to represent systemic contamination.

23.8.7 Body Processes and Defenses

Once a dose of some agent has entered, a normal biological response indicates that no altered sensitivities or hyper- or hyporesponse is initiated immediately following the dose. As part of the immediate and subsequent response to foreign agents or energies, defense mechanisms and eliminations are involved. These are related intimately to normal body processes.

Normal body behavior or normal physiology is beyond the scope of this book, but many texts are available dealing with this subject. The normal biological processes following exposure to some potentially hazardous agent may depend on the quantity of the dose, the particular agent or energies involved, and the temporal pattern of the dose. As a result of either acute (short-term) or chronic (long-term) exposure to harmful agents, the state of health of the worker may change. The normal body process should be considered as a starting point or as an initial condition in this dynamic system. That is, the "normal" processes can be changed as a result of inputs.

The human body is well equipped for defense against most foreign agents. For each mode of entry of agents or energies, the body has specific defenses or protective mechanisms. Examples of these are nasal filtration of particulates and the barrier function of the skin. Once an exposure has resulted in a dose of an agent, the defense mechanism associated with the internal biological environment may be summoned. For a thorough description, refer to appropriate biological or medical sources.

The *immune system*, for example, is a complex defensive network that relies on specific cells to recognize foreign agents and to mobilize other cells (e.g., lymphocytes) to "attack" the foreign matter (biological or chemical). Concurrently, circulating antibodies (antagonists to specific materials) are synthesized, after recognition of the foreign agent, to enter the fray to destroy the foreign agents. As part of this system, a highly specific memory is developed from previous exposures to some agents. With the aid of this recall, the response time can be shortened and the degree of response increased for subsequent exposure.

Cellular defenders, other than lymphocytes as discussed above, can be found systemically in body fluids (e.g., blood or tissue fluids) or they can be associated with a particular tissue. The manner by which cells can defend against the presence of foreign agents includes engulfing or ingesting foreign agents (phagocytosis) and thereafter exposing these agents to internal cellular materials such as enzymes, which may chemically digest them; and producing and secreting substances, which aggressively coat or attack the foreign agent.

Inflammation, broadly defined, is a defensive reaction of tissue to injury characterized by redness, heat, swelling, and tenderness in the affected area. This reaction usually occurs along with other defensive mechanisms. Inflammation involves microvascular, cellular, body fluid-associated, and systemic components. The term *inflammation* is somewhat arbitrary because, in reality, it is part of a process with ill-defined stages. Inflammation usually results from biological agents (e.g., bacteria), but energies such as those associated with sunburn, frostbite, or physical labor may produce inflammations. Inflammation is usually classified as acute or chronic. With an acute inflammation, the return to a normal condition can be expected in a matter of days as a specific population of defensive cells migrates to the inflamed area. Chronic inflammation is characterized by persistence over a long period of time, a cellular population in the injured area that is very different from that of the normal and the acute condition, and the excessive production of tissue structural materials (e.g., collagen). Inflammation, even though it is involved in tissue response to injury and the subsequent repair process, may itself have harmful effects. Inflammation with excessive pulmonary edema (accumulation of fluid in the lungs) may be life-threatening.

The body can also transform agents foreign to its internal environment. This biotransformation results from the action of enzymes associated primarily with liver function and is classified into breakdown reactions (oxidation, reduction, or hydrolysis) and synthesis reactions (conjugation). Generally, biotransformations convert materials to other forms (metabolites) that are more easily eliminated from the body, or that are less toxic, or both. These effects are achieved generally by alterations in solubility and/or in chemical activity of the xenobiotics.

In addition to the primary defense mechanisms, discussed earlier, protective characteristics can be attributed to the normal homeostatic (preservation of the stability of the system) mechanisms and to alterations in growth patterns. Altogether, the body can muster impressive defensive possibilities.

23.8.8 Elimination

Elimination is part of the dynamic and interacting system that determines the intake, distribution, and fate of agents that are potentially hazardous. As such, it can be considered an aspect of normal processing as well as a specific defense mechanism. The major excretion pathways are via urine and feces. Materials can also be eliminated in sweat or during normal surface cell turnover (exfoliation), including loss of hair and nails, and in exhaled gases.

The pathway of altered body processes acknowledges the dramatically different response of an individual with altered susceptibility to a dose of some agent. Hypersensitivity can be found in a subpopulation of individuals whose immediate response to certain agents involves immune-mediated events not characteristic of the immediate response by the normal population. The relative response by the subpopulation of hypersensitive individuals can be extreme when compared with the response of normal persons. Also, the susceptibility of the otherwise normal population can vary significantly. Some individuals can exhibit extreme responsiveness to certain agents in the absence of action by the immune system. Both the hypersensitive and normal but hypersusceptible groups display altered body processes.

Altered body processes can involve a target organ, such as the skin or the lungs, or they can be systemic. The onset of the physiological response to a foreign agent may be rapid and exaggerated or, under specific conditions, it may be delayed. In any event, the eventual long-term outcome may be very similar to that result obtained following the normal body process route.

23.9 ERGONOMICS

23.9.1 The Worker and Work

Occupational disease may result from biomechanical, psychological, or environmental stresses associated with work. *Ergonomics* is the interdisciplinary study of the interaction of the worker with the work environment. The objective of ergonomic studies is to make the human work experience more efficient while promoting the well-being of the worker. This involves establishing and maintaining compatibility between equipment, tools, tasks, and environmental factors on one hand, and human anatomical and biomechanic considerations and perceptual and behavioral characteristics on the other. Its interdisciplinary nature is a critical feature of ergonomics, which combines physiology, psychology, engineering, and anthropometry (measurements with respect to the human body). The nonpsychological aspects of ergonomics are sometimes referred to as human factors engineering. The range of subjects that can be studied from an ergonomic point of view is quite diverse and includes analyses of static and dynamic human body biomechanics, metabolic and physical work requirements, use of tools, repetitive motion tasks, climatic and other environmental effects, lighting, equipment and process design, job demands, and mental and cognitive demands, to name a few.

Ergonomic considerations include aspects of safety program and managerial components as well as engineering, equipment, equipment maintenance, medicine, and training. To investigate the possibility of potential problems in a consistent, rational manner, ergonomic checklists have been designed. These devices are used to systematically examine the wide range of factors that may be involved in a system composed of a human working in an occupational environment. Common to many of the checklists are the following:

1. Analysis of human capabilities with respect to work station design and layout
2. Investigation of equipment design regarding the reliability and ease of equipment use
3. Analysis of the risk associated with physical workloads
4. Information handling and decision-making
5. A survey of environmental factors including illumination, noise, vibration and climatic conditions
6. Consideration for work schedules

NIOSH has proposed prevention strategies for the 10 leading work-related diseases and injuries. Included in this list are musculoskeletal injuries (including back injuries), traumatic injuries including death, noise-induced hearing loss, and psychological disorders. The incidence of these injuries or diseases can be reduced by effectively applying ergonomic considerations.

23.9.2 Adverse Effects Caused by Workplace Conditions

Examples of adverse effects on a worker serve to illustrate the scope of ergonomic problems. Cumulative traumas can result in disorders to

1. The nervous system, such as damage to the peripheral nervous system
2. The tendons and tendon sheaths, such as in carpal tunnel syndrome or in epicondylitis (tennis elbow), which may result from repeated hand/wrist movements
3. The lower back, the site of some of the most costly occupational injuries based on number of injuries and associated medical expenses
4. Joints, such as in bursitis and degenerative joint diseases

The specific region associated with the symptoms of cumulative trauma is usually in the upper part of the body in either soft tissue or joints.

Workers who use vibratory equipment, such as jackhammers or chain saws, sometimes register such complaints as numbness and blanching of fingers, pain, loss of muscular control, or reduced sensitivity to heat and cold. These are the symptoms associated with Raynaud's or vibrational white finger disease. This condition can arise after prolonged and repeated minor insults (cumulative trauma) to the body such as from vibrations or from being struck by objects.

Noise-induced hearing loss can be either temporary or permanent. It can result from physical interference with the transmission of sound or it can result from neuropathologies. The causative agent is the sound power level or the level of impact noise. Industry abounds with equipment and operations that produce excessive sound pressure and noise impact levels.

Another example of an occupational disorder preventable by ergonomic considerations is that of fatigue. Though fatigue is not technically an occupational disease, it is so prevalent that it warrants mention. Fatigue is defined operationally as impairment in the ability of a person to perform efficiently because of prolonged or excessive physical or mental exertion. Causes of fatigue include monotony, work intensity, and psychological and environmental factors. Fortunately, fatigue can be cured by rest. If insufficient rest follows bouts of fatigue, then complete recovery cannot be ensured and further relapse into unproductive work may result.

Table 23.6 Conditions caused by biomechanical and environmental stresses

Injury/Symptoms	Commonly affected workers
Back problems	Material handlers
Carpal tunnel syndrome or tendonitis	Clerical workers, assembly line workers, check-out workers, stamping job workers
Raynaud's syndrome	Forestry workers, construction workers
Degenerative joint diseases	Material handlers, forestry workers
Eye strain resulting in fatigue	Clerical workers, foundry workers, high precision assembly and inspection workers
Hearing impairment	Furnace operators, truck drivers, machinery operators, construction workers
Segmented vibratory diseases	Chainsaw chipper and jackhammer operators
Loss of strength, problems with hand-eye coordination, decreased mental capacity, fatigue	Most workers

Source: NIOSH.

Table 23.6 is a selected list of occupational injuries or symptoms and the workers that fall within the field of study of ergonomics. As the average age of the workforce increases, so too will the potential for problems requiring ergonomic solutions.

PROBLEMS/QUESTIONS

23.1 What is the principal advantage of a system safety program compared with a conventional or traditional industrial safety program?

23.2 How is the construction industry different from manufacturing and other industries?

23.3 List and discuss the seven critical issues that were established as needing attention at the National Forum on Construction Safety and Health Priorities. Which one of these issues do you think is the most important? Why?

23.4 What are the root causes of accidents in the construction industry?

23.5 Contrast the perspective of the reliability engineer with that of the systems safety engineer.

23.6 Who in an enterprise establishes risk tolerance levels?

23.7 Why is it important to establish an exposure interval when evaluating risk?

23.8 What are the advantages and limitations of a risk assessment matrix?

23.9 What are the differences between an FMEA and an FMECA?

23.10 What are the five major classes of causal factors of construction failures?

23.11 Describe the role of natural disasters in causing or contributing to failures.

23.12 Define constructability. What should designers know about constructability?

23.13 What causes material failures? How do material failures lead to construction failures?

23.14 How do shop drawings and change orders impact construction site safety?

23.15 What is meant by the term *temporary structures*? What is their significance with regard to construction safety?

23.16 What should go into a policy statement prepared by top management?

23.17 Why should all projects have a set of project safety rules?

23.18 What are the most effective means of fall protection?

23.19 What is meant by the term *cave-ins* and what are the factors contributing to cave-ins during excavation and trenching operations?

23.20 Name some excavation and trenching hazards other than cave-ins.

23.21 Discuss hazard control in excavation and trenching by addressing the shoring systems, sloping, and trench shields.

23.22 Define the term *confined space*. What types of hazards are present in confined spaces? Describe the hazard control principles associated with confined space entry.

23.23 What are the principal safety hazards of operating heavy construction equipment such as bulldozers, scrapers, graders, and loaders? How can these hazards be prevented or mitigated?

23.24 What are the main modes of hazardous agents' entry into the human body?

23.25 Give four examples of adverse effects on a worker that illustrate the scope of ergonomic problems.

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Chapter 24

Engineering Projects Management

24.1 ROLE OF ENGINEERS

The planning, design, and construction of water and wastewater systems for metropolitan areas usually bring together a sizable and varied group of engineering practitioners and their consultants, not for months but for years, in a bold and busy venture. As their work is completed, elements of their most important and powerful membership move on to new enterprises; older engineers drop out; younger engineers move in; and offshoots accept parallel, usually smaller, but nevertheless important, assignments. Under proper leadership, task forces perpetuate themselves to attack new problems or deal with old ones in new ways. The science and practice of water supply and wastewater collection and treatment are preserved and promoted in this way.

Because the systems are generally in public ownership in the United States, studies, plans, specifications, and contracts for the construction of water and wastewater works are prepared by engineers normally engaged by the cities and towns or the water or wastewater districts to be served. Private water companies are increasing in number, while private sewerage corporations are still rare institutions. The engineers may belong to the professional staff of the municipal or metropolitan governmental agencies responsible for designing and managing public works, or they may be members of a firm of consulting engineers. For very large undertakings, governmental and consulting staffs may be expanded for the duration of the enterprise, as suggested in the first paragraph of this chapter. For smaller undertakings, this is seldom true. Consultant groups are given most and possibly all of the responsibility. Engineers for manufacturers of water and wastewater equipment also have a part. The engineers of contractors or construction companies bring the design into being.

24.2 STEPS IN PROJECT DEVELOPMENT

Community action leading to the study, design, construction, and operation of new or enlarged water or wastewater systems and the engineering response elicited by community action are conveniently listed in sequence here, with special reference to projects for which engineering consultants are engaged.

24.2.1 Community Action

Community actions expected for water and wastewater works project development are summarized in Fig. 24.1 and include the following sequential steps:

1. Invitation to consulting engineers to submit proposals for preparing an engineering report or appointment of a consultant to the regular or expanded engineering staff of the community.
2. Engagement of a consultant on evidence of his or her qualifications and not by competitive bidding.
3. Examination of the consulting engineering report and its acceptance or rejection. If the report is not accepted, step 1 may have to be repeated.
4. Authorization of the preparation of plans and specifications. The consultant responsible for the report, and other engineers, too, may be asked to submit proposals for doing the work under the stipulations of step 2.
5. Advertisement for construction contracts after the plans and specifications have been accepted. Local newspapers and national engineering journals are used for this purpose.
6. Selection of the contractor, generally on the basis of the lowest bid received from a technically qualified and financially responsible construction company.
7. Construction under supervision of a resident engineer, employed by the owner (the municipality or district) or the consulting engineer.
8. Acceptance of the completed works on the recommendation of the engineer. The consultant may be retained to advise and assist in putting the system into operation.

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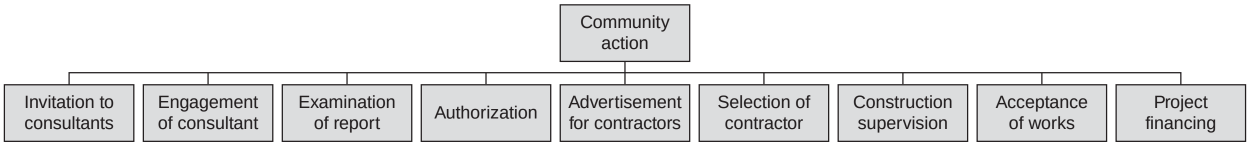

Figure 24.1 Steps for community action (*Source*: US EPA).

9. Examination and adoption of project financing. This is done at a suitable stage of project development, often with the assistance of the engineering consultant or a financial consultant.

24.2.2 Engineering Response

The engineering response for water and wastewater works project development is summarized in Fig. 24.2 and in the following list:

1. Collection and evaluation of available and required basic information: demographic, hydrologic, geologic, topographic, and industrial. The advice of local engineering practitioners who know the community and region may be useful.
2. Preparation of a preliminary or feasibility report, or of a final engineering report.
3. Preparation of plans and specifications, if the report is accepted and the detailed design authorized. Basing his or her decision on needed and available materials, the engineer will normally discuss the ideas with qualified manufacturers and suppliers.
4. Preparation of contract documents, including an estimate of construction costs after the plans and specifications have been accepted.
5. Assistance in advertising for bids and selecting the successful bidder.
6. Supervision of construction to make sure that the contractor performs the work in accordance with the plans and specifications. Approval of necessary shop drawings supplied by the contractor.
7. Authorization of payment when the work has been completed and preparation of as-built plans to record the construction work.
8. Preparation of an operating manual for the system and supervision of its operation during its early years, most often when the community does not have experienced operating personnel of its own.
9. Assistance in setting up accounting procedures and establishing appropriate rates for service.

24.3 THE ENGINEERING REPORT

Engineers are known by the quality of their reports. They are judged by the performance of completed designs. Good reports have opened up new channels of learning. Good designs and constructions have created new technologies. Of the many outstanding examples, some are allied to consultants, others to special commissions, and others to governmental agencies at all levels.

Regardless of whether the community does its own engineering work or engages a consultant to do it, its water and wastewater schemes should receive the benefit of well- documented studies of feasible projects before detailed designs are authorized. This maxim of sound engineering practice and good governmental procedure should be honored.

Engineering reports commissioned for the purpose of identifying the need for new or expanded water and wastewater constructions and offering acceptable proposals for their development are expected to state their mission clearly; analyze and summarize available and needed data; assess the technical, economic, legal, and political feasibility of projected works; offer alternative answers to the questions asked and point out the one or two most suitable replies; estimate costs; investigate methods of financing; and, as a result of the preceding studies, lay a firm base for the recommendations made and the execution of a feasible scheme.

If the report is well written, its purpose will be understood by lay boards as well as by fellow engineers. If its findings, conclusions, and recommendations are carefully worded, they can be quoted verbatim in news releases and give the public the information to which it is entitled. Bonds funding the proposed works can then receive justifiable support. If the report is imaginative and exhaustive, it should become the document of reference for future studies and further planning and development of the water resource. If the report is accepted, it should allow for the orderly and economic acquisition of needed rights and properties, the

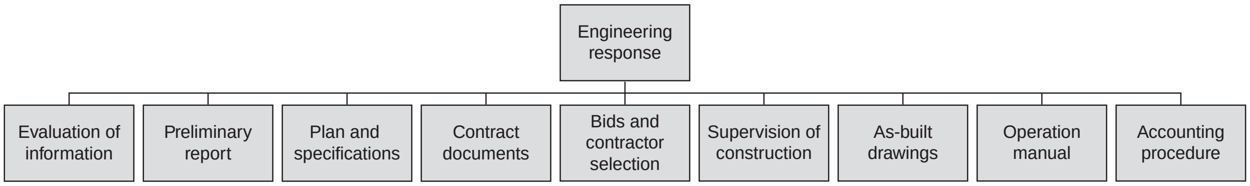

Figure 24.2 Steps for engineering response (*Source*: US EPA).

Table of Contents

(TOC)

- ➢ Letter of transmittal, addressed to the responsible agency of government
- ➢ Letter of authorization, from the agency to the engineer
- ➢ Summary of findings, conclusions, and recommendations
- ➢ Detailed report:
  1. Purpose and scope of the report
  2. The community, its geography and people, its history and expectations
  3. Existing water or wastewater works and their historical development
  4. Population: past, present, and probable future population and population density
  5. Report on water supply system:
     - a) Water use—domestic, fire, mercantile, and industrial, by area served and expected total
     - b) Available water sources, source development, pumping, transmission, treatment, distribution, and service storage
     - c) Project comparisons, including construction and operating costs
     - d) Recommended project
  6. Report on wastewater system:
     - a) For sanitary sewerage: wastewater production—domestic, infiltration, mercantile, and industrial by kind of area served and expected total
     - b) For stormwater drainage: rainfall and runoff—design storm and its recurrence interval by area served
     - c) Proposals for reuse/disposal, including interception of sewage from combined systems; stormwater management plans, wastewater treatment works, and outfalls
     - d) Effect of treated effluents including stormwater on receiving bodies of water
     - e) Project comparisons, including capital and operating costs
     - f) Recommended project
  7. Financing
  8. Rates
- ➢ Appendix charts and tables of basic data

Figure 24.3 Information itemized in engineering reports.

preparation and enactment of required legislation, and the exploration of obtainable financial support.

Most engineering reports are scheduled to contain the information itemized in the form of a table of contents as shown in Fig. 24.3.

24.4 FEASIBILITY STUDIES

Engineering feasibility implies both technical and economic practicability. Technical practicability is readily demonstrated in engineering studies such as those described in earlier chapters of this book. Although economic practicability can find expression in optimization procedures of benefits in relation to costs, either as ratios or as differences, it may not be possible to do so in full measure and in all circumstances. Resource developments competing for funds in developing countries and the national, regional, or basin-wide comprehensive planning of industrially advanced societies, for example, may create so many constraints or introduce so many decision variables that true optimization is left in suspense. Moreover, it is difficult to attach monetary values to intangible benefits. Recreation is a notable example.

Engineering feasibility may also be encumbered by legal and political restrictions. The fact that surface waters are common bounds between states or provinces and that large rivers may cross national and international boundaries in their course toward the sea may add political as well as legal constraints of much significance. The Congress of the United States may be asked to lift some of them through interstate and international compacts. However, legal contests between states are not unknown. The federal courts, in which interstate cases are tried, have handed down far-reaching decisions. In the United States, the *common law* or *riparian doctrine* governing water use in the eastern portion of the country and the *doctrine of prior use* or *prior appropriation* prevailing in the western portion may add legal constraints. They, too, may require interpretation by the courts.

24.5 ALTERNATIVES

There is no single solution to a given water resource problem, and development of the best solution is not a matter of objective optimization alone. Community and regional needs and wishes must be taken into account. To be meaningful, decisions must identify comparative advantages and disadvantages of promising alternative schemes. The community is then free to make its own choice. The most immediately economical system may not be the most acceptable system.

A recurrent example is the common preference for naturally clean water rather than water made clean by treatment. In many instances, upland supplies reaching the city by gravity meet the specification of naturally clean waters; supplies pumped from polluted rivers coursing past the city and purified in treatment works before delivery to its distribution system are descriptive of the second. Of the two, upland water supplies are usually more costly to develop, but their cost of operation, maintenance, and repair may be smaller because of differences in power and treatment requirements. Aesthetic imponderables become decision variables in cases such as this. Quite different is the example of large industrial users who must make their choice between tying into a public supply and developing one of their own. Entering into their decisions are the economy of lower interest rates available to public bodies, the economy of scale, and possible advantages of more useful water temperatures where surface and groundwater temperatures are in competition.

24.6 PLANS AND SPECIFICATIONS

More expensive and time-consuming than an engineering report, but proportionately more remunerative to the engineer, are the detailed planning and specification of works to be built. To ensure a meeting of the minds of the owner, the designer, and the contractor, plans and specifications must be comprehensive as well as precise. Vague and conflicting documents create confusion and increase bids as well as actual costs. Indeed, they may be responsible for unsatisfactory constructions.

In the course of their careers, most engineers develop specific interests and capacities. If these are given recognition in design offices—and it is generally advantageous to do so—design assignments can be based on competencies in basic engineering, structural, mechanical, and electrical elements, for instance, or on competencies in specific system components such as pipelines, pumps, filters, and other treatment units. For major projects, a team leader carries the responsibility and should be given the discretion of work assignments and the proper timing and coordination of effort by members of the team. Specifications may be compiled from individual statements, or they may be written by a separate group.

Preliminary reports, by contrast, are usually tasks performed by one or two senior engineers who generally tackle the essential subjects in sequence.

24.7 SOURCES OF INFORMATION

A busy office must be supported by a good library. A sizable engineering organization may employ a professional librarian. Professional organizations develop standards.

The shelf list of a working library usually includes the references shown in Fig. 24.4.

24.8 STANDARDS

Undoubtedly, standards have done much to improve the performance of water and wastewater works, but the underlying philosophy continues to be questioned. Standardization of pipes and other equipment for water and wastewater systems normally reduces the cost of the standard items. Together with standardization of materials of construction it simplifies not only design but also construction and the procurement of materials that meet minimum criteria and ensure compatibility. Standards of water quality set a goal to be reached in the

Contents of shelf library

➢ Engineering manuals, texts, and serial publications
➢ Standards and specifications of professional organizations. Examples are standards published by
    ✓ The American Water Works Association
    ✓ The American Society for Testing and Materials
➢ Manuals of engineering practice and design, among them those of
    ✓ The American Society of Civil Engineers
    ✓ The American Water Works Association
    ✓ The Water Environment Federation
➢ National, state, and local building and electrical codes
➢ Handbooks of associations of manufacturers:
    ✓ The Ductile Iron Pipe Research Association
    ✓ The American Concrete Pipe Association
    ✓ The Clay Sewer Pipe Association
    ✓ The Hydraulic Institute
    ✓ The Portland Cement Association
➢ Handbooks of plastic pipe manufacturers
➢ Catalogs of equipment manufacturers
➢ Reference annuals published by trade magazines

Figure 24.4 Shelf list of references in a working library.

protection of waters for the many purposes they serve. Design standards of regulatory agencies are generally written for the protection of the communities. Their aim is to promote the successful construction of water and sewerage works. To this purpose, they may specify minimum sizes and strengths of water and wastewater pipes and minimum velocities of flow in sewers, for example.

The standards of small water and sewerage works should not be applied to works of all sizes. Otherwise they may impede the development and adoption of new processes and fruitful introduction of new ideas. The engineering profession should see to it that new enterprise is not obstructed because it happens to be in conflict with existing standards. Opportunities for large-scale experimentation should be kept open. Examples of obsolescent requirements are (a) rules and regulations against curved sewers of small diameter in residential developments and (b) rules and regulations governing the maximum allowable spacing of manholes in sanitary sewers.

There can be no quarrel with the importance of health and safety standards. Standards of this kind protect water from gathering ground to points of use, maintain adequate capacity and pressures, and introduce standby power and water reserves for use in emergencies. In similar fashion, standards of this kind introduce safeguards into wastewater systems from points of collection to their disposal grounds, prevent surcharge of sewers and resulting flooding of basements and low-lying areas, and protect receiving waters.

24.9 DESIGN SPECIFICATIONS

Two extremes for specifying the work to be performed by a contractor are exemplified by (a) the *turnkey project* or *performance specification* and (b) the *descriptive specification*. Where performance is specified, the contractor becomes responsible for both design and construction. This is common practice in industry and in many developing countries. Where descriptive specifications underlie design, the owner's engineer specifies what the work is to be, and constructions are expected to perform properly if the specifications are met. As equipment becomes more complex, there is a tendency to shift to performance specifications. The engineer then becomes a coordinator of devices and relinquishes some of his or her responsibility in design.

Performance specification has a place when mass production is of benefit to the purchaser of the product. It is justified, too, when performance can be pretested, or when it is possible to test equipment after installation and to replace it if it does not meet specified performance. Pumps are examples of equipment that is generally selected on the basis of performance specifications. However, pump performance specifications are usually supplemented by descriptive specifications that provide protection against overload and ensure compatibility with other elements of the system. Performance specifications for a pumping station rather than for pumps only, and for an entire treatment plant or even for individual units of a treatment plant, are rarely appropriate. The claim that performance specifications save money because the engineer need not prepare detailed designs is valid only when standardized shelf items can be incorporated in the projected system. When this is not so, the owner, in fact, bears not only the direct cost of design but also the hidden cost of turnkey or other projects prepared for bids that were not successful.

When a descriptive specification interferes with competitive bidding by suppliers of material or equipment that will serve the design purpose equally well, the engineers normally prepare two or more alternative designs and invite bids on them.

24.10 PROJECT CONSTRUCTION

Public construction projects are generally required by law to go to the low bidder. Awards can be withheld only on evidence that a contractor does not possess the qualifications and financial backing to undertake and complete the project successfully.

Documents formulated by the engineer and used by the contractor in preparing bids are given below.

24.10.1 Notice to Bidders

The notice to bidders advertises the project and tells where copies of plans and specifications can be inspected and how they can be obtained.

24.10.2 General Conditions

The general conditions document is a statement of the conditions under which the contract is to be performed. Normally, it includes information on the following matters:

1. *Proposal requirements and conditions.* These specify the conditions under which proposals will be received, requirements for bond, necessary qualifications of bidders, the basis for disqualifying bidders, and conditions for the employment of subcontractors.
2. *Award of contract.* In the case of public agencies a statement says that if the contract is awarded, it will go to the lowest responsible bidder.
3. *Contract terms.* To ensure a meeting of the minds of the contracting parties, the terms used in the contract document are defined, and the authority of the resident engineer, management of extra work orders, and general rules for interpreting plans and specifications are stated.
4. *Bonds and insurance.* Bonding requirements and guarantees against defective workmanship are specified together with types of insurance demanded of the contractor.

5. *Responsibilities and rights of the contractor.* The specific responsibilities of the contractor on the project are stipulated. As a rule the contractor must give assurance to the owner that he does not become liable for patent infringement when patented equipment is installed.
6. *Responsibilities and rights of the owner.* The owner furnishes property surveys and, through his or her engineers, necessary baselines and benchmarks for the work to be done. He or she specifies the conditions under which he or she or the engineers have the right to inspect the operation and the mechanics of issuing changes in work. Although the engineer is employed by the owner, he or she is made the final judge in disputes between owner and contractor. However, the contractor is given a right of appeal for arbitration.
7. *Workmanship and materials.* The general basis for controlling the quality of materials and equipment is stated. The contractor must submit shop drawings, lists of materials, and other required information in ample time for review by the engineer prior to their incorporation into the construction. The contractor is generally required to field-test equipment after its installation to ensure its proper operation. In turn, the contractor may require the equipment supplier to direct the installation of specialized equipment and to supervise its initial operation.
8. *Prosecution of work.* Conditions governing the time for completion of the work are covered. Except in unusual cases there is no penalty for late compliance with the contract. However, associated costs of engineering and inspection may have to be reimbursed by the contractor. Specification of damages for delays generally raises the bid prices for contracts.
9. *Payments.* The contractor is paid periodically for work actually performed and materials brought to the construction site. A small share of the total, normally 10%, is held against project completion and final acceptance. Methods of payment for extra work and work omitted are stipulated.

24.10.3 Special Provisions

To apply to a particular project, the special provisions may include requirements for the continuation of existing services while construction is under way, for particular methods of construction the engineers believe should be followed in executing the work, and for the specific scope of work.

24.10.4 Detailed Specifications

The detailed specifications comprise the bulk and most used section of the document. Although some specifications are not changed significantly from project to project (e.g., specifications for concrete, steel, and certain kinds of pipe), most of the detailed specifications have reference only to the project for which they have been written. They are essential companions of the plans. Neither plans nor detailed specifications are self-sufficient.

The detailed specifications include such items as site preparation, demolition of existing structures, earth excavation, fill and backfill, rock excavation, preparation of foundations, embankments, paving, concrete, reinforcing steel, piping, drain piping, gates, valves, meters, specific items of equipment, metal work, painting, plumbing, heating and ventilating, electrical work, fencing, and final grading and surfacing or seeding, and planting.

24.10.5 The Proposal

The contractor's proposal, accompanied by a certain percentage of the total bid price, constitutes his or her bid. Ordinarily he or she quotes a price for each category of materials or equipment detailed in the specifications. Some of them are unit prices, for example, dollars per acre for clearing and grubbing; per cubic yard for concrete, excavation, and fill; per linear foot for piping; per square foot for paving; and *lump sum* for items of equipment. Where unit prices are requested, the engineer lists the estimated quantities. In a conflict between actual quantities and quantities listed in the proposal, the actual quantity is paid for. The more precise the engineer can make the estimates and the more extensively he or she uses unit prices, the lower the total bid is likely to be.

In arriving at cost estimates, both engineer and contractor spend much time in measuring and calculating quantities—the engineer to make sure that the bid is compatible with the budget of the client; the contractor to decide on the bids. Itemized estimates cover bulk and finished concrete, brick, painting, steel, pipe, paving, trim, planting, and many others in seemingly endless flow. Bid prices normally reflect the local situation of construction and employment.

Written too tightly, specifications may lose the advantages of competitive bidding among equipment suppliers. Written too loosely, they may allow unsatisfactory equipment to be installed. To protect the owner, the engineer may either require the bidder to identify the equipment on which the bid is based or exclude equipment from price competition. This lets the successful bidder and the engineer select the equipment after the contract has been awarded.

With the proposal, the bidder is asked to supply information on his or her experience, equipment available for constructing the project, names and qualifications of persons who will have responsible charge of the work, and personal financial resources.

24.10.6 The Contract

The contract is the agreement signed by representatives of the owner and contractor. A contract bond serves as a guarantee of performance, quality of materials, and workmanship. Normally it remains in force for 12 months beyond final acceptance of the project.

24.11 PROJECT FINANCING

Funds for the construction of major water or wastewater systems are usually borrowed. Loans normally stipulate how funds will be obtained for their repayment and for meeting other continuing obligations of the enterprise. Interest payments and operation, maintenance, and replacement (OMR) costs are examples (see Fig. 24.5).

1. *Capital costs* are the costs of the project from its beginning to the time the works are placed in operation. Included are (a) the purchase of property and rights-of-way, (b) payments for equipment and construction and for engineering and legal services, and (c) interest charges during construction. For this phase of the undertaking, money must be borrowed on short-term bond anticipation notes.
2. *Fixed charges* are the annual charges made to repay capital costs, both interest and principal, together with applicable taxes.
3. *Amortization* is the serial repayment of principal.
4. *Principal* (P) is the amount borrowed. *Repayment* (R) of principal is a part of the fixed charges.
5. *Interest* (i) is the cost of borrowing money. It is a function of the un-repaid principal and is expressed as a percentage per year. Like repayment, it is part of the fixed charges.
6. *OMR costs* include the expenditures for operation of the works, their maintenance and repair, the replacement of equipment in the normal course of operation, and minor normal extensions.
7. *Annual charges* comprehend the sum of fixed charges and OMR costs. Two or more dissimilar alternative projects are often compared on the basis of annual charges because each one must be paid for from taxes, special assessments, service charges, or commodity rates.
8. *Future value* is a function of P, i, and n (number of years for the loan). Calling FV the value of a single payment after n years, $\text{FV} = P(1 + i)^n$. A payment or loan of USD 1,000 accumulating interest at 4% annually, for instance, has a future value or repayment requirement of USD 1,480 after 10 years.
9. *Annuity* or uniform series future worth is a function of R, i, and n. If A is the value of the annuity or series of n annual end-of-the-year investments, $A = (R/i)[(1 + i)^n - 1]$. For example, USD 1,000 invested each year for 10 years at an interest rate of 4% compounded annually has a future value of USD 12,000.
10. *Sinking fund* is a fund established to retire a debt in a series of equal payments R to provide an amount A in n years. It is a function of A, i, and n, namely, $R = Ai/[(1 + i)^n - 1]$. For example, a sinking fund of USD 12,000 after a 10-year period at 4% interest compounded annually is built up by an annual investment of USD 1,000.
11. *Discounting* describes the practice of reducing future costs of benefits to an equivalent present value PV. As a function of A, i, and n, it provides common ground for estimating alternative projects by bringing them to a common present date, that is, $\text{PV} = A/(1 + i)^n$. The present worth of USD 1,480 paid at the end of 10 years is USD 1,000, for instance, if the interest rate is 4% compounded annually.
12. *Discounted series payment* determines the present value, or discounts the value of a series of equal annual future payments in terms of R, i, and n, or $\text{PV} = (R/i)[(1 + i)^n - 1]/(1 + i)^n$. Thus the present worth of USD 1,000 to be repaid each year for 10 years with interest at 4% is USD 8,111; that is, this amount invested at 4% interest compounded annually over a 10-year period would produce an income or payments of USD 1,000 a year for each of the 10 years.
13. *Capital recovery factor* is the annual payment, including both principal and interest, necessary to amortize debt A in n years at an interest rate of i, that is, $R = Ai(1 + i)^n/[(1 + i)^n - 1]$. To repay a loan of USD 8,111 by a series of payments each year over a period of 10 years, for example, requires equal annual payments of USD 1,000, if the annual interest rate is 4%.

Tables of these factors for various periods of time and different rates of interest are found in most engineering handbooks.

Project financing
Capital costs
Fixed charges
Amortization
Principal
Interest
Operation, maintenance, and replacement costs
Annual charges
Future value
Annuity
Sinking fund
Discounting
Discounted series payment
Capital recovery factor

Figure 24.5 Obtaining funds and their repayment.

EXAMPLE 24.1 FUTURE VALUE CALCULATION

What is the future value or a single repayment requirement after 5 years for a USD 20,000 loan (such as a municipal bond) with a 3% annual interest rate?

Solution:

$$\text{FV} = P(1+i)^n.$$

$$\text{FV} = 20{,}000(1+0.03)^5 = \textbf{USD 23,180.55}.$$

EXAMPLE 24.2 ANNUITY OF UNIFORM SERIES

What is the future worth of USD 20,000 invested by a city each year for 5 years at an interest rate of 3% compounded annually?

Solution:

$$A = (R/i)[(1+i)^n - 1].$$

$$A = (20{,}000/0.03)[(1+0.03)^5 - 1] = \textbf{USD 106,180.27}.$$

EXAMPLE 24.3 SINKING FUND TO PAY A CONSULTANT

With a mutual agreement, a city is to retire a debt to a consulting engineer by establishing a sinking fund of USD 24,000, for a 3-year period at 4% interest rate compounded annually. What is the annual amount the city must put aside to retire the debt after 3 years?

Solution:

$$R = Ai/[(1+i)^n - 1].$$

$$R = 24{,}000 \times 0.04/[(1+0.04)^3 - 1] = \textbf{USD 7688.36}.$$

The annual amount that the city must put aside is **USD 7688.36**.

EXAMPLE 24.4 PRESENT VALUE FOR COST COMPARISON

Two equally qualified consulting engineering firms, A and B, are bidding for a city's water works project, and both agree to be paid in full after the design project is completed. The engineering firm A can complete the project in 2 years and will charge USD 29,000. The engineering B firm can complete the project in 1 year and 6 months, and will charge USD 31,000. Which engineering firm's service charge is cheaper in terms of present values, assuming the city has plenty of time to wait, so the completion time is not a factor, and the steady interest rate is 7%?

Solution:

The present value of firm A's service charge:

$$\text{PV} = A/(1+i)^n = 29{,}000/(1+0.07)^2 = \text{USD } 25{,}329.$$

The present value of firm B's service charge:

$$\text{PV} = 31{,}000/(1+0.07)^{1.5} = \text{USD } 28{,}008.$$

Assuming the completion time is not a factor, **firm A's service is lower than that of firm B.**

EXAMPLE 24.5 DISCOUNTED SERIES PAYMENT

Determine the present worth amount, which is invested at 4% interest compounded annually over a 10-year period and will produce an income or payments of USD 50,000 a year for each of the 10 years.

Solution:

$$\text{PV} = (R/i)[(1+i)^n - 1]/(1+i)^n.$$

$$\text{PV} = (50{,}000/0.04)[(1+0.04)^{10} - 1]/(1+0.04)^{10}$$

$$= \textbf{USD 405,400}.$$

EXAMPLE 24.6 CAPITAL RECOVERY FACTOR

The city has obtained a federal loan of USD 500,000 for a water works renovation project. Determine the equal annual payments that will be necessary to amortize and repay the debt in 10 years at an interest rate of 4%.

Solution:

$$\begin{aligned} R &= Ai(1+i)^{10}/[(1+i)^n - 1] \\ &= 500{,}000\,(1+0.04)^{10}/[(1+0.04)^{10} - 1] \\ &= \mathbf{USD\,61{,}666}. \end{aligned}$$

The city must repay the federal government **USD 61,666 annually.**

24.12 METHODS OF BORROWING

For relatively simple and straightforward projects, the engineer advises on the most suitable methods of borrowing needed funds. Where funds are to be derived from several sources or for large projects, special financial advice may be sought. The methods of borrowing depend on the resources of the borrower, the sources from which funds can be borrowed, regulations of appropriate government agencies, and the nature of repayment arrangements. Borrowing is arranged through the sale of bonds. In the United States, the income from municipal bonds is not taxable. Bonds are of three general types, as discussed in the following sections.

24.12.1 General Obligation Bonds

General obligation bonds generally carry the lowest interest rates because they are backed by the full faith and credit of the community, with income generally derived from ad valorem taxes on property. When the bonds are to be repaid over a period of years, they are called serial bonds.

24.12.2 Revenue Bonds

Revenue bonds are based on repayments earned from the sale of water or from sewer service charges (sometimes called sewer rentals). The revenue bonds of an enterprise with a history of good management may carry as low an interest rate as general obligation bonds. New projects or projects for which the quality of management is uncertain may have to pay high interest rates.

24.12.3 Special Assessment Bonds

Special assessment bonds, like general obligation bonds, are backed by the value of the property they serve. They are generally short-term bonds that are normally designed to permit borrowing for a specific project serving only part of the community. Examples are lateral sewers or booster pumping stations. Only rarely are two or three types of bonds issued in combination. Reasons for combining bonds may be the allocation of charges for service in some relation to benefits received or, more pragmatically, the avoidance of legal limits on bonded indebtedness.

24.13 RATE MAKING

The principle adopted as a guide in financing and rate-making schemes has been stated by ASCE (1951) usefully as follows: “The needed total annual revenue of a water or sewage works shall be contributed by users and nonusers (or by users and properties) for whose use, need, and benefit the facilities of the work are provided, approximately in proportion to the cost of providing the use and the benefits of the works.”

On the one hand, payments for water supply and wastewater removal obtained solely through general taxation would be inequitable, because property owners would be assessed for these goods and services in full and irrespective of use. On the other hand, payments derived solely from the sale of water and through it from sewer-use charges would also be inequitable because properties benefiting from the availability of water or drainage facilities would not contribute to any part of system costs. Even if the cost of delivering water or collecting wastewater could be ascertained exactly for each water consumer or owner of property, it would be impractical to base a rate structure on such changeable information. Instead, the rate maker generally resorts to the mean; that is, he or she arrives at an approximation by averaging costs within categories of users of the same general kind. Should inequity be proved for one or more of the categories, imbalance can be righted when rates are next adjusted.

Understandably, rates must bring in sufficient income to cover fixed charges, normal OMR costs, and the cost of reasonable improvements. In some instances they may be designed to also provide a modest reserve for normal expansion of the system. Too large a reserve would place an unfair burden on current users; too small a reserve would entail frequent and expensive bond issues.

24.13.1 Water Rates

Water rates are normally structured according to the classes of consumers served and their water uses. Common classifications include manufacturing or wholesale, commercial, or intermediate, and residential or domestic users, with each category being subdivided according to rate of draft. A first or *minimum block* of charges covers the cost of metering and meter reading and of billing and collecting; it is independent of the quantity of water drawn. A second or *wholesale block* of charges is assessed in direct proportion to the cost of supplying water. Rates are obtained by dividing the system costs by the volume of water delivered. Water furnished to a neighboring community that resells it to its residents is usually charged for in this way. A third block of charges allows for the addition of incremental costs to the wholesale cost; the block is divided into subclasses with individual rates.

In terms of cost alone, unit prices generally decrease for large users. However, where water is in short supply and each incremental use adds higher costs, it is not unreasonable to increase unit prices for larger users.

24.13.2 Fire Protection

Water service for fire protection bears no relationship to the amount of water used. For this reason, charges for fire service should be subtracted from the total cost of water. Fire service is charged for in various ways. If revenue is to be proportioned according to use and benefits, for example, the cost of each element of the system must be isolated and apportioned in relation to its contribution to fire protection and to general water service. Costs of a transmission main sized to carry water for fire protection plus the coincident draft, for instance, would then be divided in proportion to these flows. By contrast, no charges for water purification would be allocated to fire protection, because a negligible quantity of water is actually consumed in firefighting.

If the primary purpose of a water system is to be a water supply service, only the incremental costs necessary to equip the system for fire protection would be so assessed. To find the cost, a hypothetical system not providing fire protection would be laid out and its cost estimated. Fire protection charges would then be based on the difference in cost between the hypothetical system (without fire service) and the actual system (with fire service).

When the cost of fire service is met from general tax funds, as it usually is, the benefits of fire protection are assumed to be proportional to property values. Actually, costs of fire service range from 10% of the total cost of water for large communities to as much as 30% for small communities.

24.13.3 Peak-Flow Demands

Peak-flow costs are exemplified by larger capacities of pipelines furnishing seasonal peak-flow demands for lawn sprinkling. There is no easy way of adjusting income received to these flows. Charges are normally the same as for other water uses. In the electrical industry, by contrast, peak costs are based on demand-meter readings, and charges per unit of electricity are related to the consumer's peak-demand rate or to demand during peak periods. Although it would probably not pay to install demand meters on residential water services, their use may be justified for wholesale customers—neighboring communities, for instance. An incidental advantage would be the resulting encouragement of wholesale customers to put in service storage and equalize the system demand.

24.13.4 Sewer Service Charges

For historical reasons, sewerage systems were paid for from general taxation. However, as communities have turned to easier, although sometimes more costly, revenue financing, and as wastewater treatment has become more common, sewer service charges have been introduced with the example of water rates in mind. Inasmuch as the wastewater released to the system is a more or less uniform fraction of the water used, the service charge is often made a fixed percentage of the water bill.

However, it can be argued that well-balanced financing of a sewerage system should actually be composed of special assessments for lateral sewers, general taxation for storm sewers, or the stormwater portion of combined sewers, as well as service charges based on water-meter readings for domestic wastewater collection and disposal.

Although metering the wastewater from households may be neither practical nor necessary where the water is metered, it may be both practical and equitable to meter industrial wastewaters. Basic charges may be determined in much the same way as for water. Surcharges may be imposed when admission of the wastewater increases the cost of treatment out of proportion to their quantity. An incidental advantage to be gained from surcharges is the possible inducement of industries to reduce and alter their waste discharges by recirculation of process water and modification of manufacturing processes. Pretreatment of wastewaters before their discharge to the sewer could be either an option or a requirement depending on the industrial wastewater quality.

24.14 SYSTEMS MANAGEMENT

A good system is a flexible system. However, if ways of taking advantage of built-in flexibility are not clearly understood and put to use by operating personnel, the advantages of flexibility are lost. Only if it is operated effectively and efficiently does an otherwise well-conceived, well-designed, and well-constructed system become a credit to the community and to the participants in the project. To meet their responsibility to society in full measure, engineers should see to it that the systems they have designed accomplish their mission. Accordingly, they must be prepared to assist the community in the operation of projects as effectively as they did in their design and construction. To this purpose, consultants may be engaged by communities for introductory or continuing surveillance of systems operations and management. A manual describing the purpose and operations of each unit and the required sequence of operations for the works as a whole may be useful. So may schematic diagrams that outline available methods of control, as well as record forms, data collection sheets, and equipment and maintenance cards.

State regulatory agencies, professional organizations, and educational institutions often assist in training plant personnel and other officials in the management of water and wastewater facilities. Both technical and fiscal operations may be covered to advantage.

PROBLEMS/QUESTIONS

24.1 Outline the role of the community in the development of water and wastewater projects.

24.2 Outline the steps of the engineering response in the development of water and wastewater projects.

24.3 Discuss the basic content of an engineering report for identifying the need for the development of new water and wastewater projects.

24.4 List the items in a table of contents for an engineering report.

24.5 Engineering feasibility implies both technical and economic practicability. Discuss the problems that could be encountered when creating engineering economic feasibility studies.

24.6 An engineering firm's office must be supported by a good library. What type of references does a typical engineering firm's library include?

24.7 The design standards of regulatory agencies are generally written for the protection of small communities. Their aim is to promote the successful construction of small works. To this purpose, they may specify minimum sizes and strengths of water and wastewater pipes and minimum velocities of flow in sewers, for example. Standards of this kind should not be applied to works of all sizes. Why?

24.8 Two extremes for specifying the work to be performed by a contractor are exemplified by (a) the turnkey project or performance specification and (b) the descriptive specification. Differentiate between the two types and discuss their pros and cons.

24.9 The document on general conditions is a statement under which the contract is to be performed. What information does this statement include?

24.10 Detailed specifications are essential companions of the plans. Neither plans nor detailed specifications are self-sufficient. What items do the detailed specifications include?

24.11 Funds for the construction of major water or wastewater systems are usually borrowed. Loans normally stipulate how funds will be obtained for their repayment and for meeting other continuing obligations of the enterprise. Give 10 examples of such costs.

24.12 The methods of borrowing depend on the resources of the borrower, the sources from which funds can be borrowed, regulations of appropriate government agencies, and the nature of repayment arrangements. Borrowing is arranged through the sale of bonds. Name and explain the general types of bonds.

24.13 Water rates are normally structured according to the classes of consumers served and their water uses. What are the three common categories of consumers?

24.14 Discuss the steps and criteria for selecting a right and qualified engineering firm to plan, design, and build future municipal water and wastewater works.

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APPENDIXES 1 TO 26

WATER ENGINEERING

Hydraulics, Distribution and Treatment

Appendix 1

Nomenclature and Abbreviations

A	ampere
A	angstrom
AA	activated alumina
a.m.	before noon
A/E	architect/engineer
A/S	air to solids ratio
ABF	automatic backwash filtration
ABS	acrylonitrile butadiene styrene
ac	acre
ADT	air dissolving tube
AGC	Associated General Contractors
AI	Aggressive Index
AISC	American Institute for Steel Construction
AOPs	advanced oxidation processes
AP	action plan
ASCE	American Society of Civil Engineers
atm	atmosphere
AWWA	American Water Works Association
BAT	best available technology
bbl	barrel
BMP	best management practice
BOD	biochemical oxygen demand
Btu	British thermal unit
c	curie
cal	calorie
CCE	carbon chloroform extracts
CECS	Center for Excellence in Construction Safety
CFC	chlorofluoro carbons
Chap.	chapter
Chaps.	chapters
CI	Riddick's Corrosion Index
CII	Construction Industry Institute
CM	construction manager
cm	centimeter
CMP	corrugated metal pipe
COD	chemical oxygen demand
CWS	community water system
D	darcy
d	day, diameter
DAF	dissolved air flotation
DAFF	dissolved air flotation-filtration
DBP	disinfection by-products
DBPR	Disinfection Byproducts Regulations
DE	diatomaceous earth
DFI	McCauley's Driving Force Index
DO	dissolved oxygen
DWF	dry-weather flow
ED	electrodialysis
EDR	reverse electrodialysis
emf	electromotive force
EPS	extended-period simulation
Eq.	equation
Eqs.	equations
ERP	emergency response plan
Fig.	figure
Figs.	figures
FMEA	failure modes and effects analysis
FMECA	failure modes, effects, and criticality analysis
FOIA	Freedom of Information Act
ft	foot
ft^2	square foot
ft^3	cubic foot
g	gram
GAC	granular activated carbon
gal	gallon
gpcd	gallon per capita per day
gpd	gallons per day
gpm	gallons per minute
gpud	gallons per unit per day
GWR	Ground Water Rule
GWUDI	groundwater under the direct influence
ha	hectare
HAAs	haloacetic acids
HAA5	The 5 haloacetic acids
HGL	hydraulic grade line
h	hour
hp	horsepower
HPC	heterotrophic plate count
ICBMs	intercontinental ballistic missiles
in.	inch
IOCs	inorganic compounds

Water Engineering: Hydraulics, Distribution and Treatment, First Edition. Nazih K. Shammas and Lawrence K. Wang.

I_t	inflow at time t
IX	ion exchange
J	joule
K	hydraulic conductivity
kg	kilogram
kWh	kilowatt-hour
lb	pound
LEPC	Local Emergency Planning Committee
L	liter
Lpcd	liter per capita per day
LRAA	locational running annual average
LSI	Langelier Saturation Index
LT2ESWTR	Long Term 2 Enhanced Surface Water Treatment Rule
m	meter
m^2	square meter
m^3	cubic meter
MAC	maximum acceptable concentration
MAF	mean annual flow
MAL	maximum allowable limit
MCL	maximum contaminant level
MCLG	maximum contaminant level goal
mD	millidarcy
me	milliequivalent
MF	microfiltration
mg	milligram
MG	million gallons
MGD	million gallons per day
mi	mile
min	minute
mL	milliliter
ML	million liters
MLD	million liters per day
MLSS	mixed liquor suspended solids
MLVSS	mixed liquor volatile suspended solids
mm	millimeter
mol wt	molecular weight
mol	mole (gram-molecular weight)
mph	miles per hour
MPN	most probable number
MPY	mils per year
MRDL	maximum residual disinfectant level
MRDLG	maximum residual disinfectant level goal
MSWLF	municipal solid waste landfill
NBS	National Bureau of Standards
NCH	noncarbonate hardness
N	Newton
NF	nanofiltration
NIOSH	National Institute for Occupational Safety and Health
NIST	National Institute for Standards and Technology
NOMs	natural organic materials
NPDWRs	National Primary Drinking Water Regulations
NSDWRs	National Secondary Drinking Water Regulations
NTNCWSs	nontransient noncommunity water systems
NTU	nephelometric turbidity units
O&M	operation and maintenance
°C	degree centigrade
°F	degree Fahrenheit
°K	degree Kelven
°R	degree Rankine
OR	overflow rate
OSHA	Occupational Safety and Health Administration
O_t	outflow at time t
p.	page
p.m.	afternoon
PAC	powdered activated carbon
PACl	polyaluminum chloride
PCA	packed column aeration
PF	peaking factor
PFS	polyferric sulfate
ppmv	parts per million, volume
psi	pound per square inch
psia	pound per square inch, absolute
psig	pound per square inch, gauge
PtD	Prevention through design
PVC	polyvinylchloride
PWS	public water systems
Q	outflow or discharge
R, R_g	gas constant
RAA	running annual average
RO	reverse osmosis
rpm	revolution per minute
rps	revolution per second
RSI	Ryznar Stability Index
RTD	residence time distribution
s	second
SBR	sequencing batch reactor
SCADA	supervisory control and data acquisition
SCBA	self-contained breathing apparatus
SCD	streaming current detector
SDGS	small-diameter gravity sewer
SDI	sludge density index
SDWA	Safe Drinking Water Act
SG	specific gravity
SI	Systeme international (metric system)
SL	solid loading
SOCs	synthetic organic compounds
SS	suspended solids
SSO	sanitary sewer overflow
STEP	septic tank effluent pump
STP	standard temperature and presure
STU	Jackson turbidity units
SVI	sludge volume index
SWTR	Surface Water Treatment Rule

t	ton (English)
T	tonne (metric)
TC	total coliforms
TDS	total dissolved solids
THMFP	trihalomethane formation potential
TL_m	median tolerance limit
TOC	total organic carbon
TON	threshold odor number
TSS	total suspended solids
TT	treatment technique
TTHMs	total trihalomethanes
US DOD	US Department of Defense
US EPA	United States Environmental Protection Agency
US	United States
UF	ultrafiltration
UNIDO	United Nations Industrial Development Organization
USAF	US Air Force
USD	United States Dollar

UV	ultraviolet
VA	vulnerability assessment
VOCs	volatile organic compounds
W	watt
$W(u)$	well function of u
WHO	World Health Organization
WL	working level
WQS	water quality standards
WTP	water treatment plant
WWTP	wastewater treatment plant
yd	yard
yd^2	square yard
yd^3	cubic yard
yr	year
Δt	hydrologic time step
μ	micron
μc	microcurie
μg	microgram
μm	micrometer

Appendix 2

Units

A	absorbance at about 254 nm
A	air solubility in effluent at 1 atm pressure (mL/L)
A	debt
A	mass flow rate of air released for flotation of suspended solids (mg/s)
A	area, ft^2 (m^2)
a	area
a	continuous gutter depression
a	empirical constant for the Langmuir isotherm
A/S	air-to-solids ratio
AA_j	average age at the node immediately upstream of node j
A_c	cross-sectional area of particle perpendicular to direction of flow
AF	alkalinity factor
A_O	area of opening, ft^2 (m^2)
A_j	age of water at jth node
A_S	surface area required to reach desired sludge consistency, ft^2 (m^2)
A_jet	area of jet stream, ft^2 (m^2)
avg	average
B	constant for the BET model isotherm
B	resistance characteristics of a formation
B	width
b	constant for the BET model isotherm
b	breadth, width, thickness
b	empirical constant for the Langmuir isotherm
BG	billion gallons
BHP	break horsepower
C	coagulant concentration, mg/L
C	concentration of the disinfectant, mg/L
C	Hazen–Williams coefficient
C	runoff coefficient
c	coefficient, Chezy coefficient
c	convex routing coefficient
c	equilibrium concentration of adsorbate in solution
c	killing concentration of HOCl, mg/L
c	rate of surge propagation
c	residual threshold value
c	runoff coefficient
c	substance concentration as a function of distance and time
c_i	concentration of the gas at the interface
c_o	concentration at time $t = 0$
c_o	threshold odor or taste of the water to be treated
c_l	concentration of the gas in the main body of the liquid
C_0	orifice coefficient
$C_1 - C_2$	mineral concentration gradient across the membrane, g/cm^3
$C1$	first concentration of liquid disinfectant, %
$C2$	second concentration of liquid disinfectant, %
C_a	air released at 1 atm pressure, mg/L
C_a	average (mixed) constituent concentration
C_as	air saturation concentration at atmospheric conditions, mg/L
C_c	coefficient of contraction, dimensionless
C_c	area at v.c./area of orifice
CD	chemical disinfectant dose, mg/L (lb/MG)
C_D	drag coefficient
C_d	coefficient of discharge, dimensionless
C_e	solubility of gas in the flotation effluent, mg/L
C_e	external source concentration into node k
C_f	dissolved gas concentration in the raw influent feed stream, mg/L
C_g	gas released at 1 atm pressure, mg/L
C_gs	gas saturation concentration at atmospheric conditions, mg/L
C_i	constituent concentrations of the inflows
C_k	concentration at node k
CL	chloride concentration, mg/L
C_mp	mineral permeability constant, cm/s
C_p	mineral or impurity in permeate or product water, mg/L
C_p	concentration of permeated solids
C_r	solubility of gas in water in the pressurized retention tank, mg/L
C_R	concentration of rejected solids
C_v	coefficient of velocity, dimensionless
C_v	actual velocity at v.c./theoretical velocity at v.c.
c	celerity, ft/s (m/s)
c_s	saturation concentration of adsorbate in solution
c_s	saturation concentration of the gas in the water, mL/L

Water Engineering: Hydraulics, Distribution and Treatment, First Edition. Nazih K. Shammas and Lawrence K. Wang.

c_{sb}	saturation concentration of DO in brackish or fresh water at normal barometric pressure (760 mm Hg), any water temperature, and any chloride concentration, mg/L
c_{sf}	saturation concentration of DO in fresh water at normal barometric pressure (760 mm Hg) and any temperature, mg/L
c_{sp}	saturation concentration of DO in any water at any barometric pressure, any water temperature, and any chloride concentration, mg/L
c_t	concentration at time t
Ct_{calc}	calculated Ct value, mg/L-min
c_{u}	underflow concentration, mg/L
C_{w}	weir coefficient
C_{wp}	water permeability constant, g/cm^2/s-atm
D	density of the specimen, g/cm^3
D	diameter
D	draft
D	housing density
D	hydraulic diffusivity
D	number of homes served
D	surface detention, in. of depth
D	UV dosage, mJ/cm^2, or mW-s/cm^2
d	air density, lb/ft^3 or g/L
d	diameter, in. (mm)
d	distance or depth
d	vertical distance the particle fall through, ft (m)
d_0	effective head at the orifice
D_{c}	column diameter, in. (mm)
D_{c}	dew-point temperature
d_{c}	critical depth
D_{d}	distribution depth, ft (m)
DD	disinfectant demand, mg/L
deg	degree
d_{i}	sieved size
d_{i}	depth at lip of curb opening
D_{i}	inside diameter of pipe
D_{p}	mean particle diameter, mm
DR	disinfectant residual (chlorine residual), mg/L
D_{w}	well drawdown
E	activation energy, cal
E	applied electric field gradient, V/cm
E	effective size
E	rate of evaporation, efficiency
E	modulus of elasticity of pipe wall, lb/ft^2 (N/m^2)
E_0	efficiency
E^0	electromotive force, V
E_{motor}	efficiency of the motor driving the pump
E_{pump}	efficiency of the pump
eq/L	equivalent/liter
E_{ww}	wire-to-water efficiency
F	flux, gpd/ft^2 (L/h/m^2)
F	fraction of gas dissolution
F	freezing index
F	frequency of occurrence

$\mathbf{F}$	Froude number
f	dimensionless friction factor
f	factor of gas dissolution at pressure P, fraction
f	porosity
f	Weisbach–Darcy friction factor
f'	porosity ratio
F_{D}	drag force
f_{d}	friction head loss in the discharge piping
f_{e}	porosity ratio of the expanded sand layer
f_{e}	relative pore space of the flocculation zone
$f_{\mathrm{e}i}$	porosity ratio of the expanded ith sand layer
F_{I}	impelling force
F_{s}	mineral flux, g/cm^2/s
f_{s}	friction head loss in the suction piping
F_{w}	water flux, g/cm^2/s
G	gas released for flotation of suspended solids, mg/s
G	gravity constant
G	velocity gradient, s^{-1}
g	acceleration of gravity, m/s^2 (ft/s^2)
G_{in}	mass flow rate of dissolved gas entering the flotation chamber, mg/s
G_{out}	mass flow rate of dissolved gas leaving the flotation chamber, mg/s
H	total hardness, mol/L
H	head, height
h	curb throat opening height
h	head, height
h	loss of head, ft (m)
h	total depth of water in column
H_0	initial column height of the interface in the settling column, ft (m)
H_{a}	energy added
H_{∞}	final depth of sludge after a long period of settling, ft (m)
H_{A}	energy at Section A
H_{B}	energy at Section B
h_{c}	critical head
h_{cg}	height of center of gravity
h_{d}	static discharge head
HF	calcium hardness factor
h_{f}	head loss
H_{f}	friction head loss or energy loss
HP	power in US customary unit
h_{s}	static suction head or lift
H_{sv}	suction head
H_t	depth of sludge at any time (t), ft (m)
H_{u}	depth at which all solids are at the desired underflow concentration (c_{u}), ft (m)
I	hydraulic gradient
I	inflow or recharge
I	recurrence interval
I	saturation index
I	UV light intensity in the bulk solution, mW/cm^2
i	intensity
i	interest rate

i	slope
I_{cg}	moment of inertia of an area about its center of gravity axis
INHP	input power in US customary unit
INMP	input power in metric unit
I_{p}	intensity of pressure at the center of gravity of an area
I_{t}	inflow at time t
$I_{t-\Delta t}$	inflow at previous time step
j	pipe flowing into node k
K	constant
K	Darcy's coefficient
K	empirical constant for the Freundlich isotherm
K	equilibrium constant
K	hydraulic conductivity
K	permeability
K, K_1, K_2, K_3	Thermodynamic and operational constants
k	coefficient of proportionality or the rate constant, 1/min
k	constant, flow multiplier for time of day
k	rate constant with dimension, t^{-1}
k'	constant
k'	$-k$ log e, 1/min
k_0	initial, or starting, rate of reaction or reaction velocity, day^{-1}
k_1	rate constant, L/h-kg (gal/h-lb)
K_{b}	pipe or elbow head loss coefficient
K_{c}	contraction head loss coefficient
K_{c}	0.376 (0.56 in US units) in gutter equation
K_{c}	0.817 (0.60 in US units) in curb inlet equation
K_{cf}	0.295 (0.090 in US units)
K_{cs}	0.0828 (0.15 in US units)
k_{d}	diffusion coefficient
$k_{\mathrm{d(g)}}$	film-diffusion or transfer coefficients in the gaseous phase
$k_{\mathrm{d(l)}}$	film-diffusion or transfer coefficients in the liquid phase
K_{e}	entrance head loss coeffricient
K_{E}	expansion head loss coefficient
K_{g}	a proportionality factor
K_{h}	hydrolysis constant, mg/L
K_{i}	ionization constant, mg/L
k_{s}	solubility constant
K_{t}	tee head loss coefficient
K_{v}	valve head loss coefficient
K_{w}	ratio of length of screen to saturated thickness of an aquifer
L	depth of filter bed
L	length of tank, m (ft)
L	saturation or maximum population
l, ℓ	length
L_{c}	column length or bed depth, ft
L_{e}	thickness of sand layer
L_{T}	length of curb inlet required to capture 100% of the flow
M	mass or weight of adsorbent, kg (lb)
m	concentration of the adsorbent
m	mass of the adsorbent
m	constant in t^m
MHP	motor horsepower
MP	power in metric unit
$(Mv_x)_1$	initial linear momentum in x-direction, slug-ft/s (kg-m/s)
$(Mv_y)_1$	initial linear momentum in y-direction, slug-ft/s (kg-m/s)
$(Mv_x)_2$	final linear momentum in x-direction, slug-ft/s (kg-m/s)
$(Mv_y)_2$	final linear momentum in y-direction, slug-ft/s (kg-m/s)
N	Kutter's or Manning's coefficient of roughness
N	number
N	pump speed, rpm
n	coefficient of dilution or order of the reaction
n	coefficient of retardance
n	coefficient of roughness, Manning's coefficient
n	constant
n	number
n	pump speed, rpm
n	response coefficient of the substances to be removed or adsorbed
N_0	initial number of organisms, #
N_{s}	specific speed
N_{v}	number of valves connected to the sewage collection station
P	pressure
P	percentile
P	perimeter of the inlet
P	population in thousands
P	power, ft-lb/s (N-m/s or J/s)
P	total force
p	fractional part of a cycle in well pumping
p	monthly percentage of daylight hours in the year
p	partial pressure of the gas in the gas phase
p	percent concentration of disinfectant, %
p	pressure of saturated water vapor, mm Hg
P_0	membrane filtration unit's effluent retentate pressure, psi (kPa)
P_{10}	10 percentile
P_{60}	60 percentile
P_{a}	absolute pressure
P_{a}	atmospheric pressure
P_{atm}	atmospheric pressure
P_{d}	vapor pressure at dew-point temperature
P_{e}	gas pressure in the flotation chamber's effluent compartment, atmosphere
p_{g}	partial pressures of the gas in the main body of the gas
P_{gauge}	gauge pressure

$\mathrm{pH}_{\mathrm{eq}}$	pH at equilibrium
pH_{s}	pH at saturation
P_{i}	membrane filtration unit's influent feed pressure, psi (kPa)
p_{i}	fractional weight
p_{i}	partial pressures of the gas at the interface
P_{k}	design year population in thousands
P_{n}	future population
P_{o}	present population
P_{p}	permeate pressure, psi (kPa)
P_{p}	past population
P_{r}	gas pressure in the retention tank, atmosphere
P_{s}	absolute pressure
P_{t}	percentage of residual ozone at time t
p_{t}	the percentage of residual ozone at time t, %
P_{v}	vapor pressure
P_{w}	vapor pressure at a given water temperature
Q	flow, or discharge ft^3/s (m^3/s)
q	charge at the shear surface
q, q_{s}	flow rate and self cleaning flow
q_{n}	discharge through an inlet orifice
q_0	maximum solid-phase concentration of the sorbed solute, kg/kg (lb/lb)
$Q1$	first flow of liquid disinfectant, L/min (ft^3/min)
Q_{19}	the useful ratio t_1/t_2 for $T_2 - T_1 = 19$
$Q2$	second flow of liquid disinfectant, L/min (ft^3/min)
Q_{a}	pumping station average flow
Q_{a}	air flow rate, ft^3/min (L/min)
Q_{avg}	average daily water demand
Q_{d}	disinfectant feed rate, mg/d (lb/d)
Q_{dp}	discharge pump capacity
Q_{e}	external source flow into node k
Q_{f}	feed flow rate, gpd (L/h)
Q_i	flow rate to the jth node from the ith node
Q_i	intercepted flow by a street stormwater inlet
Q_{i0}	flow intercepted by the inlet operating as an orifice
$Q_{\mathrm{ins\ peak}}$	instantaneous peak water demand, gpm
Q_{iw}	flow intercepted by the inlet operating as a weir
Q_j	flow in pipe j
Q_{max}	collection station peak flow
Q_{min}	pumping station minimum flow
Q_{n}	portion of influent feed stream which is not pressurized, L/s
Q_{p}	permeate flow rate, gpd (L/h)
Q_{p}	portion of influent feed stream, which is pressurized, L/s
Q_{r}	retentate flow rate, gpd (L/h)
Q_{s}	side flow at street inlet
Q_t	flow at time t
Q_{vp}	vacuum pump capacity
Q_{w}	water flow rate, L/d (MGD)
Q_{w}	flow in depressed section of gutter
Q_{w}	water flow rate, L/d or MG/d
R	capital recovery factor
R	frictional resistance

R	gas constant, 1.99 cal/fC
R	hydraulic radius
R	mineral or impurity rejection, %
R	probable rate of population increase per year
R	rate of rainfall
R	rate of corrosion, MPY
R	recirculation ratio
R	rejection
R	residuals
R	**R**eynolds number
r	hydraulic radius
r	proportionate efficiency of $\mathrm{OC1}^-$ ions relative to HOC1
r	radial distance
r	radius
$R_1, R_2, \ldots R_5$	percent removals
r_{c}	permissible distance between production and disposal wells
R_{f}	the ratio of intercepted frontal flow to the total frontal flow in the gutter
R_{g}	gas constant
R_{s}	the ratio of side flow intercepted to total side flow in the gutter
R_{T}	total amount of chlorine required to produce a given percentage of kill in a specified time
R_{T}	total concentration of chlorine required to produce a given % of kill, mg/L
R_{t2}	percent removal at time t_2
r_{w}	effective well radius
S	mass flow rate of suspended solids entering the flotation system, mg/s
S	shape factor
S	storage coefficient
s	drawdown, hydraulic gradient
S'	unit storage
S_{d}	total dissolved solids
S_{e}	composite or equivalent slope in the gutter
S_i	specific gravity of ith component of sludge
SG	specific gravity
S_{L}	road longitudinal slope
SL	solid loading
S_{n}	drawdown in the pumped well
S_{t}	tensile stress in tank wall, $\mathrm{lb/ft}^2$ ($\mathrm{N/m}^2$)
s_{s}	specific gravity of particles
S_{s}	specific gravity of sludge
S_{w}	gutter cross-slope
$S_{\mathrm{w}'}$	gutter cross-slope at the inlet location measured from the pavement cross-slope, S_{x}
S_{x}	road cross-slope
T	absolute temperature, K
T	exposure time, h
T	temperature
T	total width of flow or spread in gutter
T	transmissivity

T	alkalinity
$\%T$	percent UV transmittance, %
t	process detention time, d (h)
t	thickness
t	(given value − mean)/standard deviation
t	time, s
T_a	absolute temperature
t_c	time of concentration
T_c	time of concentration
t_d	detention time, s
t_d	flow or detention time, s
TF	temperature factor
TL_m	median tolerance limit
T_m	daily maximum temperature
t_p	time required to effect a constant percentage kill of the organisms, min
t_r	time of rise, s
T_r	mean monthly temperature
t_k	tank wall thickness, in. (mm)
t_t	travel time
t_u	time required to attain underflow concentration (c_u), s
t_w	pipe wall thickness, in. (m)
U	coefficient of nonuniformity
U	water consumptive use
V	velocity in full section
V	volt
V	volume
v	velocity ft/s (m/s)
v_s	settling velocity
V_0	splash-over velocity of the inlet
v_c	critical velocity
v_d	displacement velocity, ft/s (m/s)
v_h	hindered settling velocity, ft/s (m/s)
v_i	ideal or theoretical velocity, ft/s (m/s)
V_H	horizontal velocity (m/s)
V_o	operating volume of the collection tank
V_p	permeate volume, gal (L)
v_s	settling velocity of particle
V_s	specific volume
V_s	self-cleansing velocity
V_s	volume of silt
V_t	total volume of the collection tank
V_v	volume of voids
v_w	the wind velocity, ft/s (m/s)
W	influent suspended solids load, lb/min (g/min)
W	weight
W	width
w	wind velocity
W_g	width of grate inlet
WHP	water horsepower
W_i	weight fraction of *i*th component of sludge
W_p	wetted perimeter
X	influent suspended solids concentration, mg/L
x	distance
X_f	float suspended solids concentration, mg/L
y	number of organisms destroyed in unit time, #/min
y	quantity adsorbed
y	population
y/m	quantity adsorbed by a unit weight of adsorbent
y/y_o	proportion of adsorbate removed in t days
y_{cg}	distance of center of gravity
y_{cg}	moment of inertia of the area about its center of gravity axis
y_{cp}	distance of the center of pressure along the plane from an axis located at center of gravity
y_m	midyear population
z	elevation
Z	ratio of average pumping station flow to minimum flow
α	vertical compressibility of aquifer material
α	angle
α	coefficient
β	coefficient, compressibility and bulk modulus of elasticity
Δs	change in drawdown
Δs	change in storage volume
Δt	change in time
$\Delta h_1, \Delta h_2, \ldots, \Delta h_4$	depth increments to successive percent removal curves at time t_2
Δt	hydrologic time step
Δv	change in fluid velocity, ft/s, (m/s)
ΔO_s	osmotic pressure differential across the membrane, atm
ΔP	pressure drop, psi (kPa)
ΔP_m	pressure differential applied across the membrane, atm
ΔP_T	transmembrane pressure (TMP), psi (kPa)
δ	*thickness of diffuse layer*
ε	a measure of absolute roughness
ε	*dielectric constant*
γ	specific weight
γ_s	specific weight of solid particles
θ	inclination of the curb throat measured from the vertical direction
θ	inclination angle of tubes
θ	porosity, angle
$\theta(c)$	substance rate of reaction within the link
μ	dynamic (absolute) viscosity of water, Pa-s, N-s/m^2, kg/m-s (lb-s/ft^2)
μ	measure of central tendency, mean value

μ_g	geometric mean size
ν	kinematic viscosity of the fluid, m^2/s (ft^2/s)
ρ	mass density of the fluid
ρg	weight density or specific weight of water = 62.4 lb/ft^3 (9,800 N/m^3)
ρ_s	mass density of the particle
$\sum$	sum
$\sum F$	sum of linear impulse forces, lb (N)
$\sum F_x$	sum of linear impulse forces in x-direction, lb (N)
$\sum F_y$	sum of linear impulse forces in y-direction, lb (N)
$\sum Q_{in}$	total flow into the node
$\sum Q_{out}$	total flow out of the node
σ	standard deviation and surface tension, lb/ft (dyne/cm, N/m)
σ_g	standard deviation
τ	drag or tractive force
τ	shear stress at the liquid–sludge interface
ψ	sphericity
ζ	zeta potential

Appendix 3

Viscosity, Density, and Other Physical Properties of Water

Temperature, (°C)	Density ρ, γ (grams/cm^3)	Absolute viscosity μ, (centipoises)[a]	Kinematic viscosity ν, (centistokes)[b]	Temperature, (°F)
0	0.99987	1.7921	1.7923	32.0
2	0.99997	1.6740	1.6741	35.6
4	1.00000	1.5676	1.5676	39.2
6	0.99997	1.4726	1.4726	42.8
8	0.99988	1.3872	1.3874	46.4
10	0.99973	1.3097	1.3101	50.0
12	0.99952	1.2390	1.2396	53.6
14	0.99927	1.1748	1.1756	57.2
16	0.99897	1.1156	1.1168	60.8
18	0.99862	1.0603	1.0618	64.4
20	0.99823	1.0087	1.0105	68.0
22	0.99780	0.9608	0.9629	71.6
24	0.99733	0.9161	0.9186	75.2
26	0.99681	0.8746	0.8774	78.8
28	0.99626	0.8363	0.8394	82.4
30	0.99568	0.8004	0.8039	86.0

[a] 1 centipoise = 10^{-2} g mass/cm s   lb force s/ft^2 = $2.088 \times 10^{-5} \times$ centipoise.

[b] 1 centistoke = 10^{-2} cm^2/s   ft^2/s = $1.075 \times 10^{-5} \times$ centistokes.

Physical Properties of Water in English Units

Temp. (°F)	Specific weight, γ (lb/ft^3)	Density, ρ (slugs/ft^3)	Absolute Viscosity, $10^{-5}\mu$ (lb · s/ft^2)	Kinematic viscosity, 10^{-5} ν (ft^2/s)	Surface tension, 100 σ (lb/ft)	Vapor-pressure head, p_v/γ (ft)	Bulk modulus of elasticity, 10^3 β(lb/in^2)
32	62.42	1.940	3.746	1.931	0.518	0.20	293
40	62.43	1.940	3.229	1.664	0.514	0.28	294
50	62.41	1.940	2.735	1.410	0.509	0.41	305
60	62.37	1.938	2.359	1.217	0.504	0.59	311
70	62.30	1.936	2.050	1.059	0.500	0.84	320
80	62.22	1.934	1.799	0.930	0.492	1.17	322
90	62.11	1.931	1.595	0.826	0.486	1.61	323
100	62.00	1.927	1.424	0.739	0.480	2.19	327

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Temp. (°F)	Specific weight, γ (lb/ft^3)	Density, ρ (slugs/ft^3)	Absolute Viscosity, $10^{-5}\mu$ (lb · s/ft^2)	Kinematic viscosity, 10^{-5} ν (ft^2/s)	Surface tension, 100 σ (lb/ft)	Vapor-pressure head, p_v/γ (ft)	Bulk modulus of elasticity, 10^3 β(lb/in^2)
110	61.86	1.923	1.284	0.667	0.473	2.95	331
120	61.71	1.918	1.168	0.609	0.465	3.91	333
130	61.55	1.913	1.069	0.558	0.460	5.13	334
140	61.38	1.908	0.981	0.514	0.454	6.67	330
150	61.20	1.902	0.905	0.476	0.447	8.58	328
160	61.00	1.896	0.838	0.442	0.441	10.95	326
170	60.80	1.890	0.780	0.413	0.433	13.83	322
180	60.58	1.883	0.726	0.385	0.426	17.33	313
190	60.36	1.876	0.678	0.362	0.419	21.55	313
200	60.12	1.868	0.637	0.341	0.412	26.59	308
212	59.83	1.860	0.593	0.319	0.404	33.90	300

Physical Properties of Water in SI Units

Temp. (°C)	Specific weight, γ (N/m^3)	Density, ρ (kg/m^3)	Absolute Viscosity, 10^{-3} μ (N · s/m^2)	Kinematic viscosity, 10^{-6} ν (m^2/s)	Surface tension, 100 σ (N/m)	Vapor-pressure head, p_v/γ (m)	Bulk modulus of elasticity, 10^7 β(N/m^2)
0	9,805	999.9	1.792	1.792	7.62	0.06	204
5	9,806	1,000.0	1.519	1.519	7.54	0.09	206
10	9,803	999.7	1.308	1.308	7.48	0.12	211
15	9,798	999.1	1.140	1.141	7.41	0.17	214
20	9,789	998.2	1.005	1.007	7.36	0.25	220
25	9,779	997.1	0.894	0.897	7.26	0.33	222
30	9,767	995.7	0.801	0.804	7.18	0.44	223
35	9,752	994.1	0.723	0.727	7.10	0.58	224
40	9,737	992.2	0.656	0.661	7.01	0.76	227
45	9,720	990.2	0.599	0.605	6.92	0.98	229
50	9,697	988.1	0.549	0.556	6.82	1.26	230
55	9,679	985.7	0.506	0.513	6.74	1.61	231
60	9,658	983.2	0.469	0.477	6.68	2.03	228
65	9,635	980.6	0.436	0.444	6.58	2.56	226
70	9,600	977.8	0.406	0.415	6.50	3.20	225
75	9,589	974.9	0.380	0.390	6.40	3.96	223
80	9,557	971.8	0.357	0.367	6.30	4.86	221
85	9,529	968.6	0.336	0.347	6.20	5.93	217
90	9,499	965.3	0.317	0.328	6.12	7.18	216
95	9,469	961.9	0.299	0.311	6.02	8.62	211
100	9,438	958.4	0.284	0.296	5.94	10.33	207

Note: Mathematical models of all above water properties can be found in the following U.S. government report: Wang, L. K., Wang, M. H. S., and Terranova, D., Development of Water Property Models for Water Quality Control, U.S. Dept. of Commerce, National Technical Information Services, Springfield, VA, PB80-153224, 1980, 53 p.

Appendix 4

Vapor Pressure and Surface Tension of Water in Contact with Air

Temperature (°C)	0	5	10	15	20	25	30
Vapor pressure (P_w) (mm Hg)[a]	4.58	6.54	9.21	12.8	17.5	23.8	31.8
Surface tension (σ) (dyne/cm)[b]	75.6	74.9	74.2	73.5	72.8	72.0	71.2

[a] To convert to in. Hg divide by 25.4.

[b] To convert to (lb force) /ft divide by 14.9.

$P_w = 4.571512 + 0.352142T + 0.007386T^2 + 0.000371T^3$

$P_w = 4.571512$ (mm Hg) when $T = 0°C$

$P_w = 6.563247$ (mm Hg) when $T = 5°C$

$P_w = 12.767617$ (mm Hg) when $T = 15°C$

$P_w = 23.78818$ (mm Hg) when $T = 25°C$

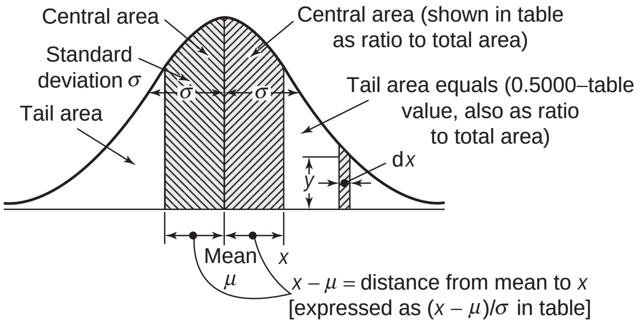
Source: Wang, L. K., Shammas, N. K., Selke, W. A., and Aulenbach, D. B. (eds.). *Flotation Technology*, Humana Press-Springer Science, NYC, New York, 2010, pp. 49–84.

Water Engineering: Hydraulics, Distribution and Treatment, First Edition. Nazih K. Shammas and Lawrence K. Wang.

Appendix 5

Area under the Normal Probability Curve

Fractional parts of the total area (1.0000) corresponding to distances between the mean (μ) and given values (x) in terms of the standard deviation (σ), that is, $(x - \mu)/\sigma = t$.



t	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.0000	0.0040	0.0080	0.0120	0.0160	0.0199	0.0239	0.0279	0.0319	0.0359
0.1	0.0398	0.0438	0.0478	0.0517	0.0557	0.0596	0.0636	0.0675	0.0714	0.0753
0.2	0.793	0.832	0.0871	0.0910	0.0948	0.0987	0.1026	0.1064	0.1103	0.1141
0.3	0.1179	0.1217	0.1255	0.1293	0.1331	0.1368	0.1406	0.1443	0.1480	0.1517
0.4	0.1554	0.1591	0.1628	0.1664	0.1700	0.1736	0.1772	0.1808	0.1844	0.1879
0.5	0.1915	0.1950	0.1985	0.2019	0.2054	0.2088	0.2123	0.2157	0.2190	0.2224
0.6	0.2257	0.2291	0.2324	0.2357	0.2389	0.2422	0.2454	0.2486	0.2517	0.2549
0.7	0.2580	0.2611	0.2642	0.2673	0.2704	0.2734	0.2764	0.2794	0.2823	0.2852
0.8	0.2881	0.2910	0.2939	0.2967	0.2995	0.3023	0.3051	0.3078	0.3106	0.3133
0.9	0.3159	0.3186	0.3212	0.3238	0.3264	0.3289	0.3315	0.3340	0.3365	0.3389
1.0	0.3413	0.3438	0.3461	0.3485	0.3508	0.3531	0.3554	0.3577	0.3599	0.3621
1.1	0.3643	0.3665	0.3686	0.3708	0.3729	0.3749	0.3770	0.3790	0.3810	0.3830
1.2	0.3849	0.3869	0.3888	0.3907	0.3925	0.3944	0.3962	0.3980	0.3997	0.4015
1.3	0.4032	0.4049	0.4066	0.4083	0.4099	0.4115	0.4131	0.4147	0.4162	0.4177
1.4	0.4192	0.4207	0.4222	0.4236	0.4251	0.4265	0.4279	0.4292	0.4306	0.4319
1.5	0.4332	0.4345	0.4357	0.4370	0.4382	0.4394	0.4406	0.4418	0.4429	0.4441
1.6	0.4452	0.4463	0.4474	0.4484	0.4495	0.4505	0.4515	0.4525	0.4535	0.4545
1.7	0.4554	0.4564	0.4573	0.4582	0.4591	0.4599	0.4608	0.4616	0.4625	0.4633
1.8	0.4641	0.4649	0.4656	0.4664	0.4671	0.4678	0.4686	0.4693	0.4699	0.4706
1.9	0.4713	0.4719	0.4726	0.4732	0.4738	0.4744	0.4750	0.4758	0.4761	0.4767
2.0	0.4772	0.4778	0.4783	0.4788	0.4793	0.4798	0.4803	0.4808	0.4812	0.4817
2.1	0.4821	0.4826	0.4830	0.4834	0.4838	0.4842	0.4846	0.4850	0.4854	0.4857
2.2	0.4861	0.4864	0.4868	0.4871	0.4875	0.4878	0.4881	0.4884	0.4887	0.4890
2.3	0.4893	0.4896	0.4898	0.4901	0.4904	0.4906	0.4909	0.4911	0.4913	0.4916
2 4	0.4918	0.4920	0.4922	0.4925	0.4927	0.4929	0.4931	0.4932	0.4934	0.4936
2.5	0.4938	0.4940	0.4941	0.4943	0.4945	0.4946	0.4948	0.4949	0.4951	0.4952

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t	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
2.6	0.4953	0.4955	0.4956	0.4957	0.4959	0.4960	0.4961	0.4962	0.4963	0.4964
2.7	0.4965	0.4966	0.4967	0.4968	0.4969	0.4970	0.4971	0.4972	0.4973	0.4974
2.8	0.4974	0.4975	0.4976	0.4977	0.4977	0.4978	0.4979	0.4979	0.4980	0.4981
2.9	0.4981	0.4982	0.4982	0.4983	0.4984	0.4984	0.4985	0.4985	0.4986	0.4986
3.0	0.4987	0.4987	0.4987	0.4988	0.4988	0.4989	0.4989	0.4989	0.4990	0.4990
3.5	0.499367		4.0	0.499968		4.5	0.499997		5.0	0.4999997

Example: For $x = 10.8$, $\mu = 9.0$, and $\sigma = 2.0$, $(x - \mu)/\sigma = 0.90$, and 0.3159 = 31.59% of the area is included between $x = 10.8$ and $x = 9.0$.

Appendix 6

Flow Velocity and Discharge Rate for Pipes Flowing Full

Velocity of flow and rate of discharge for pipes flowing full when frictional resistance is 2 ft per 1,000 (2‰) and C is 100 in Hazen–Williams formula

Diameter d (in.) (1)	Area A (ft^2) (2)	Velocity v (ft/s) (3)	Discharge Q (1,000 gpd) (4)
4	0.0873	0.96	54.3
5	0.137	1.11	97.5
6	0.196	1.24	157
8	0.349	1.49	336
10	0.546	1.71	602
12	0.785	1.92	971
14	1.07	2.12	1,380
16	1.40	2.29	2,080
18	1.77	2.48	2,830
20	2.18	2.64	3,760
24	3.14	2.97	6,060
30	4.91	3.42	10,800
36	7.07	3.83	17,500
42	9.62	4.32	26,200
48	12.57	4.60	37,300
54	15.90	4.93	50,900
60	19.64	5.29	67,200

Conversion factors: 1 in. = 25.4 mm; 1 ft^2 = 0.0929 m^2; 1 ft/s = 0.3048 m/s; 1 gpd = 3.785×10^{-3} m^3/d.

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Appendix 7

Flow Velocity and Discharge Rate for Pipes Flowing Full

Velocity of flow and rate of discharge for pipes flowing full for $\sqrt{S}/N = 1$ in Manning's formula[a]

Diameter *D* (in.)	Area *A* (ft^2)	Velocity *V* (ft/s)	Discharge *Q* (ft^3/s)	Reciprocal of velocity (l/*V*)
(1)	(2)	(3)	(4)	(5)
6	0.1963	0.3715	0.07293	2.6921
8	0.3491	0.4500	0.1571	2.2222
10	0.5455	0.5222	0.2848	1.9158
12	0.7852	0.5897	0.4632	1.6958
15	1.2272	0.6843	0.8398	1.4613
18	1.7671	0.7728	1.366	1.2940
21	2.4053	0.8564	2.060	1.1677
24	3.1416	0.9361	2.941	1.0683
27	3.9761	1.0116	4.026	0.9885
30	4.9087	1.0863	5.332	0.9206
36	7.0686	1.2267	8.671	0.8152
42	9.6211	1.3594	13.08	0.7356
48	12.5664	1.4860	18.67	0.6729
54	15.9043	1.6074	25.56	0.6221
60	19.6350	1.7244	33.86	0.5799

[a] To find *V* or *Q* for given values of *S* and *N*, multiply column (3) or (4) by $\sqrt{S}/N$. To find *S* for given values of *V* or *Q* and *N*, multiply *NV* or *NQ/A*, respectively, by column (5) and square the product.

Conversion factors: 1 in. = 25.4 mm; 1 ft^2 = 0.0929 m^2; 1 ft/s = 0.3048 m/s; 1 ft^3/s = 28.32 L/s = 0.02832 m^3/s.

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Appendix 8

Minimum Grades and Capacities of Circular Conduits Flowing Full

‰ Minimum grades and capacities of circular conduits flowing full when N is 0.013 in the Manning formula

Velocity (ft/s)		Diameter (in.)							
		6	8	10	12	15	18	21	24
2.0	S (‰)	4.89	3.33	2.48	1.94	1.44	1.13	0.923	0.775
	Q (ft^3/s)	0.393	0.698	1.09	1.57	2.45	3.53	4.81	6.28
2.5	S (‰)	7.64	5.20	3.88	3.06	2.25	1.74	1.44	1.21
	Q (ft^3/s)	0.491	0.873	1.36	1.96	3.07	4.42	6.01	7.85
3.0	S (‰)	11.0	7.50	5.58	4.37	3.24	2.54	2.08	1.74
	Q (ft^3/s)	0.589	1.05	1.64	2.36	3.68	5.30	7.22	9.42
5.0	S (‰)	30.5	20.8	16.1	12.2	9.00	6.96	5.76	4.84
	Q (ft^3/s)	0.982	1.75	2.73	3.93	6.14	8.84	12.0	15.7

Conversion factors: 1 in. = 25.4 mm; 1 ft/s = 0.3048 m/s; 1 ft^3/s = 28.32 L/s = 0.02832 m^3/s.

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Appendix 9

Hydraulic Elements of Circular Conduits

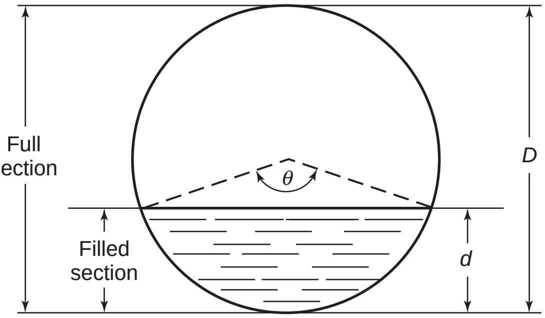
Central angle: $\cos\frac{1}{2}\theta = 1 - 2d/D$

Area: $\frac{D^2}{4}\left(\frac{\pi\theta}{360} - \frac{\sin\theta}{2}\right)$

Wetted perimeter: $\pi D\theta/360$

Hydraulic radius: $\frac{D}{4}\left(1 - \frac{360\sin\theta}{2\pi\theta}\right)$

Velocity: Manning's formula



Depth d/D	Area a/A	Hydraulic radius r/R	 R/r	 $(r/R)^{1/6}$	Velocity v/V for $N/n = 1.0$	Discharge q/Q	Roughness N/n
(1)	(2)	(3)	(4)	(5)	(6)[a]	(7)[a]	(8)
1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.00
0.900	0.949	1.192	0.839	1.030	1.124	1.066	0.94
0.800	0.858	1.217	0.822	1.033	1.140	0.988	0.88
0.700	0.748	1.185	0.843	1.029	1.120	0.838	0.85
0.600	0.626	1.110	0.900	1.018	1.072	0.671	0.83
0.500	0.500	1.000	1.00	1.000	1.000	0.500	0.81
0.400	0.373	0.857	1.17	0.975	0.902	0.337	0.79
0.300	0.252	0.684	1.46	0.939	0.776	0.196	0.78
0.200	0.143	0.482	2.07	0.886	0.615	0.088	0.79
0.100	0.052	0.254	3.94	0.796	0.401	0.021	0.82
0.000	0.000	…	…	…	…	0.000	…

[a] For values corrected for variations in roughness with depth multiply by roughness ratio *N/n* in column 8.

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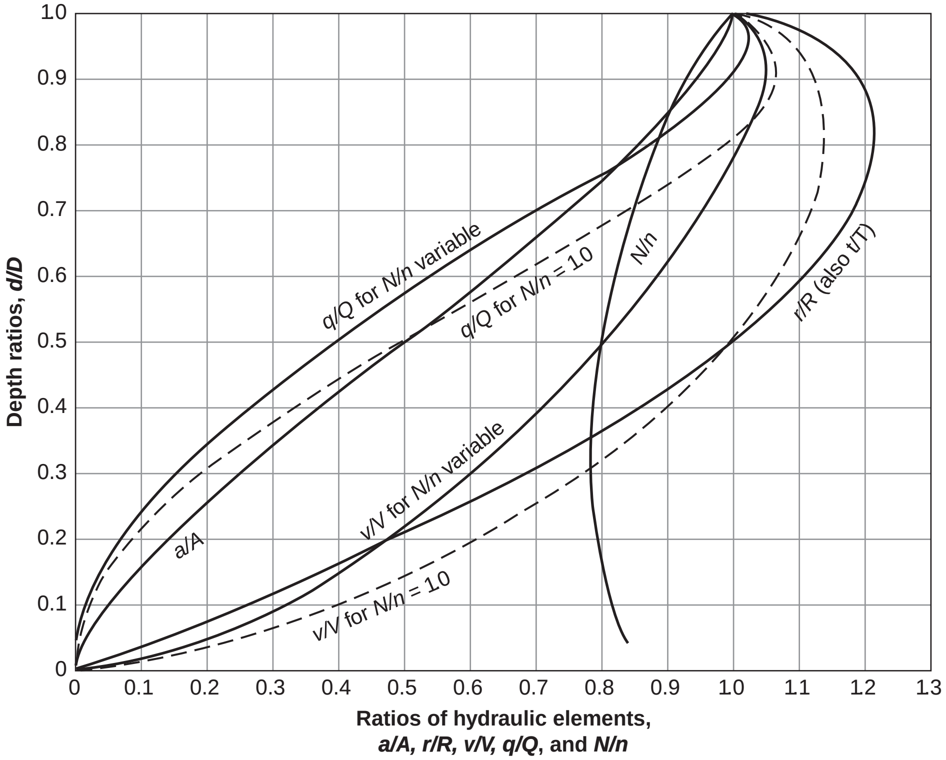
Appendix 10

Basic Hydraulic Elements of Circular Conduits for All Values of Roughness and Slope



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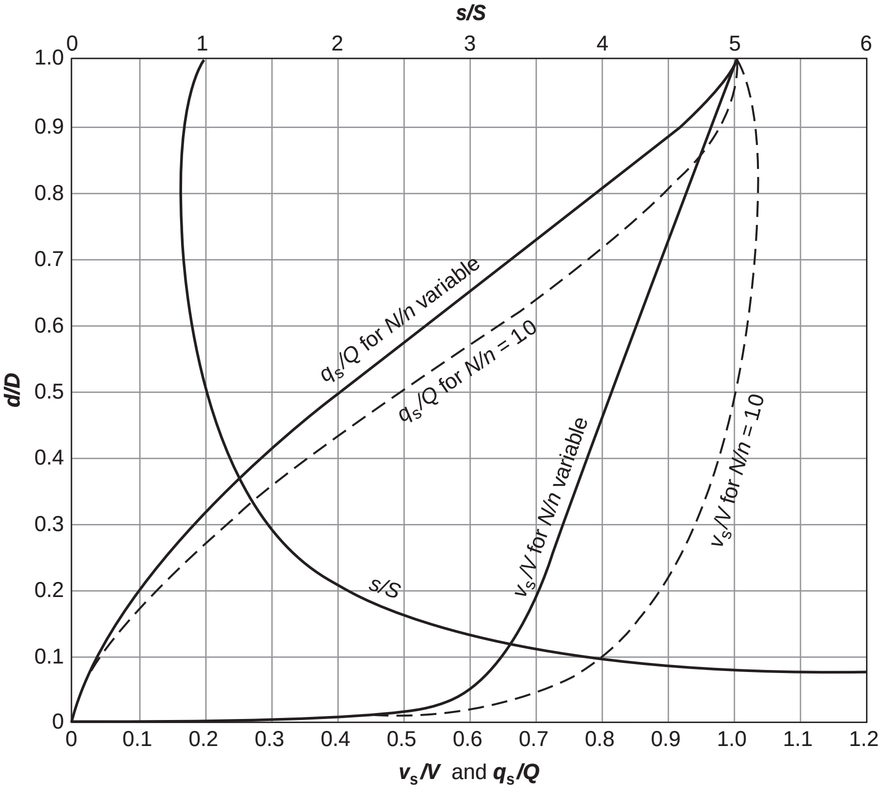
Appendix 11

Hydraulic Elements of Circular Conduits with Equal Self-Cleansing Properties at All Depths



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Appendix 12

Values of Hydraulic Elements in Circular Conduits

Values of h/D, $v_c/\sqrt{gD}$, and $[q/(A\sqrt{gD})]^2$ for varying values of d_c/D in circlar conduits

d_c/D	h/D	$v_c/\sqrt{gD}$	$[q/(A\sqrt{gD})]^2$
0.1	0.134	0.261	1.184×10^{-4}
0.2	0.270	0.378	2.86×10^{-3}
0.3	0.408	0.465	1.37×10^{-2}
0.4	0.550	0.553	4.18×10^{-2}
0.5	0.696	0.626	9.80×10^{-2}
0.6	0.851	0.709	1.97×10^{-1}
0.7	1.020	0.800	3.58×10^{-1}
0.8	1.222	0.919	6.23×10^{-1}
0.9	1.521	1.11	1.12

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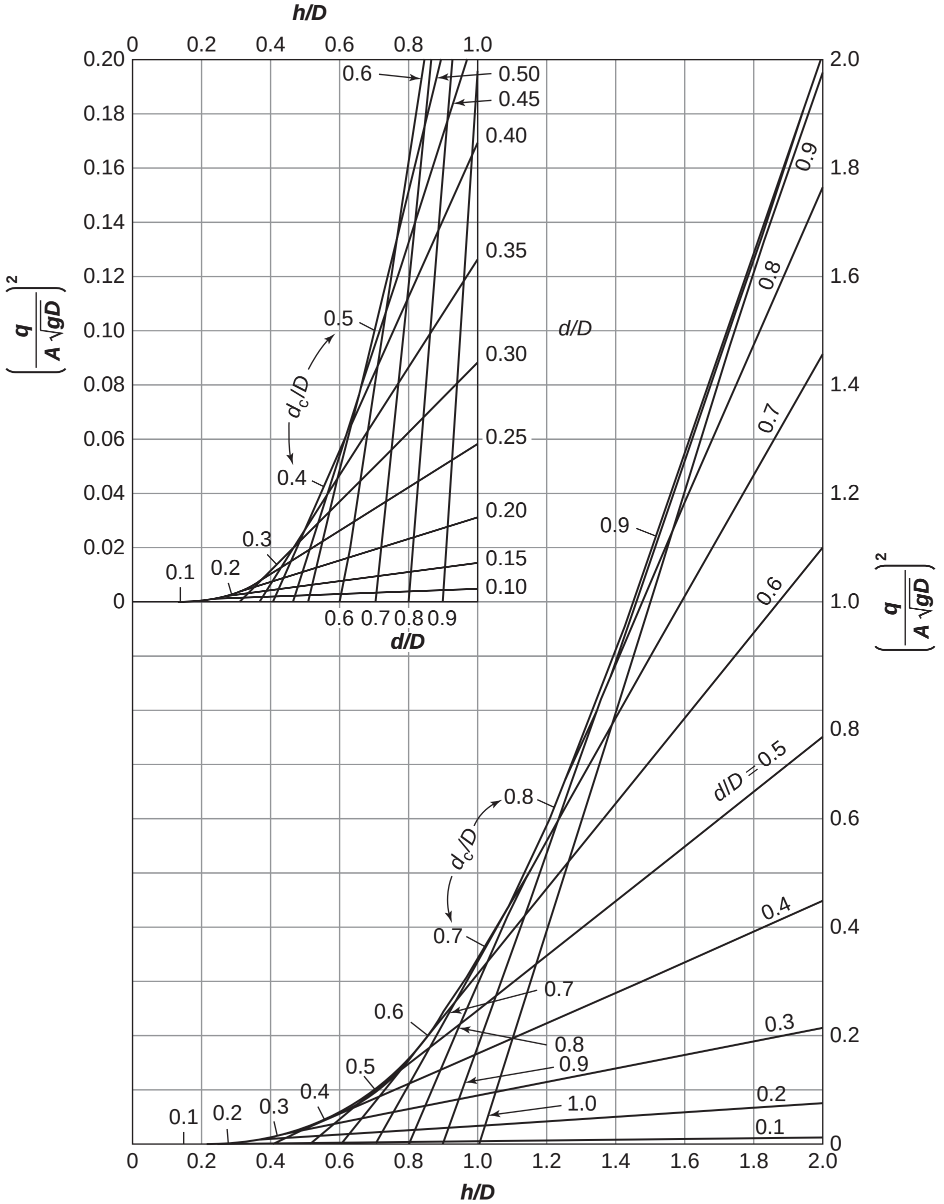
Appendix 13

Alternate Stages and Critical Depths of Flow in Circular Conduits



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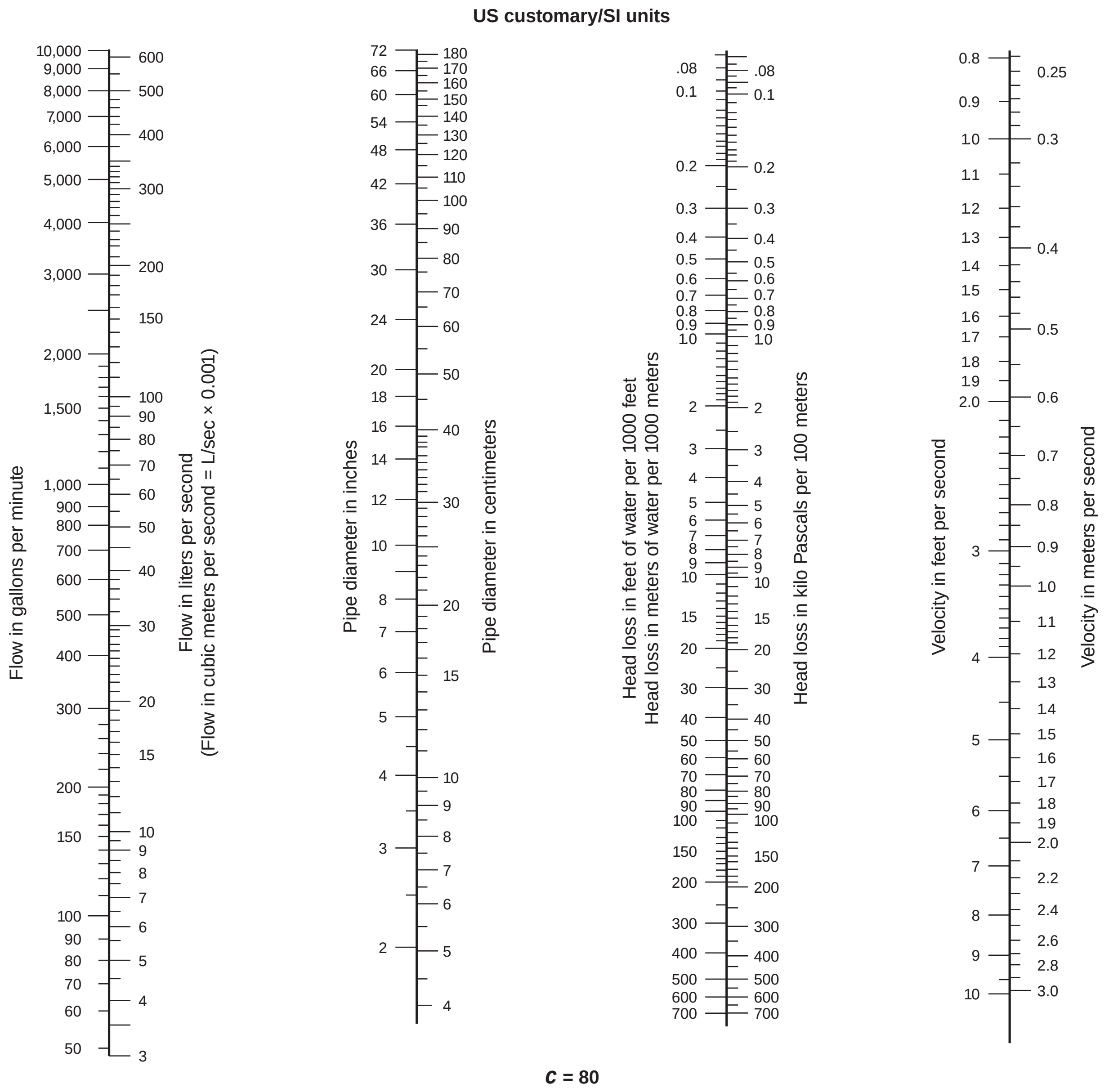
Appendix 14

Nomograms for Solution of Hazen–Williams Pipe Flow Equation



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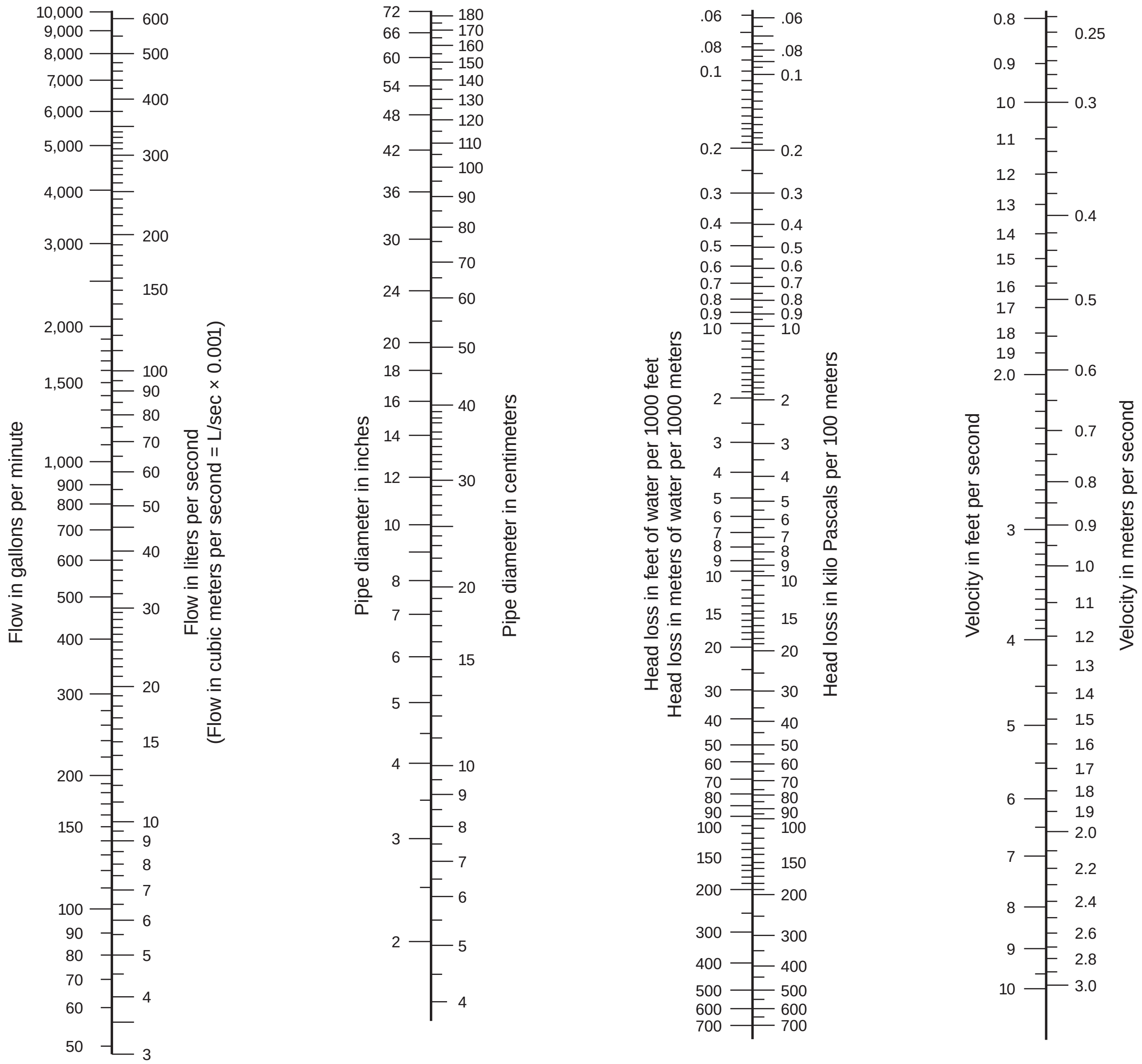
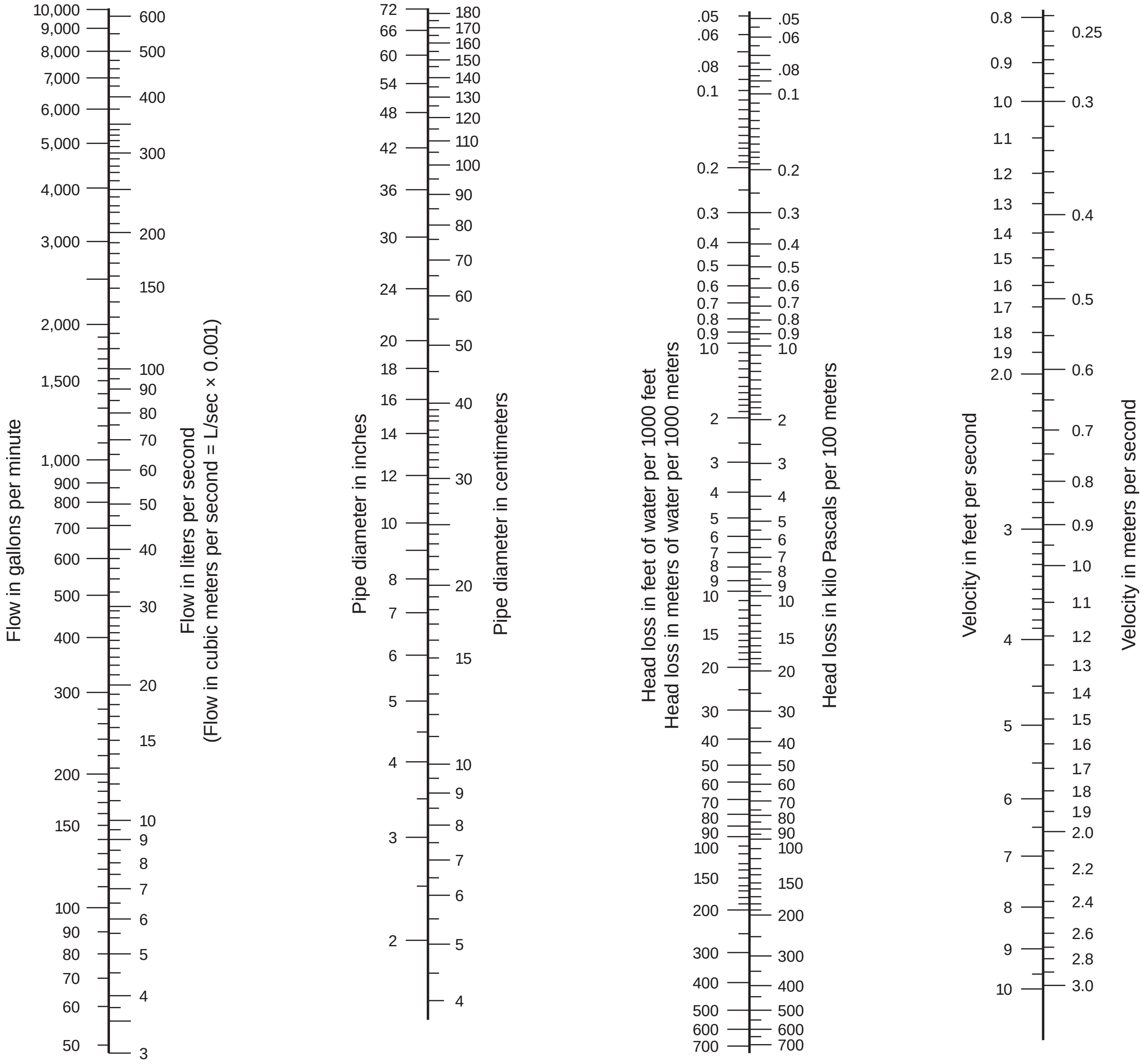
US customary/SI units



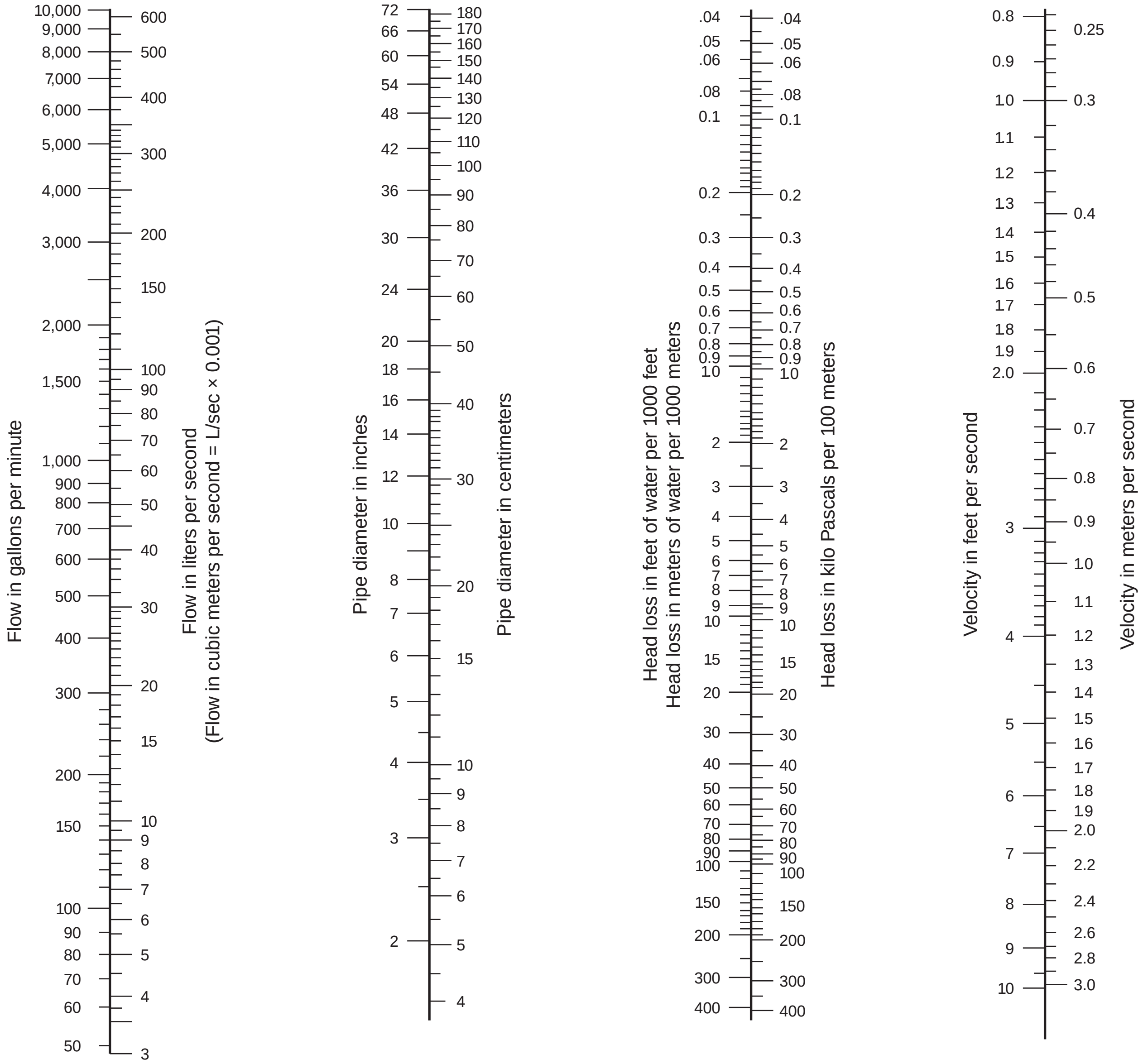
$C = 90$

US customary/SI units



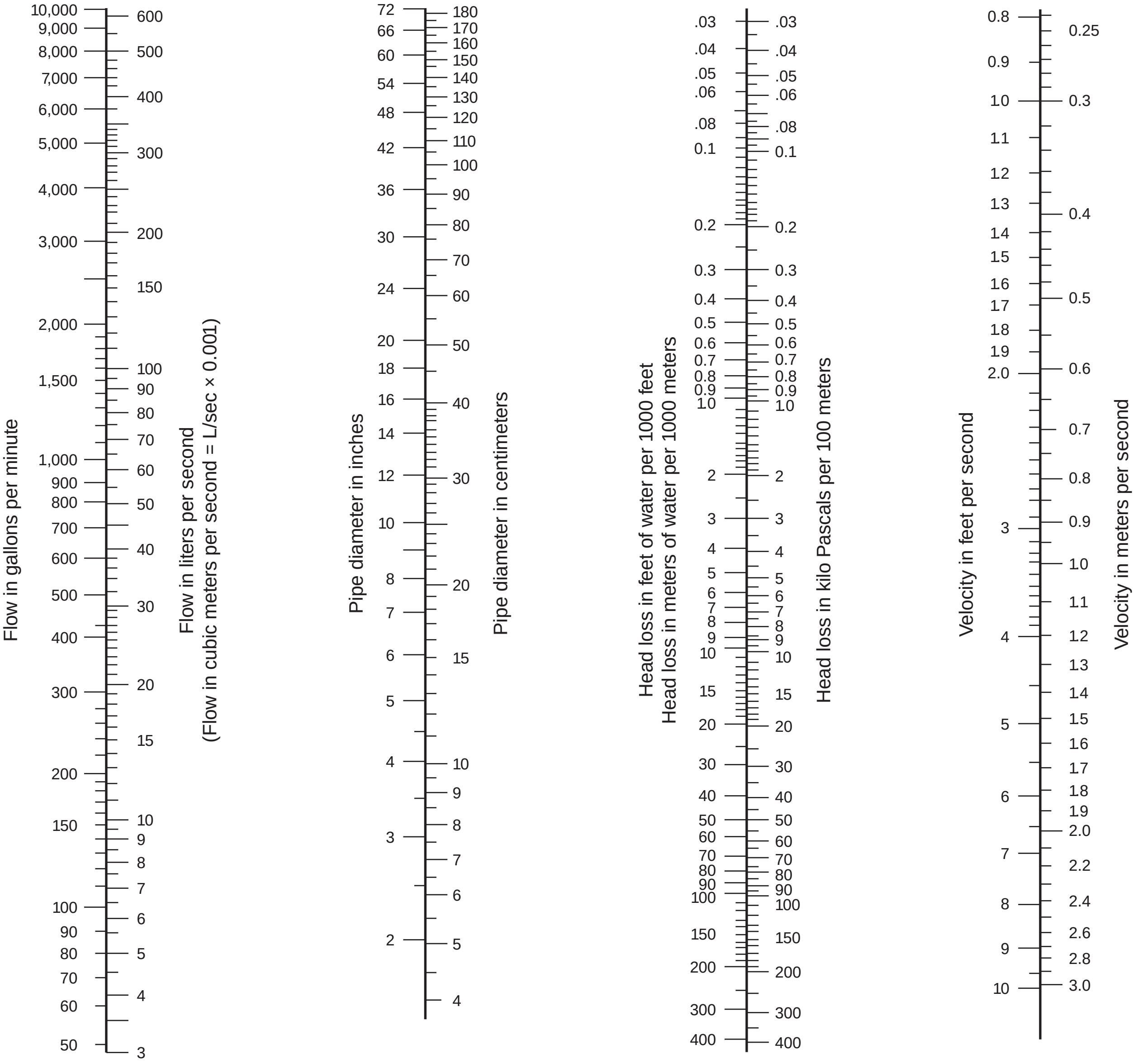
***C* = 100**

US customary/SI units



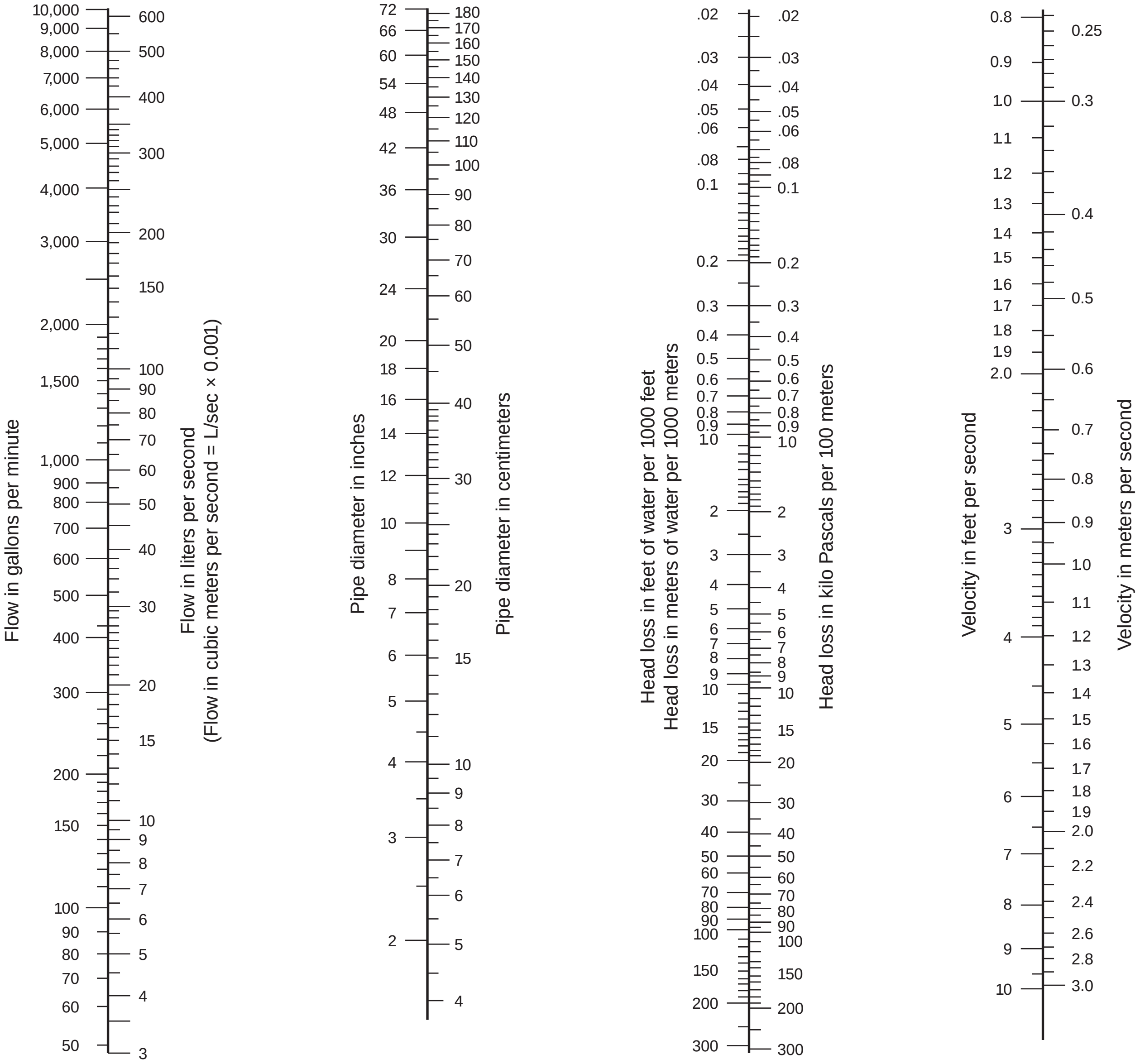
C **= 110**

US customary/SI units



C **= 130**

US customary/SI units

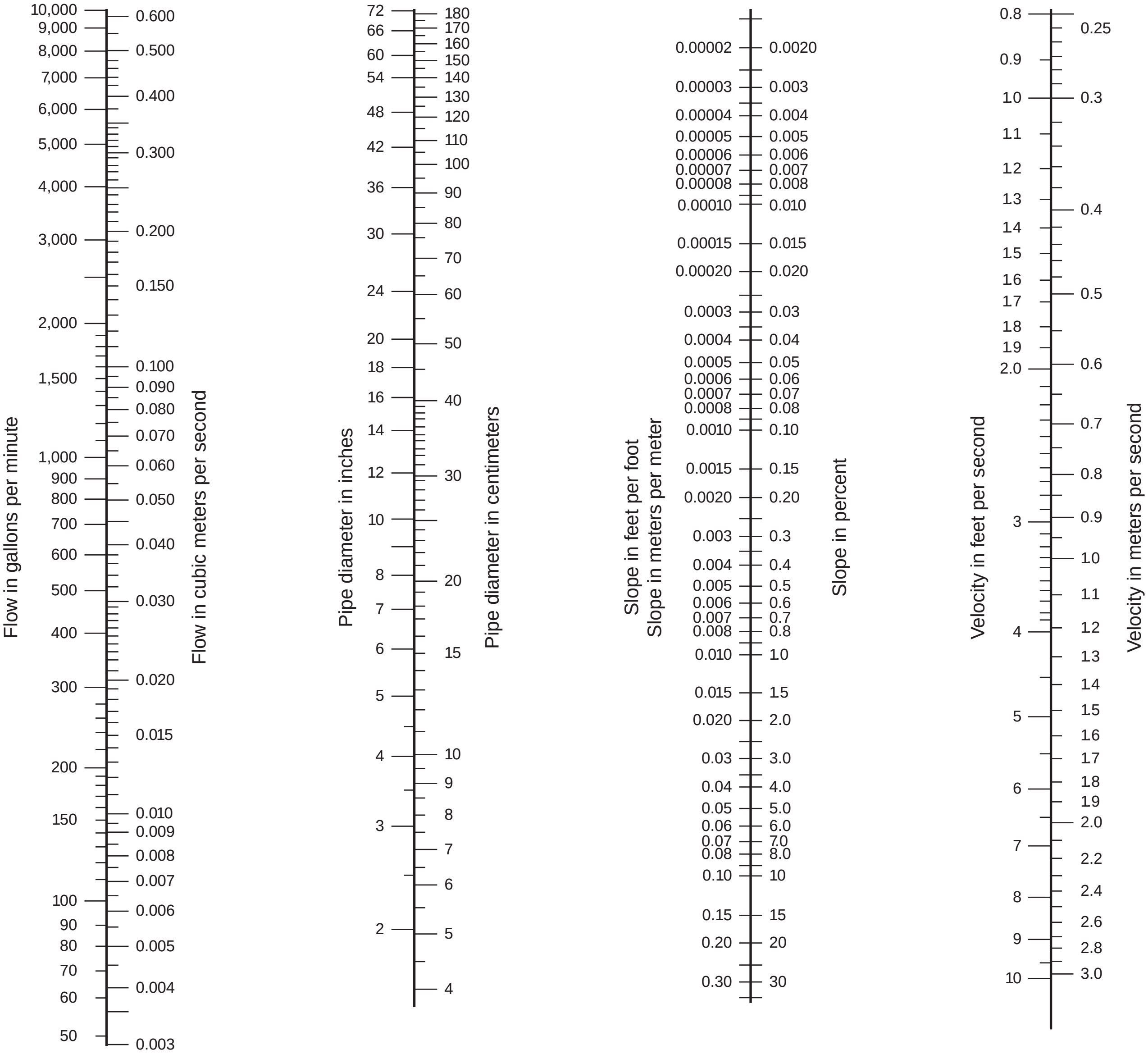


***C* = 150**

Appendix 15

Nomograms for Solution of Manning's Equation for Pipes Flowing Full

US customary/SI units



n **= 0.009**

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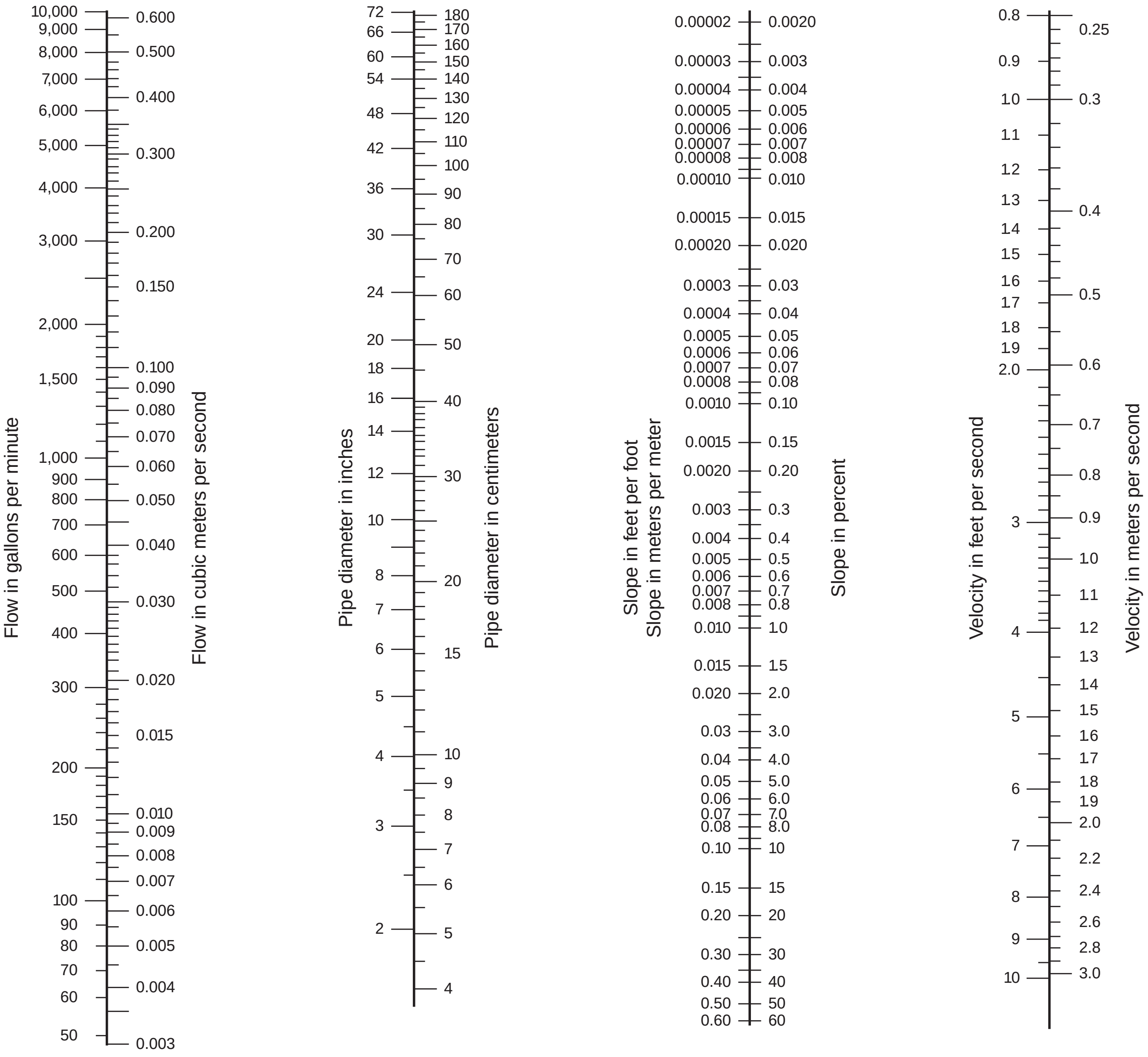
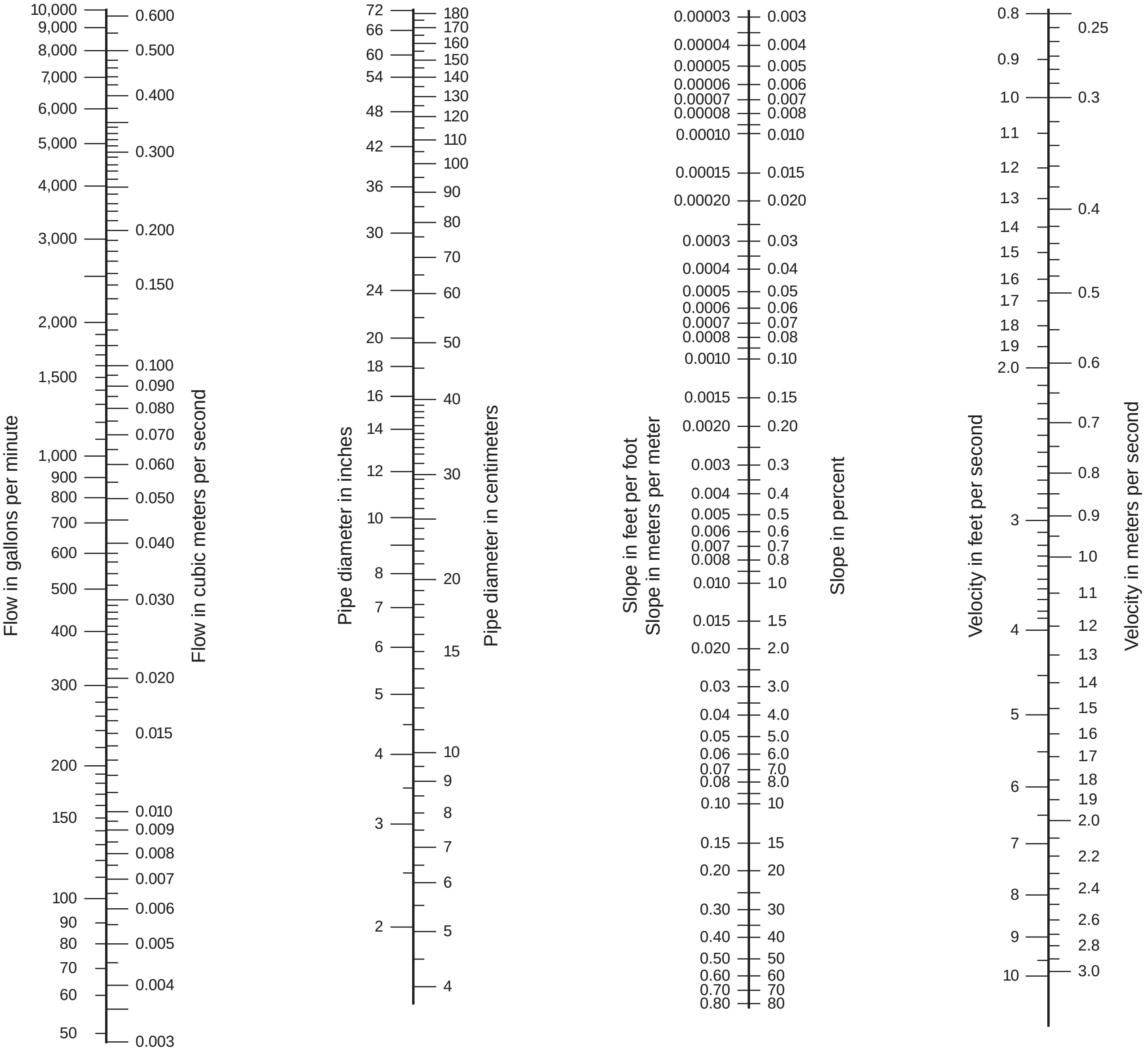
US customary/SI units



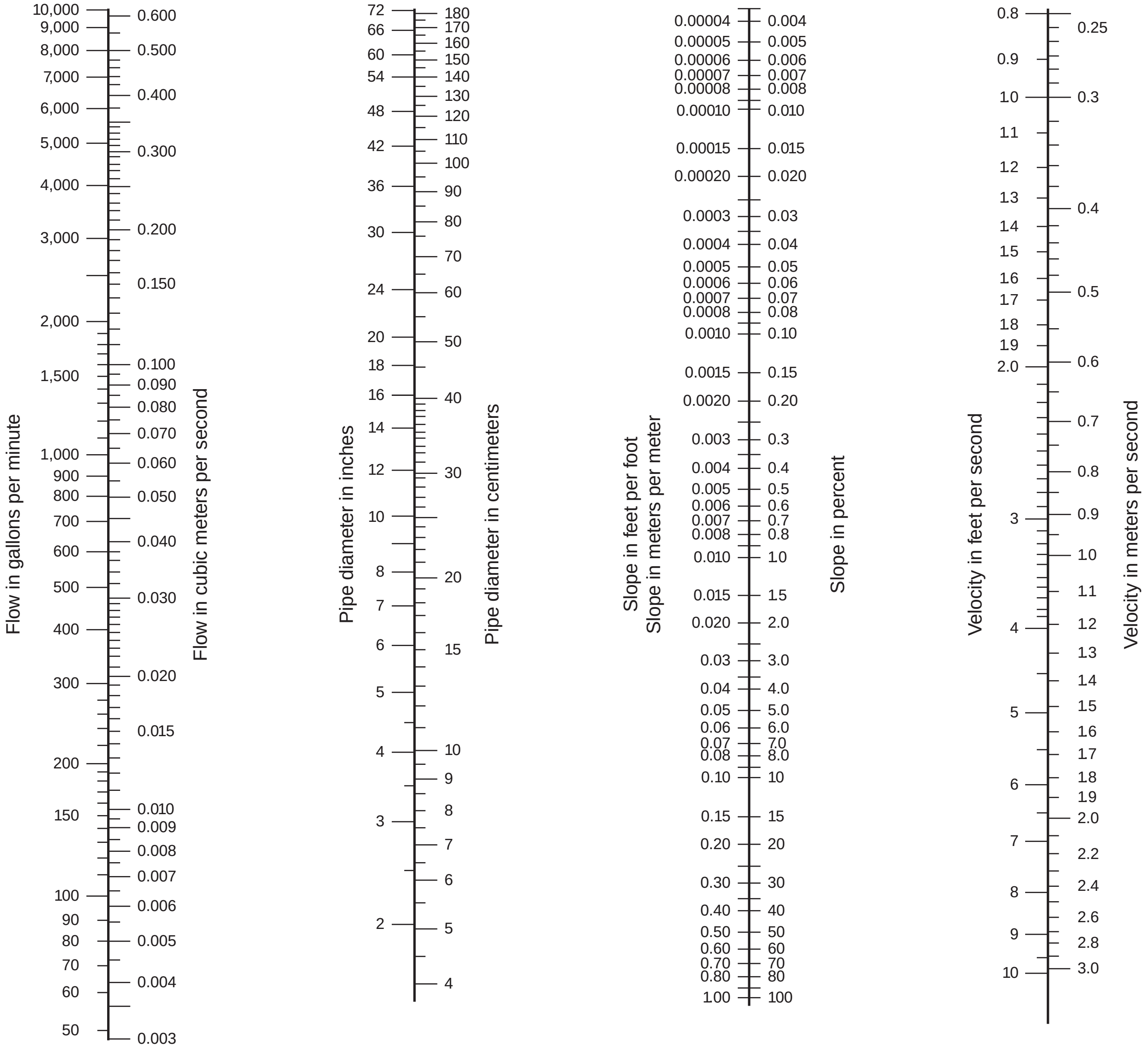
***n* = 0.011**

US customary/SI units



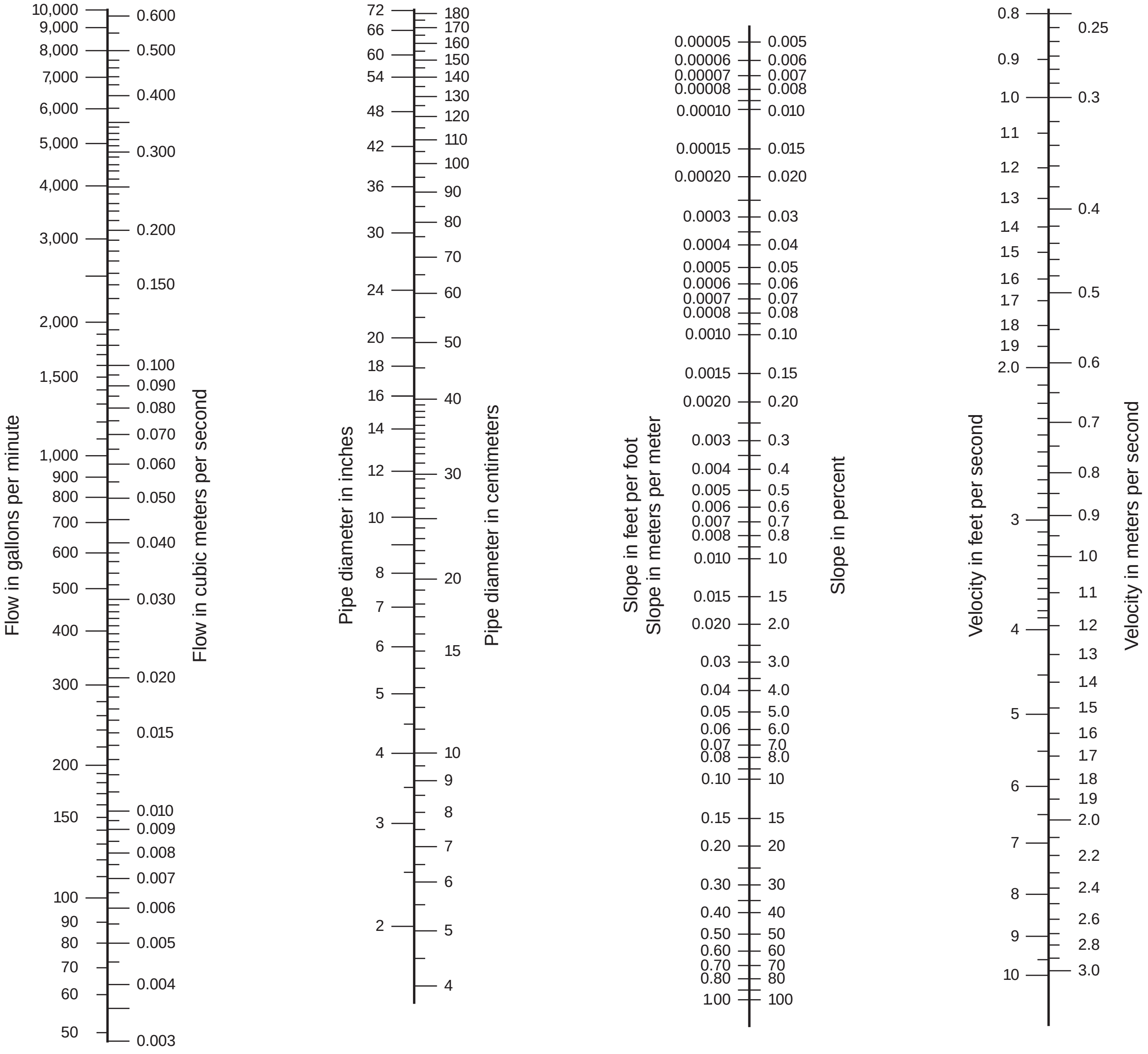
***n* = 0.013**

US customary/SI units



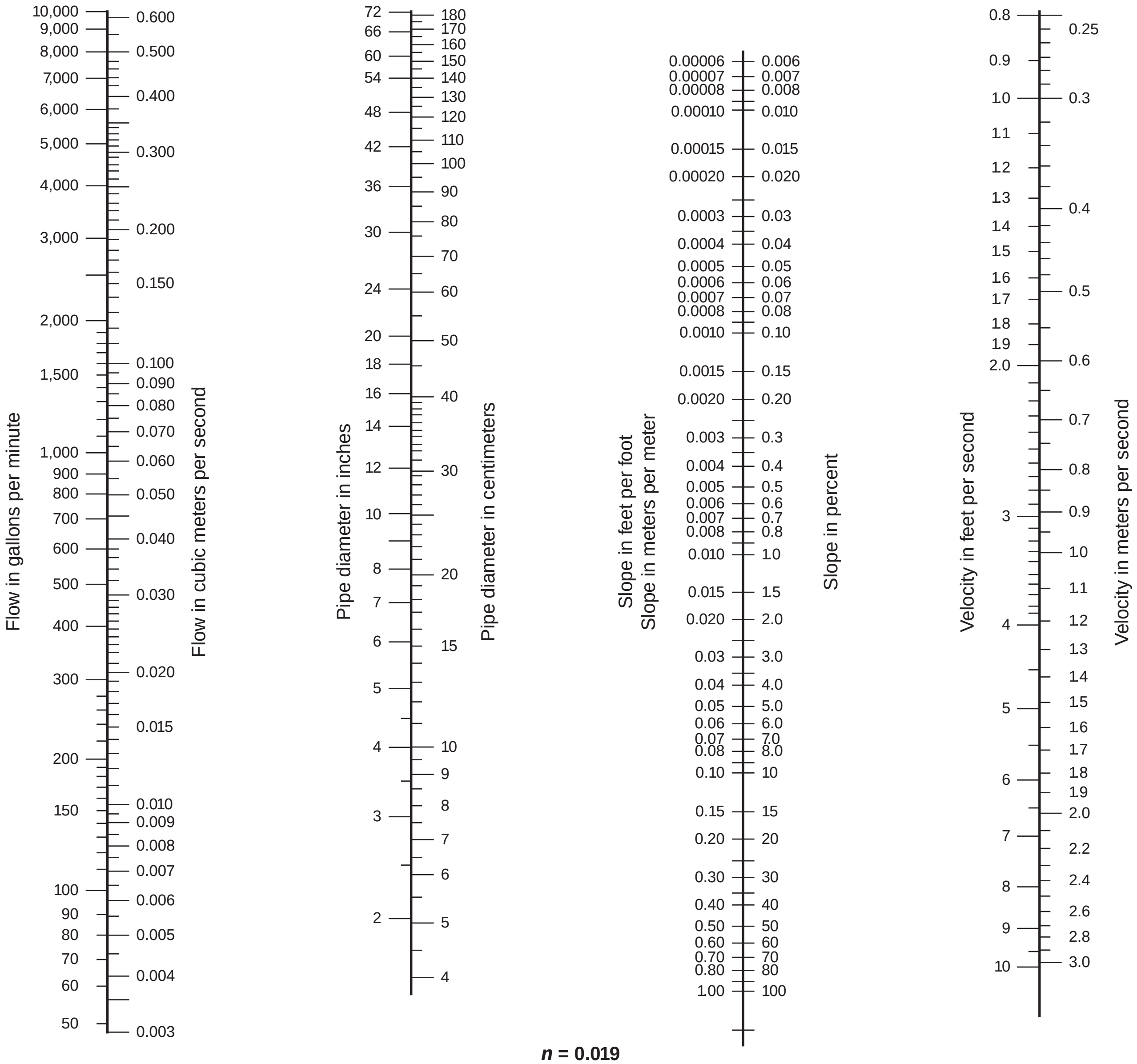
***n* = 0.015**

US customary/SI units



$n = 0.017$

US customary/SI units



***n* = 0.019**

Appendix 16

US Army Corps of Engineers Civil Works Construction Yearly Average Cost Index for Utilities

Year	Index*	Year	Index*
1967	100	1992	399.07
1968	104.83	1993	410.63
1969	112.17	1994	424.91
1970	119.75	1995	439.72
1971	131.73	1996	445.58
1972	141.94	1997	454.99
1973	149.36	1998	459.40
1974	170.45	1999	460.16
1975	190.49	2000	468.05
1976	202.61	2001	472.18
1977	215.84	2002	486.16
1978	235.78	2003	497.40
1979	257.20	2004	563.78
1980	277.60	2005	605.47
1981	302.25	2006	645.52
1982	320.13	2007	681.88
1983	330.82	2008	741.36
1984	341.06	2009	699.70
1985	346.12	2010	720.80
1986	347.33	2011	758.79
1987	353.35	2012	769.30
1988	369.45	2013	776.44
1989	383.14	2014	790.52
1990	386.75	2015	803.83
1991	392.35		

*US ACE, Yearly Average Cost Index for Utilities. In: *Civil Works Construction Cost Index System Manual*, 1110-2-1304, U.S. Army Corps of Engineers, Washington, DC, 2014. (PDF file is available on the Internet at http://www.nww.usace.army.mil/Missions/CostEngineering.aspx).

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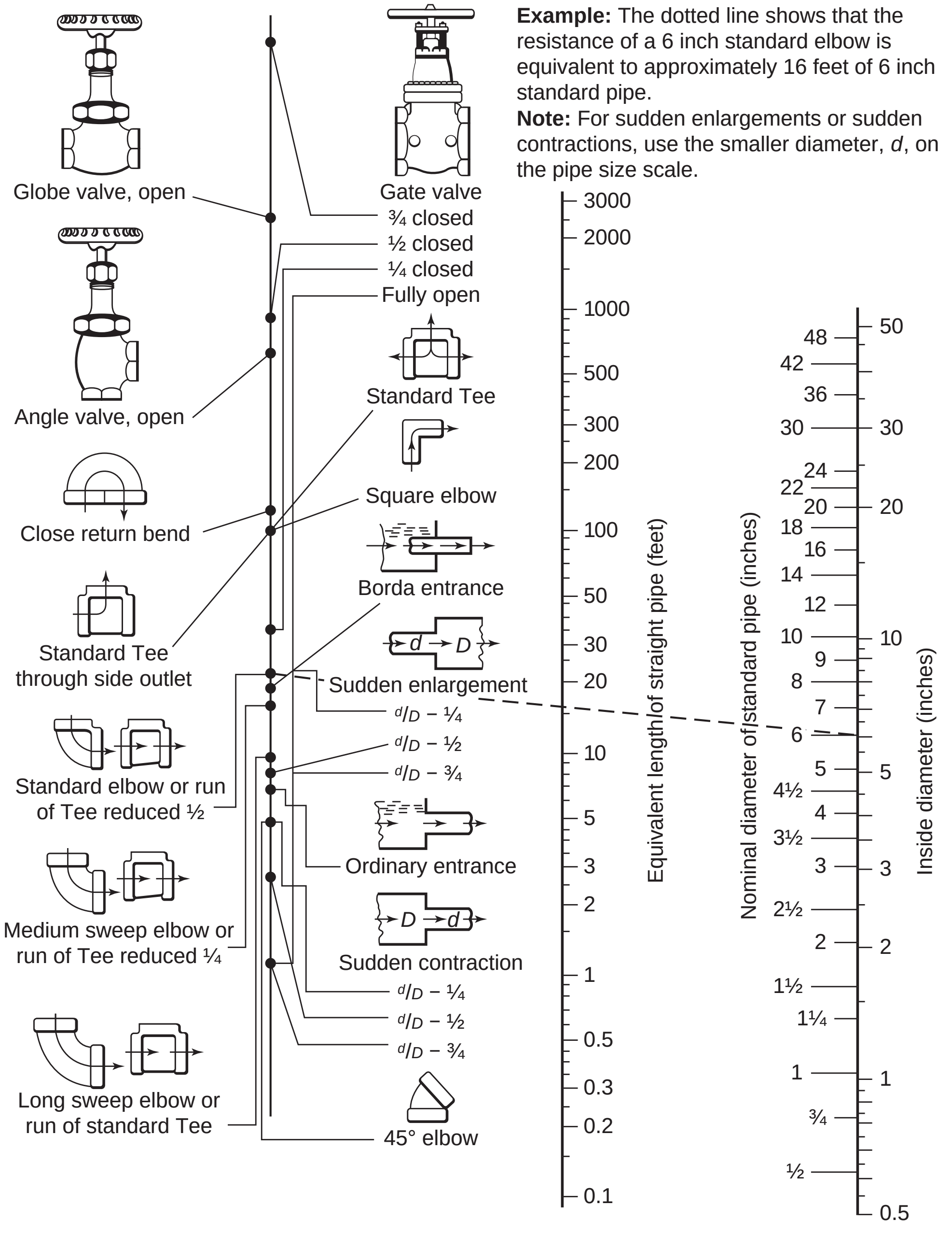
Appendix 17

Equivalent Pipe Length to Head Loss in Fittings



Example: The dotted line shows that the resistance of a 6 inch standard elbow is equivalent to approximately 16 feet of 6 inch standard pipe.
Note: For sudden enlargements or sudden contractions, use the smaller diameter, *d*, on the pipe size scale.

After Seelye, E. E, *Data Book for Civil Engineers*, *Volume 1, Design*, John Wiley & Sons, Inc., New York, 1960.
Conversion factors: 1 foot = 1 ft = 0.3048 m; 1 inch = 1 in. = 254 mm

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Appendix 18

Absorption Coefficients of Common Gases in Water

Absorption coefficients (mL/L) (mL of gas, reduced to 0ºC and 760 mm Hg, per L of water when partial pressure of gas is 760 mm Hg)

Gas	Mol. wt	Weight at 0°C and 760 mm Hg (g/L)	Absorption coefficients (°C)				Boiling Point (°C)
			0	10	20	30	
Hydrogen, H_2	2.016	0.08988	21.4	19.6	18.2	17.0	−253
Methane, CH_4	16.014	0.7168	55.6	41.8	33.1	27.6	−162
Nitrogen, N_2	28.01	1.251	23.0	18.5	15.5	13.6	−196
Oxygen, O_2	32.00	1.429	49.3	38.4	31.4	26.7	−183
Ammonia, NH_3[a]	17.03	0.7710	1,300	910	711	–	−33.4
Hydrogen sulfide, H_2S[a]	34.08	1.539	4,690	3,520	2,670	…	−61.8
Carbon dioxide, CO_2[a]	44.01	1.977	1,710	1,190	878	665	−78.5
Ozone, O_3	48.00	2.144	641	520	368	233	−112
Sulfur dioxide, SO_2[a]	64.07	2.927	79,800	56,600	39,700	27,200	−10.0
Chlorine, Cl_2[a]	70.91	3.214	4,610	3,100	2,260	1,770	−34.6
Air[b]	…	1.2928	28.8	22.6	18.7	16.1	…

[a] Total solubility.

[b] At sea level dry air contains 78.08% N_2, 20.95% O_2, 0.93% A, 0.03% CO_2, and 0.01% other gases by volume. For ordinary purposes it is assumed to be composed of 79% N_2 and 21% O_2.

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Appendix 19

Saturation Values of Dissolved Oxygen

Saturation values of dissolved oxygen in fresh and sea water exposed to an atmos-phere containing 20.9% oxygen under a pressure of 760 mm of mercury[a]

Temperature (°C)	Dissolved oxygen (mg/L) for stated concentrations of chloride (mg/L)					Difference per 1,000 mg/L chloride
	0	5,000	10,000	15,000	20,000	
0	14.7	13.8	13.0	12.1	11.3	0.165
1	14.3	13.5	12.7	11.9	11.1	0.160
2	13.9	13.1	12.3	11.6	10.8	0.154
3	13.5	12.8	12.0	11.3	10.5	0.149
4	13.1	12.4	13.8	11.0	10.3	0.144
5	12.8	12.1	11.4	10.7	10.0	0.140
6	12.5	11.8	11.0	10.4	9.8	0.135
7	12.1	11.5	10.8	10.2	9.6	0.130
8	11.8	11.2	10.6	10.0	9.4	0.125
9	11.6	11.0	10.4	9.7	9.1	0.121
10	11.3	10.7	10.1	9.5	8.9	0.118
11	11.0	10.4	9.9	9.3	8.7	0.114
12	10.8	10.2	9.7	9.1	8.6	0.110
13	10.5	10.0	9.4	8.9	8.4	0.107
14	10.3	9.7	9.2	8.7	8.2	0.104
15	10.0	9.5	9.0	8.5	8.0	0.100
16	9.8	9.3	8.8	8.4	7.9	0.098
17	9.6	9.1	8.7	8.2	7.7	0.095
18	9.4	9.0	8.5	8.0	7.6	0.092
19	9.2	8.8	8.3	7.9	7.4	0.089
20	9.0	8.6	8.1	7.7	7.3	0.088
21	8.8	8.4	8.0	7.6	7.1	0.086
22	8.7	8.3	7.8	7.4	7.0	0.084
23	8.5	8.1	7.7	7.3	6.8	0.083
24	8.3	7.9	7.5	7.1	6.7	0.083
25	8.2	7.8	7.4	7.0	6.5	0.082
26	8.0	7.6	7.2	6.8	6.4	0.080
27	7.9	7.5	7.1	6.7	6.3	0.079
28	7.7	7.3	6.9	6.6	6.2	0.078
29	7.6	7.2	6.8	6.5	6.1	0.076
30	7.4	7.1	6.7	6.3	6.0	0.075

[a] For barometric pressures other than 760 mm of Hg, the solubilities vary approximately in proportion to the ratio of the actual pressure to the standard pressure.

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Appendix 20

Transfer Coefficients and Liquid Film Thickness for Gases

Transfer coefficient and liquid film thickness for gases of low solubility in water

Gas	Temperature (°C)	Test condition	Transfer coefficient, $k_{d(l)}$ (cm/h)	Diffusion coefficient, k_d (cm^2/h)	Film thickness, $k_d/k_{d(l)}$ (cm)
		Plane surfaces in contact with 50 mL of water at stated rate of stirring			
O_2	25	0 rpm	0.41	9.4×10^{-2}	2.3×10^{-1}
		76 rpm	1.20		7.8×10^{-2}
		171 rpm	3.00		3.1×10^{-2}
		486 rpm	5.43		1.7×10^{-2}
		1,025 rpm	7.64		1.2×10^{-2}
CO_2	25	0 rpm	1.46	7.4×10^{-2}	5.1×10^{-2}
		93 rpm	1.65		4.5×10^{-2}
		171 rpm	2.73		2.7×10^{-2}
		486 rpm	4.84		1.5×10^{-2}
		1,025 rpm	8.35		8.9×10^{-2}
		Gas bubbles of stated size in wate			
O_2	25	10 mL	20	9.4×10^{-2}	4.7×10^{-3}
	20–21	1.51–2.13 mL	88		1.1×10^{-3}
		Water droplets of stated size			
CO_2	24	0.1 mL	260	7.4×10^{-2}	2.8×10^{-3}

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Appendix 21

Solubility Constants of Important Chemical Substances in Water at 25^{o}C

Substance	Ion product	pK_s = −log K_s	Application
$Al(OH)_3$	$(Al^{3+})(OH^-)^3$	32.9	Coagulation
$AlPO_4$	$(A1^{3+})(PO_4{}^{3-})$	22.0	Phosphate removal
$CaCO_3$	$(Ca^{2+})(CO_3{}^{2-})$	8.32	Softening; corrosion control
CaF_2	$(Ca^{2+})(F^-)^2$	10.41	Fluoridation
$Ca(OH)_2$	$(Ca^{2+})(OH^-)^2$	5.26	Softening
$Ca_3(PO_4)_2$	$(Ca^{2+})^3(PO_4{}^{2-})^2$	26.0	Softening; phosphate cycle
$CaHPO_4$	$(Ca^{2+})(HPO_4{}^{3-})$	7.0	Phosphate cycle
$CaSO_4.\ 2H_2O$	$(Ca^{2+})(SO_4{}^{2-})$	4.62	Hardness; scale formation
$Cu(OH)_2$	$(Cu^{2+})(OH^-)^2$	19.25	Algae control
$Fe(OH)_2$	$(Fe^{2+})(OH^-)^2$	15.1	Corrosion; deferrization
$FeCO_3$	$(Fe^{2+})(CO_3{}^{2-})$	10.68	Iron cycle
FeS	$(Fe^{2+})(S^{2-})$	17.3	Anaerobic corrosion
$Fe\ (OH)_3$	$(Fe^{3+})(OH^-)^3$	38.04	Coagulation; deferrization
$FePO_4$	$(Fe^{3+})(PO_4{}^{2-})$	21.9	Phosphate cycle
$MgCO_3.\ 3H_2O$	$(Mg^{2+})(CO_3{}^{2-})$	4.25	Hardness
MgF_2	$(Mg^{2+})(F^-)^2$	8.18	Defluoridation
$Mg(OH)_2$	$(Mg^{2+})(OH^-)^2$	10.74	Softening
$MnCO_3$	$(Mn^{2+})(CO_3{}^{2-})$	10.06	Manganese cycle
$Mn(OH)_2$	$(Mn^{2+})(OH^-)^2$	12.8	Demanganization
$Zn(OH)_2$	$(Zn^{2+})(OH^-)^2$	17.15	Corrosion

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Appendix 22

$CaCO_3$ Solubility and Carbonate Equilibria in Terms of Dissociation Constants, p*K* = (−log *K*)

		Temperature (°C)						
	Reaction[a]	5	10	15	20	25	40	60
1.	$CO_2(s) + H_2O = H_2CO_3*; K$	1.20	1.27	1.34	1.41	1.47	1.64	1.8
2.	$H_2CO_3* = HCO_3^- + H^+; K_1$	6.52	6.46	6.42	6.38	6.35	6.30	6.30
3.	$HCO_3^- = CO_3^- + H^+; K_2$	10.56	10.49	10.43	10.38	10.33	10.22	10.14
4*a*.	$CaCO_3(s) = Ca^{2+} + CO_3^-; K_s$	8.09	8.15	8.22	8.28	8.34	8.51	8.74
4*b*.	$CaCO_3(s) + H^+ = Ca^{2+} + HCO_3^-: (K_s/K_2)$	−2.47	−2.34	−2.21	−2.10	−1 99	−1.71	−1.40
4*c*.	$CaCO_3(s) + H_2CO_3* = Ca^{2+} + 2HCO_3^-; (K_sK_1/K_2)$	4.05	4.12	4.21	4.28	4.36	4.59	4.90
5.	$H_2O = H^+ + OH^-; K_w$	14.73	14.54	14.35	14.17	14.00	13.54	13.02

Salinity Corrections for Equilibrium Constants[b]

$pK_1{}^I = pK_1 - (\mu)^{0.5}/[1 + 1.4(\mu)^{0.5}]$ $\quad$ $pK_2{}^I = pK_2 - 2(\mu)^{0.5}/[1+1.4(\mu)^{0.5}]$

$pK_s{}^I = pK_s - 4(\mu)^{0.5}/[1 + 3.9(\mu)^{0.5}]$ $\quad$ $\log K_s{}^I/K_2{}^I = \log K_s/K_2 + 2.5(\mu))^{0.5}/[\ 1 + 5.3(\mu)^{0.5} + 5.5(\mu)^{0.5}]$

Estimation of μ

$\mu \simeq 2.5 \times 10^{-5}\ S_d$, where S_d is the total dissolved solids content of the water, or

$\mu \simeq 4H - T$, where H = total hardness in moles/L; and T = alkalinity in eq/L (10^3 me/L)

[a] H_2CO_3* refers to the sum of dissolved CO_2 and H_2CO_3. Because $[H_2CO_3] \ll [CO_2]$, $[CO_2]$ is essentially equal to $[H_2CO_3*]$.

[b] K and K^I are thermodynamic and operational constants, respectively. The operational constants can be used with mass law expressions containing concentration terms, with the exception of H^+, which is always expressed in activities.

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Appendix 23

Atomic Numbers, Weights, and Valence of Chemical Elements

Atomic numbers, weights, and valences of chemical elements[a]

Element	Symbol	Atomic number	International atomic weight	Valence
Aluminum	Al	13	26.98	3
Arsenic	As	33	74.91	3, 5
Barium	Ba	56	137.36	2
Boron	B	5	10.82	3
Bromine	Br	35	79.92	1, 3, 5, 7
Cadmium	Cd	48	112.41	2
Calcium	Ca	20	40.08	2
Carbon	C	6	12.01	2, 4
Chlorine	Cl	17	35.46	1, 3, 5, 7
Chromium	Cr	24	52.01	2, 3, 6
Cobalt	Co	27	58.94	2, 3
Copper	Cu	29	63.54	1, 2
Fluorine	F	9	19.00	1
Gold (*aurum*)	Au	79	197.2	1, 3
Hydrogen	H	1	1.008	1
Iodine	I	53	126.92	1, 3, 5, 7
Iron (*ferrum*)	Fe	26	55.85	2, 3
Lead (*plumbum*)	Pb	82	207.21	2, 4
Magnesium	Mg	12	24.32	2
Manganese	Mn	25	54.93	2, 3, 4, 6, 7
Mercury (*hydrargyrum*)	Hg	80	200.61	1, 2
Nickel	Ni	28	58.69	2, 3
Nitrogen	N	7	14.01	3, 5
Oxygen	O	8	16.00	2
Phosphorus	P	15	30.98	3, 5
Platinum	Pt	78	195.23	2, 4
Potassium (*kalium*)	K	19	39.10	1
Selenium	Se	34	78.96	2, 4, 6
Silicon	Si	14	28.09	4
Silver (*argentum*)	Ag	47	107.88	1
Sodium (*natrium*)	Na	11	23.00	1
Strontium	Sr	38	87.63	2
Sulfur	S	16	32.07	2, 4, 6
Tin (*stannum*)	Sn	50	118.70	2, 4
Zinc	Zn	30	65.38	2

[a] Elements encountered in radioactive wastes are not included. For a complete list, see *Handbook of Chemistry and Physics*, Chemical Rubber Publishing Company, Cleve-land, OH

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Appendix 24

Designations and Size of Openings of US Sieve Series

Sieve designation, number[a]	Size of opening (mm)	Sieve designation, number	Size of opening (mm)
200	0.074	20	0.84
140	0.105	(18)	(1.00)
100	0.149	16	1.19
70	0.210	12	1.68
50	0.297	8	2.38
40	0.42	6	3.36
30	0.59	4	4.76

[a] Approximately the number of meshes per inch.

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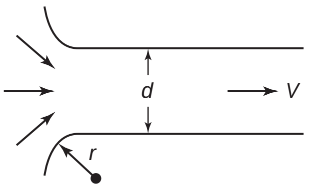
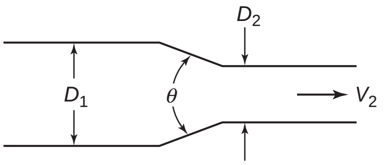
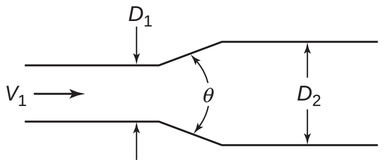
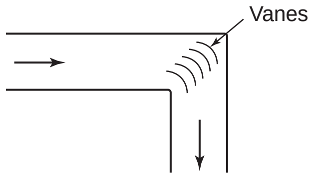
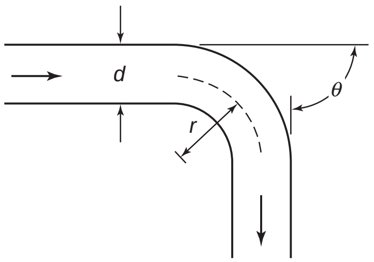
Appendix 25

Loss Coefficients for Transitions and Fittings

Description	Sketch	Additional data	K	
Pipe entrance		r/d	K_e	
		0.0	0.50	
		0.1	0.12	
$h_f = K_e\ V^2/2g$		>0.2	0.03	
Contraction			K_c	K_c
		D_2/D_1	$\theta = 60°$	$\theta = 180°$
		0.0	0.08	0.50
		0.20	0.08	0.49
$h_f = K_c V_2^2/2g$		0.40	0.07	0.42
		0.60	0.06	0.32
		0.80	0.05	0.18
		0.90	0.04	0.10
Expansion			K_E	K_E
		D_1/D_2	$\theta = 10°$	$\theta = 180°$
		0.0		1.00
		0.20	0.13	0.92
		0.40	0.11	0.72
$h_f = K_E V_1^2/2g$		0.60	0.06	0.42
		0.80	0.03	0.16
90° miter bend		Without vanes		$K_b = 1.1$
		With vanes		$K_b = 0.2$
$h_f = K_b V^2/2g$				
		r/d	K_b	K_b
Smooth bend			$\theta = 45°$	$\theta = 90°$
		1	0.10	0.35
		2	0.09	0.19
		4	0.10	0.16
		6	0.12	0.21
$h_f = K_b V^2/2g$				

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Description	Sketch	Additional data	K
Threaded pipe fittings	Globe valve-wide open		$K_v = 10.0$
	Angle valve-wide open		$K_v = 5.0$
	Gate valve-wide open		$K_v = 0.2$
	Gate valve-half open		$K_v = 5.6$
	Return bend		$K_b = 2.2$
	Tee		$K_t = 1.8$
	90° elbow		$K_b = 0.9$
	45° elbow		$K_b = 0.4$

Appendix 26

Constants for Langelier Saturation Index Determination

		$pK_2 - pK_s$			
Ionic strength	TDS (mg/L)	0°C	10°C	20°C	25°C
0.000	0	2.45	2.23	2.02	1.94
0.002	80	2.62	2.40	2.19	2.11
0.005	200	2.71	2.49	2.28	2.20
0.010	400	2.81	2.59	2.38	2.30
0.015	600	2.88	2.66	2.45	2.37
0.020	800	2.93	2.71	2.50	2.42

$$\text{Langelier Saturation Index (I)} = \text{pH} - \text{pHs}$$

pHs is pH of saturation, that is the pH of water, with the same calcium concentration, alkalinity, temperature, and ionic strength, would have if it was in equilibrium with calcium carbonate. When the Langelier Saturation Index is less than zero, protective scale would not be deposited, any existing scale would be removed, and the risk of corrosion exists. When the Langelier Saturation Index is greater than zero, a protective scale would form. However, it should be noted that the actual numerical value of Langelier Saturation Index is not by itself an indication of the magnitude of these effects.

The pH of saturation, pH_s, can be computed as follows. Assuming that it falls between 6.5 and 9.5, and that the total dissolved solids concentration is less than 2,000 mg/L:

$$pH_s = pCa^{2+} + (pK_2 - pK_s) + p(\text{Alk}),$$

where Ca^{2+} = molarity of calcium, K_2 = the dissociation equilibrium constant for bicarbonate adjusted for temperature and ionic strength, K_s = solubility product constant for calcium carbonate at the given temperature and ionic strength, (Alk) = alkalinity in equivalents per liter, and the *p* function, as usual, indicates the negative logarithm of a value. Values for $pK_2 - pK_s$ are given for various temperatures and total dissolved solids concentrations (TDS) in this appendix.

A procedure should be used if the calculated value for pH_s is outside the range 6.5–9.5, or if the ionic strength is above that, for which data are given in the table. An example of the calculation of the Langelier index is presented in Example 21.1.

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